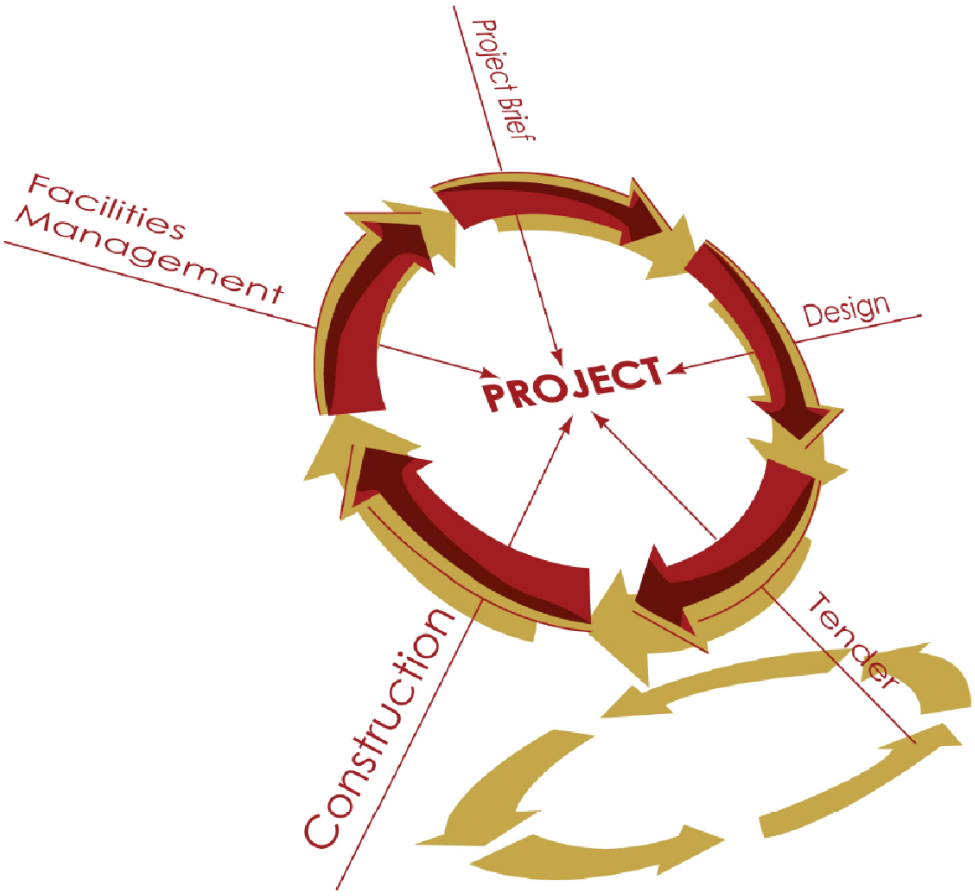
MCRJ SPECIAL ISSUE Volume 13 | No. 2 | 2021 eISSN 2590 - 4140

Malaysian Construction Research Journal

ENGINEERING TECHNOLOGY INTERNATIONAL CONFERENCE 2020 (ETIC 2020)



CREAM CONSTRUCTION RESEARCH INSTITUTE OF MALAYSIA
CIDB MALAYSIA

MALAYSIAN CONSTRUCTION RESEARCH JOURNAL (MCRJ)

SPECIAL ISSUE Vol. 13 | No. 2 | 2021

Engineering Technology International Conference 2020 (ETIC 2020)

The Malaysian Construction Research Journal is indexed in
Scopus Elsevier

eISSN No.: 2590 – 4140

Construction Research Institute of Malaysia (CREAM)
Level 29, Sunway Putra Tower,
No. 100, Jalan Putra,
50350 Kuala Lumpur
MALAYSIA

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INTRODUCTION

Welcome to the Malaysian Construction Research Journal (MCRJ) special issue in conjunction with the Engineering Technology International Conference 2020 (ETIC 2020). This conference has been successfully organized by the Faculty of Engineering Technology, Universiti Tun Hussein Onn from the Malaysia Technical University Network (MTUN).

This Special Issues of MCRJ for the ETIC 2020 consists of 20 selected papers by the scientific conference committee and expert reviewers. The conference having the main theme as Spurring High Technology Development for Sustainable Future with eight (8) tracks: Transportation, Energy, Electrical & Electronics, Mechanical, Manufacturing & Process Technology, Civil, Building, Infrastructure & Environmental Management, Chemical, Biotechnology & Sustainable Materials, Pharmaceuticals and OSHA.

This special issue is a platform for researchers and engineers to share their knowledge and experiences in creating a sustainable and environmentally friendly world for the present and the future. The accepted papers covered Civil, Building, Infrastructure & Environmental Management track. Each paper presented and published in this conference had undergone a rigorous review process by leading experts who are knowledgeable in the pertinent subject area.

It is believed that this special issue is significant to the academics and professionals with research works related to the construction industry and sustainable construction. In addition, it also brings opportunity for all authors to share ideas as well as stimulating awareness of this important field.

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Editorial

Welcome from the Editors

Welcome to the Malaysian Construction Research Journal (MCRJ) special issue for the Engineering Technology International Conference 2020 (ETIC 2020). We want to express our sincere gratitude to our contributing authors, reviewers, organizers and readers.

This Special Issues of MCRJ for the ETIC2020 contains twenty (20) interesting papers covering the theme of 'Spurring High Technology Development for Sustainable Future' consist of eight tracks: Transportation, Energy, Electrical & Electronics, Mechanical, Manufacturing & Process Technology, Civil, Building, Infrastructure & Environmental Management, Chemical, Biotechnology & Sustainable Materials, Pharmaceuticals and OSHA. It is hoped that the readers would greatly benefit from the scientific content and quality of papers published in this issue:

Brief introduction of each article is given as hereunder:

Suraya Hani Adnan et al., analyzed the mechanical performance of lightweight brick at elevated temperature. The research objective is to determine the effectiveness of Expanded Polystyrene (EPS) as a substitute for sand in the concrete mix. Therefore 20%, 30%, 40% and 50% of EPS percentage were replaced by sand volume. To examine the brick performance, hardened brick density, compressive strength, water absorption and fire resistance test were conducted. The result shows that the brick sample with 50% EPS is considered lightweight brick with its density of 1435.71 kg/m^3. It concluded that EPS might cause a significant reduction to compressive strength, water absorption and fire resistance of brick.

Mohamad Hairi Osman et al., presented the performance of Expanded Polystyrene Beads (EPS) as a fine aggregate replacement and Palm Oil Fuel Ash (POFA) as cement replacement in reinforced concrete (RC) beam on the flexural behaviour of the beam. This study aims to produce a low density concrete using EPS and POFA as a concrete replacement material. 10%, 20% and 30% of EPS had been used to replace fine aggregate and 10%, 20% and 30% of POFA had been used to replace cement. The result shows the crack pattern on the reinforced concrete beam that consisting of 10% to 30% of EPS and POFA can be classified as flexural cracks and some crack tend to be shear crack due to the angle of crack close to 45 degrees. The conclusion can be made that the higher percentage of EPS and POFA decrease the concrete compressive strength and RC beam capacity.

Tuan Noor Hasanah Tuan Ismail et al., presented the efficiency of the geo-cellular mat (GCM) to evaluate the settlement behaviour via field performance. GCM is lighter, stronger, and stiffer, with a density of 125 ± 10 kg/m^3, compressive strength in the range of 2.8 to 4.5 MPa, and stiffness in the range of 100 to 150 MPa. A series of field test was conducted to simulate a particular cross-section of road embankment with a uniform static load for nearly a year to verify the effectiveness of GCM. This study's findings successfully prove the efficacy of the GCM block to diminish settlements over peat ground, compared with sand fill. The results show that the maximum absolute settlement and differential settlement were reduced with the GCM technology application.

Aslila Abd Kadir et al., explored the light transmission of light pipe using reflective materials that are easily available locally at a lower cost. A straight light-pipe using 74% reflective material for the internal surfaces was installed in a test room located in Batu Pahat. Indoor illuminance and concurrent outdoor illuminance were monitored at 5-minutes interval for a month. Results showed that clouds, solar altitudes and external illuminance were affecting the performance of the system. This average internal illuminance level fulfilled the MS 1525:2014 recommendation for application in Malaysian buildings. This indicates that the light pipe system has potential as a tool for introducing indoor daylight in Malaysia.

Tuan Noor Hasanah Tuan Ismail et al., analysed the strength performance of soil reinforced with coconut fibre (CF). This research focuses mainly on investigating the compaction parameters and bearing capacity of CF–laterite soil. Standard compaction test and California bearing ratio (CBR) test were performed on soil randomly included with three different CF proportions (0%, 0.5%, and 1.0%). Results indicate that the optimum moisture content increased and the maximum dry density decreased as the proportion of CF increased to 1.0%. The coconut fibre-reinforcement makes the stabilised soil ideal as a subbase material for road pavement applications. In addition to enhancing soil strength efficiency, CF reuse would also reduce agricultural waste disposal to landfills and prevent environmental problems.

Husam Mansour et al., outlined a conceptual model underpinned by the resource-based view theory and the CIMO logic. A review of the literature revealed that there is still a lack of research on the benefits of Industry 4.0 in the construction industry context, particularly concerning the impact on performance. Therefore, this study seeks to understand the impact of Industry 4.0 technologies as resources, and the mediating role of IT and operations capabilities considering the construction industry complex con-text through the integration challenges.

Tiffany Valarie Kerry et al., presented the significant factors that affecting safety management implementation at construction sites based on the perceptions of different construction stakeholders. The factors affecting safety performance that formed the questionnaire are commonly accepted physical safety management performance factors that were derived from previous studies involving accident rates and the satisfaction of construction stakeholders. The present study's findings revealed a high similarity of OSH risk perception within and among stakeholder groups. This paper offers an important contribution by providing significant information that benefits the construction stakeholders to further facilitate the benchmarking process and various initiatives that can improve the safety performance of construction industry as a whole.

Nurvin Ratha et al., explained the conceptual development and the laboratory testing of a prototype which would be a composite recyclable material that would meet the need for a strong, stiff, permeable, lightweight mat as alternative fill material. The combination of polyurethane and waste plastic bottles meet the criteria of such an alternative fill. The findings show that the Polyurethane-Plastic Bottle (PUPB) block with a yield strength of 130kPa has about 2.6 times higher compressive strength than the Geofoam also the well-known lightweight field material in the industry. This implies that the PUPB block is greener but and stiffer and more suitable to be used as an alternative lightweight fill to mitigate hazardous settlement of constructions built on soft soil.

Ahmed Mokhtar Albshir Budiea et al., explored the behaviour of corrosion in pipelines steel when placed in seabed sediment environment. Simulations on the sediment of different characteristics have been carried out to examine their effects on steel coupons buried in them. Consequently, the corrosion rate of the steel specimens were determined via weight loss technique. The results show that metal weight loss increases as the duration of the exposure to seabed sediment environment becomes longer. The corrosion order for types of sediment existing in seabed simulated environment is found as follows; fine sand < medium sand < clay < coarse sand. In conclusion, the indoor simulation of seabed sediment can be conducted and is found relatively reliable to explain some corrosion phenomena for a short period studies.

Nor Haslinda Abas et al., discussed work-life balance in the Malaysian construction industry, to determine the trends of work-life balance, analyzing the main factors that affect work-life balance and the best initiative that can be taken in practising work-life balance practices among workers. The result shows the employee only had a medium work-life balance and can either be dropped into worst or improved. It also shows the item 'employee feels satisfied when they can give their best performance to the company even they need to sacrifice their time' is the top factor under the career satisfaction factor. Work-life balance is a good practice in improving employees' work performance and makes employees feel more comfortable to focus at work. The factors affecting work-life balance can either bring a good or bad implication depending on how employees face the challenge and their working environment.

Alvin John Lim Meng Siang et al., analysed a new concept of lightweight fill material construction in embankment using arch structure. One of the techniques in improving marine clay-type of soil is to use expanded polystyrene (EPS) as lightweight filler materials in road embankments. This is one of the initiatives to improve soil bearing performance and reduce soil settlements. Due to the irregularities in the design and an increase in demand and cost for EPS, this study proposes a new design of using an arch structure concept in lightweight fill design. A small-scale model of the road embankment will simulate where settlement will be measured on the model using EPS as the filler material. This research will focus on the different models in the arrangement of fill materials and the pattern of failures from settlement analysis. It is found that the arch structure provides less settlement while maintaining using limited EPS materials.

Zalipah Jamellodin et al., identified the physical properties of blended cement concrete containing palm oil fuel ash and eggshell powder. This paper presents the results of an experiment incorporating palm oil fuel ash (POFA) and eggshell powder (ESP) to partially replace ordinary Portland cement (OPC) in concrete-making. The results were promising, whereby both water absorption and UPV had improved while the density maintained within the normal range. The advancement of this technology is envisioned to drive down construction cost due to the reuse of waste material and contribute towards sustainability efforts through carbon footprint reduction.

Shahrul Niza Mokhatar et al., determined the modified rubberized concrete as a flexible concrete barrier utilizing waste materials under impact loading. The waste CR (Crumb Rubber) and RHA (Rice Husk Ash) were used as a replacement for fine aggregate and cement which 6% and 10% of waste CR with 10% RHA replaced in the normal concrete mix. The

results reveal the increasing rubber content in the concrete mix causing high penetration depth. In this study, 6% of CR was considered a good result since the crater diameters closed to the nose hemispherical projectile diameter (less percentage). In terms of energy absorption, the result was directly proportional to penetration depth. Thus, rubberized concrete able to improve the function of concrete as a concrete barrier in reducing impact since it shows high energy absorption.

Wan Amizah Wan Jusoh et al., assessing the engineering properties of reinforced concrete containing a combination of steel and polypropylene fibres. In this research, the physical and engineering properties of fibre reinforced concrete (FRC) are investigated by using hooked end steel fibre (SF) and virgin fibrillated polypropylene fibre (PPF). The results indicated that the best combination of fibres was for concrete containing SF 75% + PPF 25%. The combination of SF and PPF fibres in concrete was able to enhance the engineering properties and controlling the growth of cracks in concrete. The results also indicated that concrete with both SF and PPF produced higher tensile and flexural strengths than the control by 77% and 170%, respectively.

Faisal Sheikh Khalid et al., studied the performance of partial coarse recycle concrete aggregate (CRCA) containing insulator synthetic wire waste as fiber materials can reduce the amount of waste dumped into landfills. A study was made using coarse recycle concrete aggregate and wire waste as materials in the concrete mixture. The strength increases at 15% CRCA and 1.5% wire waste for 9.8% compared to the normal specimen. Therefore, it can be concluded that the optimum percentage for CRCA is 15% replacement materials and addition of 1.5% wire waste as fiber.

Siti Norsalkini Mohd. et al., explored the characterization and classification of sedimentary rock mass, located in Iskandar Puteri, Johor, Malaysia by utilizing the combination of geophysical investigation and geomechanical assessment. Resistivity and seismic refraction method were employed for geophysical investigation while the geomechanical classification involves Q system and Rock Mass Rating (RMR). The geophysical methods were carried out on the rock outcrop to estimate the rock mass quality index. In conclusion, the classification of sedimentary rock mass based on the correlations between the geophysical result from seismic velocity and resistivity and geomechanical classification using Q-system and RMR can be used as useful tools in site investigation to enhance the subsurface information during site investigation.

Norhaliza Wahab et al., analyzed the potential of Palm Oil Fuel Ash (POFA) as a profitless industrial by-product to substitute cement as it possesses cementitious behaviour when subjecting in cement hydrating process. The partial replacement of cement dosage in four others mixing proportions were replaced with 5 %, 10 %, 15 %, and 20 % of POFA. At 28 curing days, C85P15 achieved the highest value in unconfined compressive strength (UCS) test which was 107.6 kPa. In comparison to C100 and C80P20, C85P15 had lower carbon content, higher calcium content and lower silica and alumina content. Therefore, the ability of POFA to enhance the peat strength and effectively reduce the cement usage in peat stabilization is proved.

Husam Mansour et al., developed a conceptual model to identify the impact of planning on construction companies' quality by referring to three mediator factors: the concept of

planning, the importance of planning, and the basics of efficient planning. Additionally, the model aims to quantitively measure the impact of planning on the quality of projects. A semi-structured questionnaire was distributed randomly to 159 Jordanian Engineers. The results indicate that there is a medium level of planning in Jordanian construction companies. The respondents and statistical analysis showed a significant impact of planning on the quality of construction projects. Finally, some recommendations were made to improve construction quality and planning.

Agus Sulaeman et al., obtained the effect of w/c ratio to pile capacity and original soil, the small scale model drum. Clay soil, bored pile model and procedures of the simulation were also prepared. The simulation of the pile loading test (PLT) tests was carried out in various w/c ratio in ambient condition on first liquidity index. The results reveal that there are changes of clay properties due to migration of water into a void or crack inside surrounding clay, mixing cement mortar with clay mineral and compaction from hammer drop. The higher water content of original clay the bigger adhesion, α value as clay soil is softer and the optimum w/c-ratio (w/c = 0.4) provides more pile capacity. Understandably, water plays an important role. Water from cement mortar migrates to surrounding clay at various degree of moisture content, which contribute to the change of surrounding clay properties.

Ahmed Mohammed Awad Mohammed et al., studied the physical characteristics of kaolin change upon treatment with different concentrations of granulated ground furnaces blast slag (GGBS); changes in particle size and compaction properties are some areas of interest. Furthermore, the study assesses how a specific quantity of GGBS may be used for enhancing kaolin strength when it is cured for up to 28 days. Seven days curing time for 20% and 25% GGBS-kaolin blend leads to strength enhancement that exceeds the subgrade layer requirement. Considering 25% GGBS-kaolin blend cured for 28 days, the maximum strength was determined to be 8.4 MPa. The inclusion of GGBS leads to changes in crystal phase and structure which can be observed using microstructure analysis.

MECHANICAL PERFORMANCE OF LIGHTWEIGHT BRICK AT ELEVATED TEMPERATURE

Suraya Hani Adnan[1], Adriana Farniza Pozi[1], Mohamad Hairi Osman[1], Mohamad Luthfi Ahmad Jeni[1], Nurain Izzati Mohd Yassin[2], Zalipah Jamellodin[2], Noor Azlina Abdul Hamid[2], Norhafizah Salleh[2] and Masni A. Majid[2]

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Abstract

This study presents the production of lightweight concrete brick containing Expanded Polystyrene (EPS). The objective of this study is to determine the effectiveness of EPS as a substitute for sand in the concrete mix. The replacement percentage of EPS is 20%, 30%, 40% and 50% from the sand volume. Hardened brick density, compressive strength, water absorption and fire resistance test were conducted in order to examine the brick performance. The dimension of the brick is 215mm x103mm x 65mm. The total of samples for different testing were 60 samples of bricks where 30 samples were undergoing for density and compressive strength test with curing age 7 and 28 days, 15 specimens for water absorption test and 15 specimens for fire resistance test required. All brick samples were subjected to air curing. From the results, it is found that brick sample with 50% EPS is considered as lightweight brick because its density is 1435.71 kg/m^3. The results indicated that increasing the volume of EPS causes significant reduction to compressive strength, water absorption and fire resistance of brick.

Keywords: *Compressive strength; fire resistance; Expanded Polystyrene; sand replacement; lightweight concrete brick.*

INTRODUCTION

A concrete brick commonly known as a small rectangular piece with the ingredients comprises of cement and aggregates usually sand which is formed in moulds and cured. Concrete brick has different types of shapes such as solid and hollow brick whilst the sizes are varied according to the standard. Density of the brick is one of the important properties that contributes to the total weight of the brickwork. According to ASTM C90-09 (2012), concrete brick could be classified into three classes which are normal weight (more than 2000 kg/m^3), medium weight (1680-2000 kg/m^3) and lightweight (less than 1680 kg/m^3). In addition, there are another properties of bricks that have to be considered, but the foremost imperative properties is strength, which the compressive strength of brick can be shift from 3 N/mm^2 to 40 N/mm^2 depends on raw materials utilized for the brick production and the manufacturing process.

Nowadays, construction industry is focuses on the production of lightweight building material. This is because by having lightweight building materials, the total dead load of a building could be reduced hence this will make a building more stable against the earthquake forces (Veiseh & Yousefi, 2013). Next, lightweight materials will reduce the cost of construction which includes the cost of production, transportation and handling process (Xu et al., 2012). Due to these, Expanded Polystyrene (EPS) has become one of the most preferable lightweight materials due to its characteristic which is extremely light (Sayadi et al., 2016). It is also well known as a stable low-density foam of non-absorbent, hydrophobic,

artificial ultra-lightweight and closed-cell nature aggregate (Ling & Teo, 2013). The properties of EPS in construction moreover critical which is including low thermal conductivity, mechanical resistance, low water retention which is it does not absorb moisture. In addition, its thermal and mechanical properties are unaffected by damp, humidity or moisture, ease of dealing with and installing, chemical resistance, flexible, ageing resistance and secure in construction works.

EPS has been utilized in construction industry as a replacement material for aggregates in the concrete mixture. Other than its ability in reducing the density of building material, the replacement of EPS also could minimize the usage of natural resources such as aggregates or river sand.

Besides density and strength, fire resistance performance of a building material is also a vital parameter to ensure the stability and serviceability of building after been exposed to high temperature. Therefore, this study aims to investigate the properties and the optimum replacement of Expanded Polystyrene (EPS) in the brick and also its performance after been exposed to elevated temperature.

MATERIALS AND METHODS

Materials Preparation

In this study, the lightweight concrete bricks are made up of Ordi-nary Portland Cement (OPC), fine aggregates (river sand), expanded polystyrene (EPS), tap water and superplasticizer.

Ordinary Portland Cement (OPC)

OPC used in this study is classified in type 1 based on the ASTM C150 (2012). The supplier for this cement is Tasek Corporation Berhad which certified by SIRIM. Cement has been stored in an air tight container to prevent any moisture contact. The chemical composition of OPC has shown in Table 1.

Table 1. Chemical Composition of OPC

Chemical Composition	(%)
Silicon Dioxide (SiO_2)	14.6
Aluminium Oxide (Al_2O_3)	3.95
Ferric Oxide (Fe_2O_3)	3.46
Calcium Oxide (CaO)	57.1
Potassium Oxide (K_2O)	0.51
Magnesium Oxide (MgO)	1.62
Sodium Oxide (Na_2O)	-
Sulfur Trioxide (SO_3)	3.43

Fine Aggregates (Sand)

In this study, river sand has been used as fine aggregates. River sand was sieved by sieve plate to ensure that it does not consist any impurities and in order to have a consistent size.

Earlier than that, a process of oven dried for a minimum of 24 hours at the temperature of 105±5 ^{o}C was conducted. In this research, the sieve analysis of sand was as accordance to ASTM C 136-06 (2012).

Expanded Polystyrene (EPS)

The size of EPS used in this study was between 1.18mm to 2.36mm. EPS used was collected from ST Polyfoam Industries Sdn. Bhd which located in Batu Pahat, Johor. Figure 1 shows the shape of EPS used in this study.



Figure 1. Expanded Polystyrene (EPS)

Brick Sample Preparation

The mix ratio of the brick is 1:3 (cement: sand). For this study, there are 5 different types of bricks which varied in percentage of EPS replacement. The water cement ratio is 0.5 and super plasticizer is 8 ml for every 1 kg binder. The size of the brick is 215 mm x 103 mm x 65 mm. Table 2 shows the mix proportion of concrete mix for each brick. The total specimens needed for density and compressive strength test in this study were 30 specimens, 15 specimens of bricks used for water absorption test. Meanwhile, 15 specimens of bricks were conducted for fire resistance test.

Table 2. Mix Proportion of Bricks

Mixture	Mix Proportion				EPS Percentage (%)
	Cement (kg)	Sand (kg)	EPS (g)	w/c ratio (kg)	
1	0.71	2.14	0	0.36	0%
2	0.71	1.72	5.74	0.36	20%
3	0.71	1.50	8.61	0.36	30%
4	0.71	1.29	11.48	0.36	40%
5	0.71	1.07	14.35	0.36	50%

Mixing and Casting Process of Brick

For the mixing process, cement and sand were mixed before water and super plasticizer were added. After 5 minutes of mixing, the EPS was added and thoroughly mixed in the concrete mixture for 3 minutes. Then, the concrete mix was casted into the timber moulds and was left inside the laboratory for 24 hours. After, 24 hours all the brick samples were demoulded and again all the brick samples were left inside the laboratory for curing process until the day of test.

RESULT AND DISCUSSIONS

Density Test

The density of brick is calculated as mass per unit volume. According to ASTM C90-09 (2012), there are three classes of concrete masonry unit which is normal weight (more than 2000 kg/m³), medium weight (1680-2000 kg/m³) and lightweight (less than 1680 kg/m³). Based on the bar graph in Figure 2, it shows that the density of samples decreases with EPS added in their production because EPS has lower density than sand. This lower density indicted that the particle in the brick are not closely packed due to inability of EPS to bind with other substance because of its smooth surface.

Control sample with contained 0% of EPS recorded the highest density over another samples which is 2150 kg/m^3 for 7 days and 2050 kg/m^3 for 28 days. The dry density of E20P0 and E50P0 decrease for 7 days which is 13.3% and 32.2% respectively. However, the decrease in dry density of E20P0 and E50P0 for 28 days are 9.8% and 30% respectively where they are influenced by adding EPS it would be possible to produce lighter weighting cement sand brick. The sample with 50% of EPS contained recorded the lowest density which is 1457.14 kg/m^3 for 7 days and 1435.71 kg/m^3 for 28 days.

In compliance with ASTM C90-09 (2012), E0P0 for 7 days and E0P0 for 28 days of testing day are considered as normal weight brick as there are all above 2000 kg/m³. Furthermore, for 7 days of curing period, E20P0, E30P0, E40P0 and for 28 days of curing period, E20P0, E30P0 are considered as medium weight brick classification as the density is within 1680-2000 kg/m³. However, another mix ratio is considered as lightweight brick classification as there are is below 1680 kg/m³ which is E50P0 for 7 days and E40P0, E50P0 for 28 days.

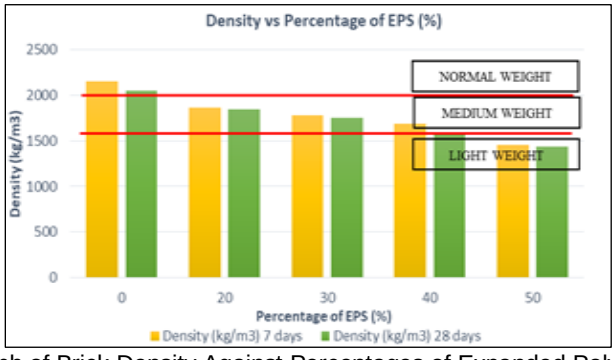


Figure 2. Graph of Brick Density Against Percentages of Expanded Polystyrene (EPS) at The Age of 7 and 28 Days

Compressive Strength Test

Compressive strength test on bricks was carried out to determine the load carrying capacity of bricks under compression. This test was conduct in accordance to ASTM C129-11 (2012). Based on the bar graph in Figure 3, it shows that the overall graph of compressive strength values of brick decreases uniformly with increasing replacement percentage of EPS. The compressive strength value for the sample with 20% of EPS drop which is about 17.14%

compare to control sample at 7 days and 14.96% at 28 days. While, for the sample with 50% of EPS also decrease which is about 45.52% compare to 30% of EPS at 7 days and 34.15% at 28 days. The rate of strength was greater initially and decreased as the age increased.

In conclusion, with increasing percentage volume of EPS directly affects the compressive strength and density of brick. The EPS contain in bricks causes to lightweight and have low strength. This is possibly due to close to zero strength of EPS particles and high compressibility behaviour which causes formation of micro cracks at the transition zone of particles and cement paste (Sayadi et al., 2016 & Adnan et al., 2019). The compressive strength of concrete increased when the age of concrete also increased. Moreover, the compressive strength of brick decreased as more sand were replaced by EPS. From the bar graph, for 7 and 28 days of air curing, the brick sample with 50% of EPS (E50P0) was recorded the lowest strength among the other samples which is 7.9 and 13.5 MPa respectively. While, the highest compressive strength for 7 and 28 days of air curing, was recorded for control sample (E0P0) that not added with the EPS as sand replacement which is 21.0 and 27.4 MPa respectively. In this study, all mixture specimens have fulfilled the minimum strength in non-load bearing classification (4.14 MPa) which is stated in ASTM C129-11, Non-Load Bearing Concrete Masonry Unit (2012).

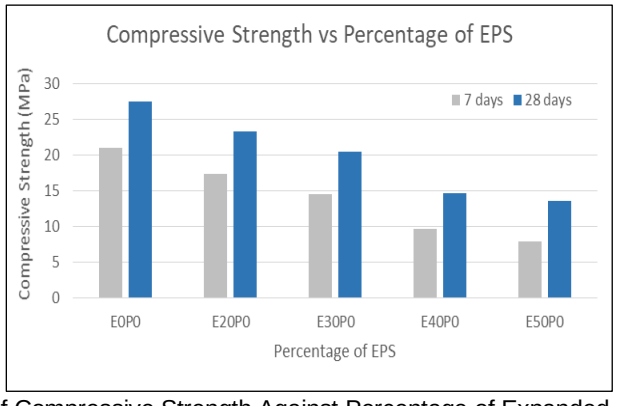


Figure 3. Graph of Compressive Strength Against Percentage of Expanded Polystyrene (EPS) at The Age of 7 and 28 Days

Water Absorption Test

Water absorption test was done based on the procedure given in BS1881 Part 112 (1983). The percentage of water absorption of concrete cube was obtained from each sample after immersed in water for 30 minutes and dried in the oven for 72 hours. The sample with 0% of EPS recorded the highest average percentage of water absorption compare to the sample with 50% of EPS.

Figure 4 depicts that the sample with more percentage of containing EPS has lower absorption ability. This indicates that the addition of EPS in the mixes have a significant effect on the water absorption. The water from outside was not absorbed in concrete due to the inability of the EPS to absorb water. The sample with 0% EPS has the highest percentage of water absorption which is 11.23%. Meanwhile, for the sample with 50% of EPS contain lowest water absorption which is 4.37%. In general, the percentage of water absorption decreases with the increasing percentage of EPS and age of sample.

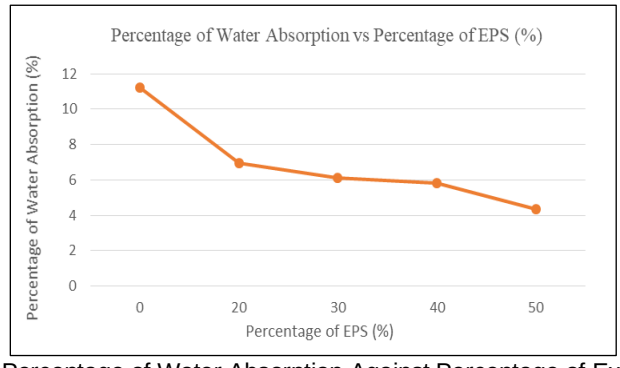


Figure 4. Graph of Percentage of Water Absorption Against Percentage of Expanded Polystyrene (EPS) at Age of 28 Days

Fire Resistance Test

Concrete as a material has a good ability to withstand fire. The high temperature test is designed to determine the ability and duration of all the bricks produced in this investigation to maintain compressive strength when exposed to high temperatures (Khalaf & DeVenny, 2004). In this study, the bricks were tested for compressive strength after exposed to fire. Fifteen samples of concrete sand bricks were taken from each mix for strength tests at different temperatures. One sample of brick for each temperature were required from each different mix to test for compressive strength as well as five bricks that were crushed at room temperature to act as a control. ASTM E119-00a (2012) is used to decide the fire resistance of a complete assembly. The building code specialists ordinarily assign the duration of fire resistance required in a building.

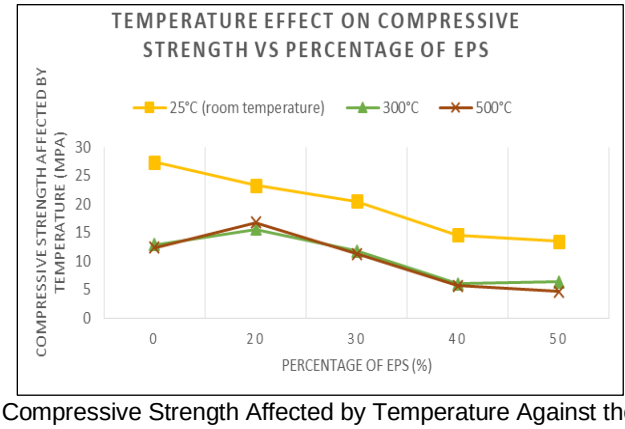


Figure 5. Graph of Compressive Strength Affected by Temperature Against the Percentage of EPS

Based on the graph in Figure 5, it shows that the overall graph of compressive strength values affected by temperature at 25°C, 300°C and 500°C. The strength of bricks was peak at 20% of EPS contain and thorough at 40% at temperature 300°C and 500°C. For control sample of brick contain 0% of EPS (E0P0), the strength of brick was 27.4 MPa at room temperature, and seen it was decrease due to increasing temperature which is 12.9 MPa at 300°C and 12.4 MPa at 500°C. The compressive strength of brick for the proportion of 20% of EPS was increased up to 15.6 MPa and 16.8 MPa at 300°C and 500°C respectively, as the strength for control sample were 12.9 MPa and 12.4 MPa at both temperatures respectively.

The strength of samples was decrease uniformly due to increasing the percentage of EPS at room temperature. Furthermore, the increase of the temperature up to 500°C, the compressive strength decreases.

All the mixture specimens after exposed to the fire have fulfilled the minimum strength in non-load bearing classification (4.14 MPa) as stated in ASTM C129-11, Non-Load Bearing Concrete Masonry Unit (2012). Although the compressive strength of samples decreases within the increasing of temperature, the value is still acceptable as it above than 4.14 MPa according to standard.

CONCLUSIONS

The following conclusions were discussed from the study:

1. The density of the brick diminishes with expanding in EPS content. The most extreme density at 28 days was seen as 2050 kg/m³ for E0P0 sample and least density at 28 days was seen as 1435.71 kg/m³ for block with E50P0 content. E50P0 it is all the more daintily about 29.97% contrast with E0P0 at 28 days curing.
2. The compressive strength of the brick diminishes with expanding in EPS content. The most extreme compressive strength at 28 days was seen as 27.4MPa for E0P0 sample and least compressive strength at 28 days was seen as 13.5MPa for brick with E50P0 content.
3. The optimum sample, the percentage utilized was 50% EPS substitution to sand (E50P0) is order as lightweight brick classification, as indicated by ASTMC90-09.
4. Substitution of EPS to sand individually produces an optimum mix proportion in lightweight bricks, which have turned into an important trend for green buildings. This is on the grounds that lightweight bricks diminish the building weight. The quantities of internal pores in lightweight bricks are much prominent than in typical concrete sand bricks. From the discoveries of this study, it has given a potential solution for the construction industry. For generally speaking, the expansion of EPS in brick could prompt great execution of brick that reasonable for non-load bearing construction activity.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the Universiti Tun Hussein Onn Malaysia and Ministry of Higher Education for providing the financial support through the Fundamental Research Grant Scheme (FRGS), Vot 1618 and Postgraduate Research Grant (GPPS), Vot U77.

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THE PERFORMANCE OF REINFORCED CONCRETE BEAM CONTAINING EXPANDED POLYSTYRENE BEADS (EPS) AND PALM OIL FUEL ASH (POFA) IN TERM OF FLEXURAL PERFORMANCE

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Abstract

This paper focuses on study related with the usage of Expanded Polystyrene Beads (EPS) as fine aggregate replacement and Palm Oil Fuel Ash (POFA) as cement replacement in reinforced concrete (RC) beam. The reinforced concrete beam with dimension of 1200mm x 200mm x 150 mm had been prepared and all beams were simply supported on two rolling bearing spaced 1000 mm centre to centre. One-point loading was act at the top mid-span face of reinforced concrete beam. The percentage of substitution of Expanded Polystyrene Beads (EPS) and Palm Oil Fuel Ash (POFA) were 10%, 20% and 30%. The concrete was design to achieved the target strength 25 MPa at the age of 28 days and while, the reinforced concrete beams were designed to bear 30 kN point load at beam mid-span. Over this study, the result pattern showed that the higher percentage of EPS and POFA used reduce the concrete compressive strength and also reduce the capacity of RC beam. While, 10% to 20% of POFA in this study seem to improve the compressive strength of concrete containing EPS with result close to the normal concrete strength with lower density. The flexural performance and load carrying capacity of reinforced concrete beam containing EPS and POFA showed decrease trend. Even though the result of actual point load capacity showed the decrease trend but the maximum point load for 20% EPS and POFA still greater than the design load 30 kN and may apply as structure member. The crack pattern on the reinforced concrete beam that consist of 10% to 30% of EPS and POFA can be classified as flexural cracks and some crack tend to be shear crack due to the angle of crack close to 45 degrees.

Keywords: *Palm oil fly ash; expanded polystyrene; fine aggregate; compressive strength; flexural performance.*

INTRODUCTION

Palm oil industry plays an important role in the economic development several Asian countries. Although current market prices have dropped for palm oil production but it is still continued and expected to recover soon. In general, the problem of industrial waste such as polystyrene in Malaysia is increasing critically which has caused many environmental pollution problems. Besides that, an estimated 24% of plastic waste are disposed of landfills in Malaysia (Noor Hasanah et al., 2014). Furthermore, a lot of waste were produce from the process of oil palm, the waste should be well managed and more productive when it can be re-used for other purpose.

This study was conducted to produce a low density concrete using polystyrene beads (EPS) and palm oil fuel ash (POFA) as a replacement material into concrete. As is known, palm oil waste is a highly readily available industrial waste and is selected to replace cement because of its delicate nature and can also react as binder. In addition, 10%, 20% and 30% for

the use of expanded polystyrene beads (EPS) and 10%, 20% and 30% for the use of palm oil fuel ash (POFA). The goal of the study is to investigate the flexural performance of reinforced concrete beam containing expanded polystyrene beads (EPS) and palm oil fuel ash (POFA) as a replacement material. The compressive strength of concrete containing of expanded polystyrene bead (EPS) and palm oil fuel ash (POFA) also had been compared with normal concrete to see the optimum percentage of replacement. The reinforced concrete beam had been design based on Eurocode 2 to bear point load 30 kN.

This study focuses on replacement of the expanded polystyrene beads (EPS) as a fine aggregate and the palm oil fuel ash (POFA) as a cement in term of flexural behavior of the beam. The materials used for this study were cement, fine aggregate, coarse aggregate, expanded polystyrene beads (EPS), palm oil fuel ash (POFA) and water. 10%, 20% and 30% of expanded polystyrene beads (EPS) had been used to replace fine aggregate and 10%, 20% and 30% of palm oil fuel ash (POFA) had been used to replace cement. The compressive test and point load test for beam had been carry out.

UTILIZATION OF POFA AND EPS IN CONCRETE

EPS was taken to substitute fine aggregate due to its plastic properties that can potentially make concrete lighter. Meanwhile, POFA was selected to replace cement because of the high silica contain that make it suitable as a binder. Numerous researches have been conducted on the usage of POFA and EPS in concrete manufacture. The use of polystyrene aggregate has a number of advantages including the decrease of natural fine aggregate use and also may decrease the density of concrete. The Mechanical characteristic such as lightness, rigidity, and formability, shows that the EPS can be used in a variety of applications. Besides, EPS concrete has the possibility to be applied as structural members such as exterior panels, curtain walls, load bearing wall, composite flooring systems, shielding layers, and isolated concrete (Coppola et al., 2016). POFA is the one of agriculture waste ashes which is chemical composition contains a huge amount of silica. Thus, it can theoretically be used as a cement replacement or as a filler to produce high strength and durable concrete (Munir et al., 2015).

Expanded Polystyrene Beads (EPS)

EPS is one of the materials used as packaging materials or insulation materials. EPS have a very low density. Expanded Polystyrene Beads are round and contain only about 2% polystyrene and about 98% air. When expands occurs, the beads will not absorb water and therefore can be used as a lightweight aggregate and produce lightweight concrete. Expanded polystyrene is a stable low density foam and consists of discrete air voids in a polymer matrix. The polystyrene beads can be easily incorporated in mortar or concrete to produce lightweight concrete with a wide range of density (Sri Ravindrarajah & Tuck, 1994). In the previous research, the use of polystyrene beads in concrete produced segregation due to extreme lightness and hydrophobic nature of the untreated beads (Sri Ravindrarajah & Tuck, 1994).

Palm Oil Fuel Ash (POFA)

Numerous studies have stated that oil palm ash has good pozzolanic properties and can be used as an additive in cement (Munir et al., 2015). Therefore, recycling palm oil waste helps to mitigate environmental pollution problems and preserve the construction industry.

POFA are usually gray and darker with increased levels of non-combustible carbon. Although the particles have different sizes, they are generally taken into account and specific gravity less than the usual Portland cement (Thomas et al., 2017). Compared to the size of Portland cement particle size and the size of the palm oil ash particle size is smaller and the texture is porous (Sata et al., 2004).

Reinforced Concrete Beam

Structural members made of concrete are usually reinforced by steel bars. Steel reinforcement is used to resist tension, to distribute cracks and to limit cracks' width. But the first aim of reinforcement is to protect against a brittle failure. Previous studies on reinforced concrete beam using POFA and EPS as combination materials were highly limited. Normally, previous study on usage of POFA and EPS in reinforced concrete beam is separated. In certain project research POFA had been used in reinforced concrete beam to produce a new retrofitting material called Green-USM-Reinforced Concrete GUSMRC and the result showed the load carrying capacity increased up to 37% over to control beams, also significantly improve the serviceability of the damaged beam (Aldahdooh et al., 2016). Other project research regarding EPS is lightweight expanded polystyrene concrete reinforced with steel fiber. In this research showed that EPS produce low density reinforced concrete and low strength material with good energy-absorbing characteristics. However, due to lightweight of EPS beads and their hydrophobic surface, EPS concrete is prone to segregation during casting, which results in poor workability and lower strength. The combination with silica fume greatly improved the bond between the EPS beads and cement paste and increased the compressive strength of EPS concrete. In addition, adding steel fiber significantly improved the draying shrinkage (Chen & Liu, 2004).

EXPERIMENTAL DETAILS

The materials used in this study were ordinary Portland cement compatible to BS12: 1991, sand with fineness modulus of 2.85, crushed granite with maximum size of 20 mm, Palm Oil Fuel Ash (POFA), and Expanded Polystyrene Beads (EPS). The size of EPS used mostly 3.0 mm size beads. In this study the reinforced concrete beam with different proportion of EPS and POFA as replacement material had been produced to perform flexural test with size of 1200 mm x 200 mm x 150 mm. The concrete cylinder with 150 m diameter size with different proportion of EPS and POFA as replacement material had been produced for studying the compressive strength at 7 days and 28 days. The 6 numbers of concrete beams were cast with used the percentage of EPS and POFA were 10%, 20% and 30 % respectively.

Concrete Mix Proportion and Mixing

In the present study tests were carried out on normal strength concrete of design compressive strength equal to 25 MPa. The DOE method was used as guideline to design the concrete mix as per showed in Table 1 and mixing was made manually till homogeneous mix was obtained. Beside the control mix, three percentages of POFA and EPS waste of 10%, 20% and 15 % as cement and sand replacement respectively were used for producing different concrete mixes. POFA were replaced the cement content based on weight meanwhile EPS were replaced the sand content based on volume due to the deferent density of EPS and sand too high.

Table 1. Concrete Mix Design Grade 25 (Control) (Normal Concrete Design Mixes)

Water Cement Ratio	0.5
Cement	320 kg/m^3
Water	160 kg/m^3
Fine aggregate	405 kg/m^3
Coarse aggregate	1440 kg/m^3

Beam Specimens

Twelve reinforced concrete beams casted are divided to four groups. Each group has identical the content of EPS and POFA as replacement material in concrete. Details of dimensions, flexural and shear reinforcements are shown in Figure 1. All beams were designed to fail in flexure. For the first group of beams were acted as control specimens with 0% EPS and POFA as replacement in concrete. For second group of beams were contained 10% EPS as sand replacement and 10% POFA as cement replacement. For third group of beams were contained 20% EPS as sand replacement and 20% POFA as cement replacement. For the fourth group of beams were contained 30% EPS as sand replacement and 30% POFA as cement replacement.

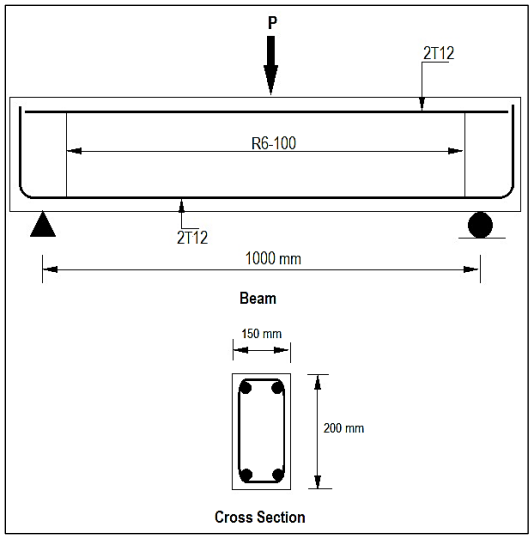


Figure 1. Dimensions and Reinforcements Details of Beams

Testing And Measurement

After 28 days curing, all beam specimens were ready to test for flexural under three-point bending test. Cylinder specimens were weighed and all beams were white painted to show cracks formation during testing. Measurements were made for concrete density and compressive strength and average values of three specimens were taken. Each beam was tested on simple span under one central point loads of arrangement shown in Figure 2. For testing concrete cylinders and beams a universal testing machine was used. Cylinder specimens were tested for compressive strength according to ASTM C39 specification (ASTM-C39, 2018). View of one beam setup and ready for testing is shown in Figure2. Central deflection was measured using linear variable differential transducers (LVDT). After testing photographs were taken to show the cracking pattern for the failed beam.



Figure 2. Testing Machine Used for Beams and Concrete Specimens



Figure 3. Compressive Strength Test

RESULT AND DISCUSSION

Workability

The slump value was taken for 4 batches of concrete mix with different percentage of EPS and POFA. The slump test conducted according to BS 1881 – Part 102. The high workability concrete provided easier handling, compaction and finishing. Table 2 below shows the results of slump test for 4 specimens with difference batches.

Table 2. Slump value

Specimens	EPS (%)	POFA (%)	Slump Value (mm)
1	0	0	50
2	10	10	48
3	20	20	42
4	30	30	38

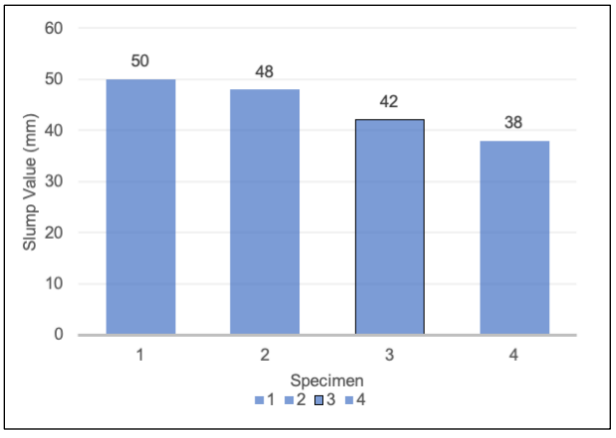


Figure 4. Slump Value

The slump value shows decrease trend when the percentages of EPS and POFA increase. Slump value recorded for normal concrete was 50mm. While, the slump value for 10% EPS and 10% POFA was 48mm. For 20% EPS and 20% POFA, the value of slump was 42mm. Lastly, the value of slump for 30% EPS and 30% POFA decreased 38mm respectively. The workability of the concrete decreased when the percentages of EPS and POFA increase due to the characteristic of POFA which is high on water absorption. But the values of slump test above still can be accepted and fulfil requirement of slump between 30mm and 60mm according to concrete mix design. There was no segregation while preparing and compacting the concrete. Tay and Show (1995) also noticed that there is a reduction in the workability of concrete with the increase in the amount of POFA. Then, according Awal and Shehu (2015) have observed that the slump decreased with the increasing amount of POFA in concrete.

Concrete Compressive Strength

Result of concrete compressive strength and load capacity of tested beams are shown in Table 3. Result indicated that the compressive strength of concrete decrease as the percentage of POFA and EPS increases. Average strength reduction is 5% when there is 10% EPS and 10% POFA in concrete. In general, the reduction takes place in the strength due to the non-homogeneity of concrete because of the existence of EPS in concrete. The added of POFA in concrete containing EPS also seems to be not helpful to increase the concrete strength. But the result of concrete containing 10% to 20% EPS and 10% to 20% POFA shows the close compressive strength with normal concrete. The same result was found by Tay (1990), the optimum level of 10% replacement with POFA was suggested, so that it may not affect the long term strength formation of concrete. Tangchirapat et al (2009) noticed the compressive strength of concrete containing 10% to 30% POFA was higher than control concrete at 28 days, 90 days and 180 days, while higher strength was noticed in the concrete containing 20% POFA. In the case of this study the existence of EPS in concrete cause the concrete strength drops but concrete strength containing 10% to 20% EPS and 10% to 20% POFA were not decrease too much and acceptable. In addition, EPS beads are hydrophobic, which results in weak bonds with the cement paste. According to Chen and Liu (2004), in term of concrete density, the result shows the decrease as the percentage of POFA and EPS increase. The failure mode of concrete containing EPS under compression more gradual and the specimens were capable of retaining the load after failure, without full collapse. Therefore, concrete

containing EPS may be considered for absorbed vibrations. A similar type of failure was also reported by Babu (2003) for EPS concrete.

Table 3. Cube Compressive Strength

Cube Specimens	EPS (%)	POFA (%)	Compressive Strength (MPa)		Concrete Density Kg/m^3
			7 days	28 days	
1	0	0	18.5	27	2460
2	10	10	17.5	25.5	2374
3	20	20	12.3	22.5	2273
4	30	30	9.83	14.57	1830

Maximum Load of Beams

Maximum load of beams result is shown in Table 4. Beam capacity shows decrease trend as the percentage of POFA and EPS increases. The reduction in the load capacity are 37%, 51%, 67% for 10%, 20 % and 30% for EPS and POFA content respectively. The reduction in load capacity up to 20% EPS and POFA not too high and still higher then beam design load capacity 30 kN. The reduction in load capacity may be due to the high porous structure of concrete with POFA that lead to the stress concentration and weakening aggregate and cement. The characteristic of EPS as mention before also shows not helping the bonding between aggregate and cement. Ranjbar el al. (2016) also observed a reduction in the flexural strength of self-compacting concrete containing POFA.

Table 4. Maximum Displacement and Maximum Load Capacity of Beams

Beam Specimens	EPS (%)	POFA (%)	Max Displacement (mm)	Max Load (kN)
1	0	0	12.83	71
2	10	10	11.75	44.5
3	20	20	3.57	34.5
4	30	30	2.91	23

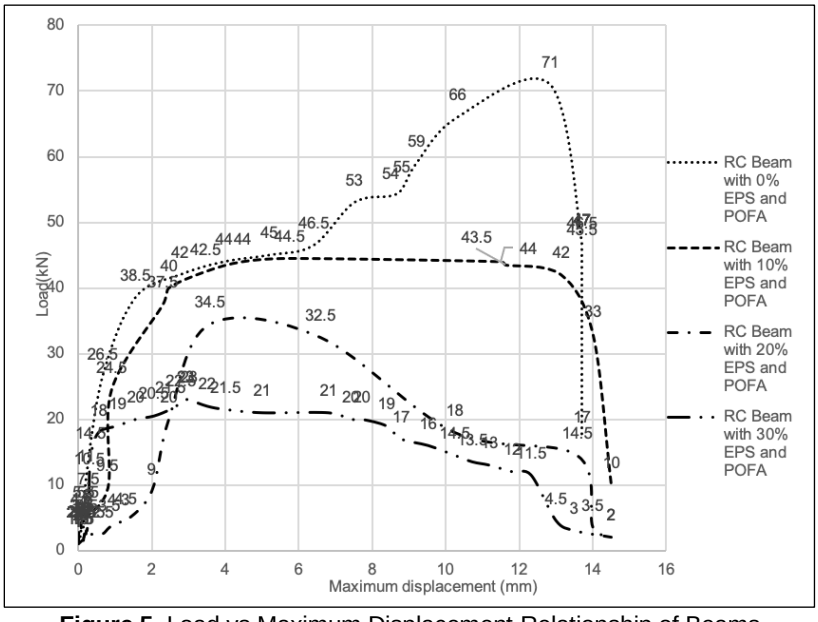


Figure 5. Load vs Maximum Displacement Relationship of Beams

Figure 5 shows the result of load vs maximum displacement curve and the stiffness of four specimens initially almost the same pattern. For control specimen 1 the beam shows failed suddenly and for specimen 2, 3 and 4 the beam shows failed slowly, it is may be due to the effect of EPS cause the specimens capable to bear the load after failure without full breakdown.

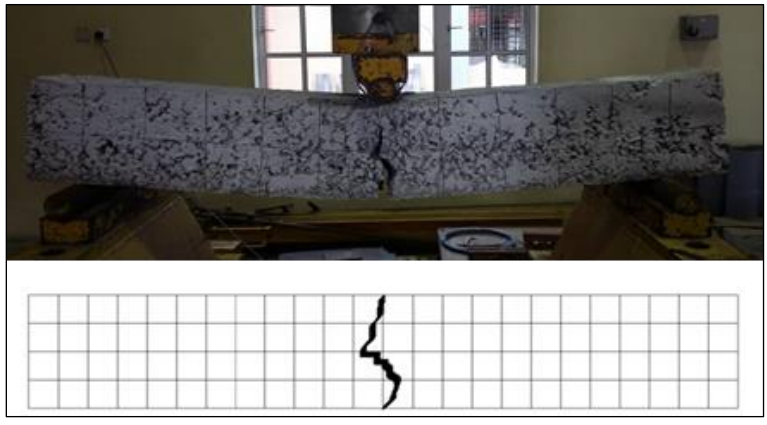


Figure 6. Crack Pattern of Reinforce Concrete Beam Without EPS and POFA (Control)

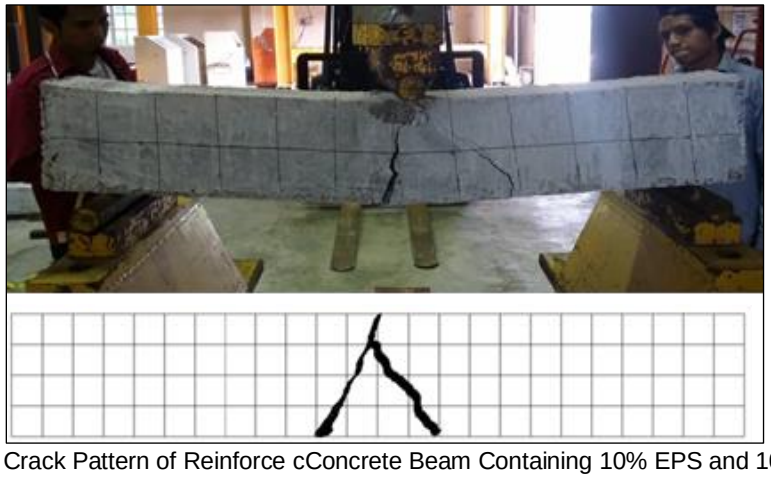


Figure 7. Crack Pattern of Reinforce cConcrete Beam Containing 10% EPS and 10% POFA

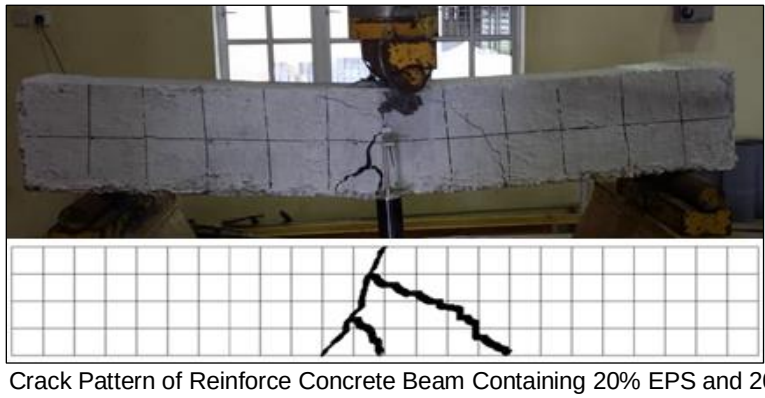


Figure 8. Crack Pattern of Reinforce Concrete Beam Containing 20% EPS and 20% POFA

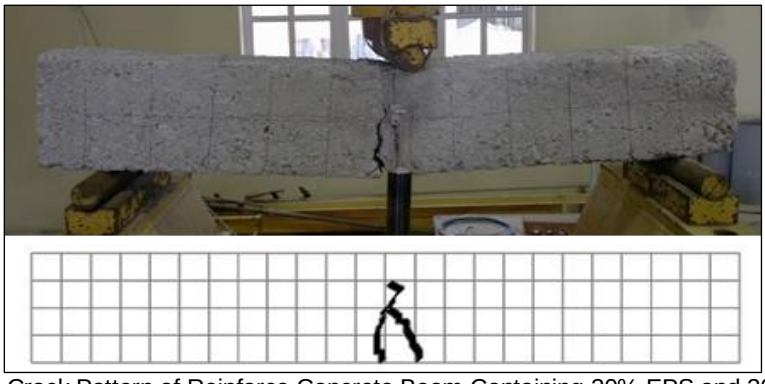


Figure 9. Crack Pattern of Reinforce Concrete Beam Containing 30% EPS and 30% POFA

Figure 7 to Figure 9 shows the crack pattern on the reinforced concrete beam that consist of 10% to 30% of EPS and POFA and generally all the crack can be classified as flexural cracks and some crack tend to be shear crack due to the angle of crack close to 45 degrees. Figure 6 shows the crack pattern for control beam and the crack pattern totally classify as flexural crack. Flexural cracks normally occur due to maximum load applied on the beam at the center point of the beam. The flexural crack is a type of crack that occurs at the middle of the beam. Besides that, the flexural cracks occur when the bending process started because of the flexural capacity of the beam is inadequate. The EPS and POFA in reinforced concrete beam seem not give the big influent to the crack pattern. Cracking pattern behaves close similar to that of normal reinforced concrete beam, with the deflection at maximum load seem lower than normal reinforced concrete beam. But in general the result Figure 5 shows that when the percentage of EPS increased the displacement of beam also increased. Herki and Khatib (2017) stated that when the replacement levels of EPS increased, the deflection also increased regardless of the reinforcement arrangement and the mode of failure was largely a shear failure.

CONCLUSION

In this research work the performance of reinforced concrete beams containing EPS and POFA were studied by laboratory test. The conclusion can be made that the higher percentage of EPS and POFA used decrease the concrete compressive strength and the capacity of RC beam. POFA in this study seem to improve the bond between EPS beads and cement also seem to improve the compressive strength of concrete close to the normal concrete strength with lower density. The optimum percentages for cement and fine aggregate replacement for concrete compressive strength were determined as 10% to 20% of POFA and 10% to 20% of EPS, which was showed the close compressive strength with normal concrete and fulfilled the requirements strength of structural concrete. The flexural performance and load carrying capacity of reinforced concrete containing EPS and POFA showed decrease trend. Even though the result of flexural strength showed the decrease trend but the maximum point load for 20% EPS and POFA still greater than the design load 30 kN and perhaps may apply as building structure members. The crack pattern on the reinforced concrete beam that consist of 10% to 30% of EPS and POFA can be classified as flexural cracks and some crack tend to be shear crack due to the angle of crack close to 45 degrees.

ACKNOWLEDGEMENT

The author would like to give truthful recognition to Office for Research Management Centre (RMC), Universiti Tun Hussein Onn Malaysia for the commitments especially on giving the budget for fee through TIER 1 research grant (Grant code H177).

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EFFICACY OF GEO-CELLULAR MAT (GCM) TO MITIGATE SETTLEMENT VIA FIELD PERFORMANCE

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Abstract
Road construction over soft yielding ground like peat soil presents a considerable technical challenge. Therefore, the soil needs to be stabilised to improve its geotechnical properties and to ensure the embankments constructed on it are stable and strong without risking excessive settlement and bearing failure. Recently, more attention has been given to lightweight fill materials as an alternative technology for road construction over soft ground. Hence, the development and use of innovative green technology products to meet the current pressing geo-environmental challenges are critically examined in this study. The main purpose of this paper is to evaluate the settlement behaviour of geo-cellular mat (GCM) block and compared the response from conventional natural fill material (sand fill). This is followed by the conceptual development of a GCM block by recycling plastic. GCM is lighter, stronger, and stiffer, which has a density of 125 ± 10 kg/m^3, compressive strength in the range of 2.8 to 4.5 MPa, and stiffness in the range of 100 to 150 MPa were measured via axial compression test. A series of field test was conducted to simulate a particular cross-section of road embankment with a uniform static load for nearly a year to verify the effectiveness of GCM. The settlement plate gauges were used to determine the total ground movement beneath the fill loading at the interface between fill materials and subgrade surface. The findings of this study successfully prove the efficacy of the GCM block to diminish settlements over peat ground, compared with sand fill. The results show that the maximum absolute settlement and differential settlement were reduced by 35% and 70%, respectively with the application of GCM technology.

Keywords: *Soft yielding ground, settlement; lightweight fill material; geo-cellular mat; recycled plastic.*

INTRODUCTION

The characteristic of existing soil at the construction site is not always totally suitable for supporting structures. Settlement of highway embankments over soft soils such as silty, clayey, or excessive organic soils (peat) is a prime problem encountered in maintaining structure facilities (Makusa, 2012). Such soils with high moisture content, low strength, and high compressibility tend to lack shear strength, bearing capacity, and long-term creep, causing many post-construction problems. Therefore, ground improvement plays an important role in geotechnical engineering because it is the only way to stabilise and modify the properties of soil, especially for constructions over soft ground (Huat, 2010). Historically, numerous ground improvement technologies have been suggested and adopted, encompassing shallow, medium, and deep soil treatment and involving preloading, sand or stone column, pre-fabricated vertical drains, piles, fibre-reinforced materials, chemical stabilisation (fly ash, cement, and lime), and additive materials (shredded tires, sawdust, wood chip, clams shells, expanded clay, etc.) (Gaafer et al., 2015; Bryson & Naggar, 2013). However, some of these methods are constrained by technical feasibility, expensive construction cost, and space and time constraints.

Innovative use of lightweight fill material can meet the geotechnical challenges posed by soft yielding ground by reducing settlement (Marradi et al., 2012; Ismail et al., 2019). The stress on the subsoil can be reduced so that the settlement is reduced or eliminated if the road embankment is constructed out of fill materials lighter than soil. In this respect, various types of lightweight materials such as sawdust, fly ash, slag, cinders, cellular concrete, lightweight aggregates, expanded polystyrene (EPS), shredded tires, and seashells have been proposed for road embankment construction. However, Ismail (2017) claimed that such practices are still ineffective as they give rise to differential settlement.

EPS-block geofoam is among the popular technology that has been used for decades when dealing with soft yielding ground. Unfortunately, two failures associated with buoyancy forces on EPS and water fluctuations have been reported by Frydenlund & Aaboe (2001) and Horvanth (2010). The first failure occurred in 1987 in Norway, where the flotation of the mat structure was due to an extreme flood event. The second failure occurred in Thailand involving an unexpected high water level that caused a complete road fill to be washed away. Another failure caused by fire was reported in Norway. Ordinary polystyrene is a combustible material and will burn when set on fire.

Hence, this study takes a holistic approach to develop a sustainable lightweight technology with an open cellular structure and recycled plastic, which is known as geo-cellular mat (GCM) block. It is made up of a cell structure that promotes load sharing to compensate for any weak-performing isolated cells and eventually reduces excessive and differential settlement. Moreover, the open porous cellular structure accommodates high permeability characteristic of unhindered flow. This open structure permits excess water pressure through them and reduces potential buoyance or floatation when submerged in any groundwater regime. Furthermore, this structure also has the added benefit of accelerating the consolidation settlement of the subgrade consequent to the imposition of marginal surcharges from highway live loading that causes intermittent changes in excess pore water pressure which the GCM facilitates to dissipate. Another unique advantage of the cellular structure is that it surpasses the performance of particulate lightweight fill materials by its favourable soil structure interaction performance. Prevalent to all cellular structures is the large weight reduction compared to a solid mass material; this also inherently reduces the material cost due to the open cell structure. The present study is focused on investigating the efficacy of GCM with the intent to mitigate absolute and differential settlements via field performance.

The idea is to use recycled plastic as an alternative material in producing GCM, considering their characteristics of high strength, durability, and stiffness with low self-weight. Moreover, the use of plastic waste throughout the world in a wide variety of applications including for engineering purposes appears to be one of the popular routes in plastic waste management, leading and contributing to sustainability (Consoli, 2002; Babu & Choukse, 2011; Nsaif, 2013). Furthermore, these plastic wastes remain non-degraded in the natural environment, even after a long period of exposure. Verma (2008) claimed that they take a long time to break down or maybe stay unchanged.

MATERIALS AND METHODS

GCM block is principally produced using recycled polypropylene (rPP). The production of GCM is divided into four phases, which are; (1) raw material in pellet form, (2) produced

in tube form, (3) production of GCM block in the cellular mat structure, and (4) process to cover both the top and bottom of the GCM block with high strength fabric. The first and second phases were done by Tiger Polymer (M) Sdn. Bhd. and this research started from the third phase, which was GCM manufacture. GCM is produced as a cellular mat structure that has high permeability, facilitating unhindered fluid flow due to the open porous-structured cells. GCM has a distinct advantage of having virtually no buoyancy upon submergence. The open porous cellular structure of the GCM helps accelerate the consolidation settlement within the subgrade through the resulting ease in the dissipation of any excess pore water pressure. Both the top and bottom surface of the open porous GCM were covered with high strength filter fabric to avoid soil particle from passing through them. Additionally, the cellular mat structure further enables the sharing of the load as well as minimises differential settlement. Figure 1 shows the GCM blocks with a dimension of 1.0 × 1.0 × 0.2 m was used in this test, having a density of 125 ± 10 kg/m^3, approximately 16 times lighter than conventional earth fill (average density of 2000 kg/m^3). Its density was reduced through the open-cell structure, which was influenced by the geometry of cell.



Figure 1. GCM Block

A thermal behaviour study shows that the melting temperature of rPP is higher than 100 °C. It can be used as a safe and stable material for construction as the underground temperature ranges from 25 to 40 °C. Gan and Tan (2003) reported that the EPS geofoam is still stable when subjected to a temperature of 75 to 85 °C, but beyond this temperature some instability might form.

Moreover, the GCM block provides superior mechanical characteristics in terms of compressive strength (2.8 to 4.5 MPa) and initial stiffness (100 to 150 MPa) compared to EPS geofoam. As noted by Zornberg *et al.* (2005), the maximum compressive strength and initial stiffness of EPS is around 40 to 690 kPa (0.04 to 0.69 MPa) and 4000 to 10000 kPa (4 o 10MPa), respectively.

Soil Condition

Field testing was conducted at Parit Nipah Darat, Johor, Malaysia. It is located at Latitude of 1°50’07.1” and Longitude of 103°11’04.6”. The area has a peat layer approximately 4 m deep and is underlain by a layer of soft clay. In this area, four boreholes were drilled up to 10 m depth. Field vane shear tests were carried out to measure the soil shear strength from four locations.

The natural moisture content was in the region of 698% and 721%. Laboratory determination of bulk density shows that the unit weight of peat soil is in the range of 920–1100 kg/m^3. The specific gravity of peat layer obtained from pycnometer test is about 1.44. The measured shear strength of peat layer obtained from the field vane shear test is of low shear strength in the range of 8 to 14 kPa, as revealed from the natural water content. As expected, due to its low shear strength, this soil has low bearing capacity, and hence would not be able to support much foundation loading. All tests were carried out accordance with British Standard BS1377:1990.

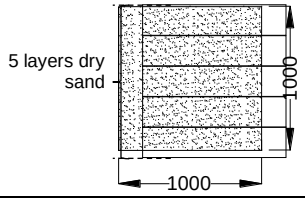
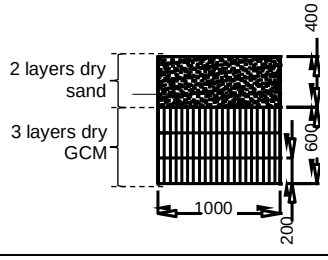
Environmental Observation

The environmental conditions include rainfall, groundwater table fluctuation and soil surface subsidence at the site were regularly monitored during testing. It was done to compensate errors arising from the environmental effect as well as to obtain the net settlement of the embankments due to fill loading only. The rainfall observations were secondary data that was provided by Department of Irrigation and Drainage. Whilst, a water level indicator with a flat measuring tape was used to measure the depth to the local groundwater level in standpipes that was permanently installed in the ground.

Field Experimental Setup

The study involved fieldwork in order to verify the effectiveness of the GCM fill in mitigating excessive and differential settlement in soft ground compared to conventional natural fill material (sand fill). This field study consisted of two field tests as listed in Table 1, and it was conducted for nearly a year at full scale. Test 1 consisted of conventional sand fill ($F1_{SF}$) as a flexible foundation base, and Test 2 consisted of GCM fill ($F2_{GCM}$) representing a rigid foundation base. Both fill materials were constructed to simulate a particular cross-section of road embankment with a uniform static load corresponding to a height of 1 m and a base area of 1 m^2. Dry sand fill was chosen to represent natural fill material for easy controlling of their condition (weight, moisture content, etc.) as well easy handling and placement on site. Sand fill was also used by Montepara & Giuliani (2000) and Freilich & Zornberg (2009) to compare their research on embankment fills.

Table 1. Details of Field Test

Field Test Code	Diagram	Description
$F1_{SF}$		Total of 1 m thick of sand (density of 1400 kg/m^3 ± 2%) placed on the ground in 5 layers (200 mm thick for each layer). Without treatment.
$F2_{GCM}$		Total of 0.6 m thick (3 layers) of dry GCM (density of 109.52 kg/m^3) with block size of 1 × 1 × 0.2 m. About 0.4 m thick (approximately 2 layers) of sand was spread on the top of the GCM as soil cover.

Each GCM block was 200 mm thick; three such blocks constituted a total thickness of 600 mm of lightweight fill. A total of 400 mm of sand was placed above the top GCM as a soil cover, making the total height of the GCM fill and conventional fill to be 1 m as illustrated in Table 1. The thickness of 200 mm was chosen for convenience (due to the availability of the material, ease of handling, cost and cutting technology), but it can be of any size depending on the design. A minimum 400 mm thickness of soil cover was used to minimise the stress impact on the GCM fill and to limit the overall bearing pressure exerted by the embankment and traffic loading. That was also recommended by Ganasan, Lim and Wijeyesekera (2016), and Stark and Arellano (2003).

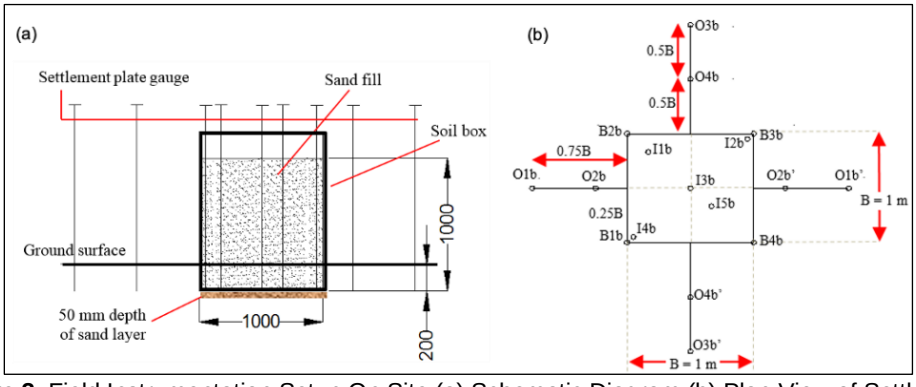


Figure 2. Field Instrumentation Setup On Site (a) Schematic Diagram (b) Plan View of Settlement Plate Gauge

Settlement plate gauges were used to measure the total ground movement (settlement or heave) beneath the fill loading at the interface between fill materials and subgrade surface. Monitoring of the field settlement was using an improvised digital automatic level. Seventeen settlement plate gauges were installed on the interface ground surface of each field test as shown in Figure 2(a) and (b) (setup for $F1_{SF}$), and a similar setup was installed for $F2_{GCM}$. Five settlement plate gauges were installed at various locations beneath the fill loading. One of the settlement plate gauges was installed at the centre of the fill loading to monitor total (maximum) settlement at the centre. Furthermore, eight settlement gauges were installed at distances of 0.5B, 1.0B, and 1.5B (B is equal to width of loaded area = 1 m) from the fill loading area. This was to observe and verify the ground movement adjacent to the loading area. The additional settlement plate gauges were also attached at each corner of the soil box. These were monitored regularly over the test period.

All the field tests were carried out under waterproof tents to protect fill material from rain. This was also to protect the instrumentation from various environmental conditions (hot and cool weather) to minimise temperature and rainfall effects. This was so that the field tests were only influenced by the presence of underground water. The effect of rising and falling groundwater table was carefully monitored. The general construction sequences for field tests are as follows:

1. Approximately 200 mm of topsoil was excavated and removed from ground surface, it is root depth from the local pineapple plantation.
2. About 50 mm depth of sand layer was spread on the ground to provide a flat surface.

3. The hollow soil box was placed in position on test area and settlement plate gauges were installed at fixed position. (A new innovative settlement plate gauge with barcode staff was specially developed for this research.)
4. For $F1_{SF}$, soil boxes were filled with sand in 200 mm fills, up to 1000 mm depth in five layers (200 mm for each layer). For GCM fill ($F2_{GCM}$), three blocks (each block being 1 × 1 × 0.2 m) which constituted a total GCM thickness of 600 mm were placed on the 50 mm-levelled surface of sand. A further 400 mm of fill consisting of two layers of 200 mm thickness of sand were placed above the top GCM layer (which was recommended as a minimum thickness of soil cover in highway embankment construction).
5. The construction loading settlements were monitored for every 200 mm depth of fill material layered in the soil box up to 1000 mm throughout settlement plate gauge, as shown in Figure 3. The settlement was continuously monitored for nearly a year.

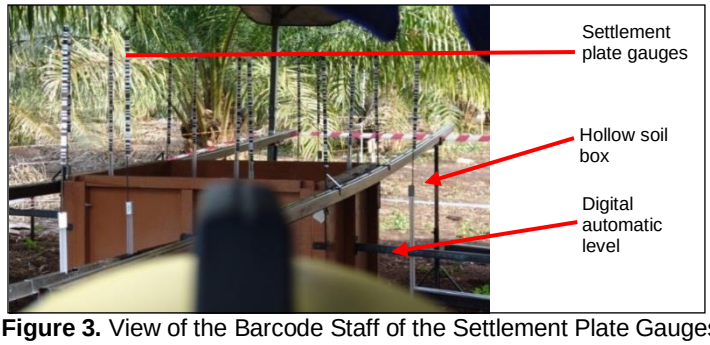


Figure 3. View of the Barcode Staff of the Settlement Plate Gauges

RESULTS AND DISCUSSIONS

Heave and Settlement of Ground Due to Change in Groundwater Level

A typical settlement plate gauge was used to measure settlement at the centreline of the fill loading. The readings over the test period are shown in Figure 4, background of the groundwater level. The ground initially settled approximately −33 and −50 mm and between the installation day and 176 h thereafter for both $F1_{SF}$ and $F2_{GCM}$. The overall observation shows the total settlements were reduced by 35% to 50% with the application of GCM fill compared to sand fill.

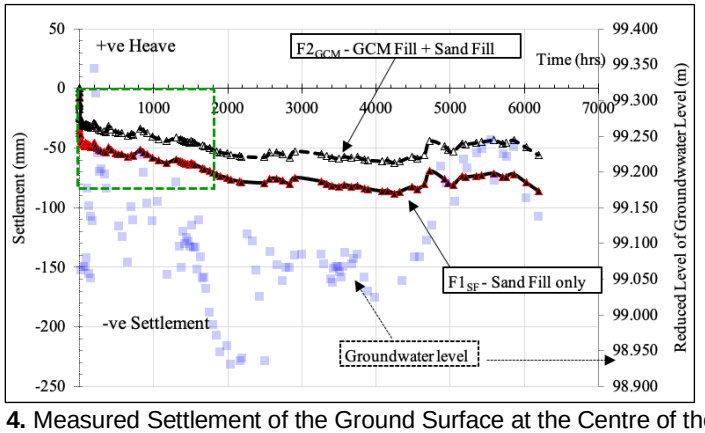


Figure 4. Measured Settlement of the Ground Surface at the Centre of the Field

Figure 5(a) shows other events, such as heave point, that affect the settlement profile. Meanwhile, Figure 5(b) shows the rainfall and groundwater level patterns during the test period. This figure clearly shows the groundwater level responses to three conspicuously high rainfall events at time, t = 191 hrs (denoted as A), 815 hrs (denoted as B) and 1367 hrs (denoted as C). At t=191.20 hrs during a period of heavy rainfall, the ground under both $F1_{SF}$ and $F2_{GCM}$ fill loading swelled, and the foundation then heaved to 41 and 21 mm, respectively. At the same period, it was noticeable that the reduced groundwater level rose to 99.4300 m. A similar trend also occurred at points B and C within the test period up to 1500 hrs when the groundwater level rose. Over the test period, the settlement plate gauge readings fluctuated (+ve heave and −ve settlement reading), depending on the rising (up) and falling (down) of groundwater level. This was reflected in the reduced effective stress and swelling of the ground due to the increased moisture content. High temperature reduces ground moisture, increasing effective pressure and causing it to settle.

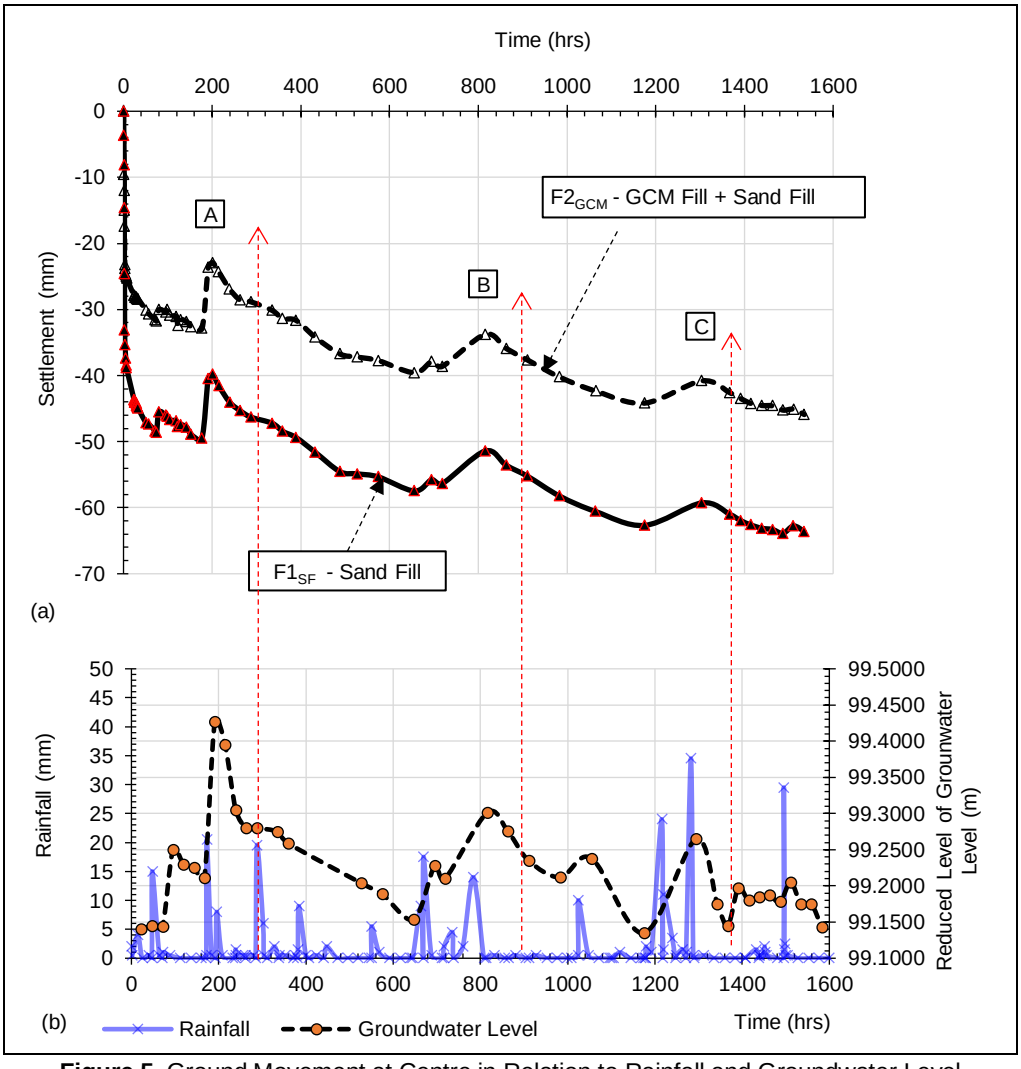


Figure 5. Ground Movement at Centre in Relation to Rainfall and Groundwater Level

This effect has been similarly reported by previous researchers such as Vipulanandan et al. (2008), Atkinson (2007), Platt (2013), and Salem & El-Sherbiny (2014). Based on the observation, with heavy rainfall event, the ground swelled (causing it to heave), and settled when groundwater level dropped, causing it to shrink in dry seasons. This is related to the changes of effective stress in the ground. The reduction of effective stress causes ground to swell and lifts the foundation (fill material), and vice versa.

Settlement Performance Under $F1_{SF}$

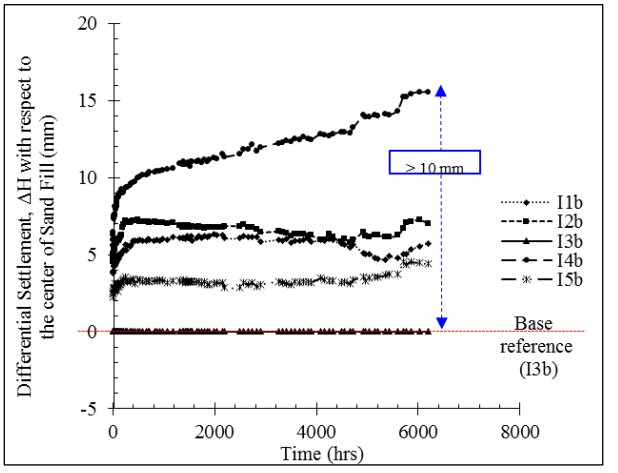


Figure 6. Differential Settlement with Respect to the Centre of $F1_{SF}$

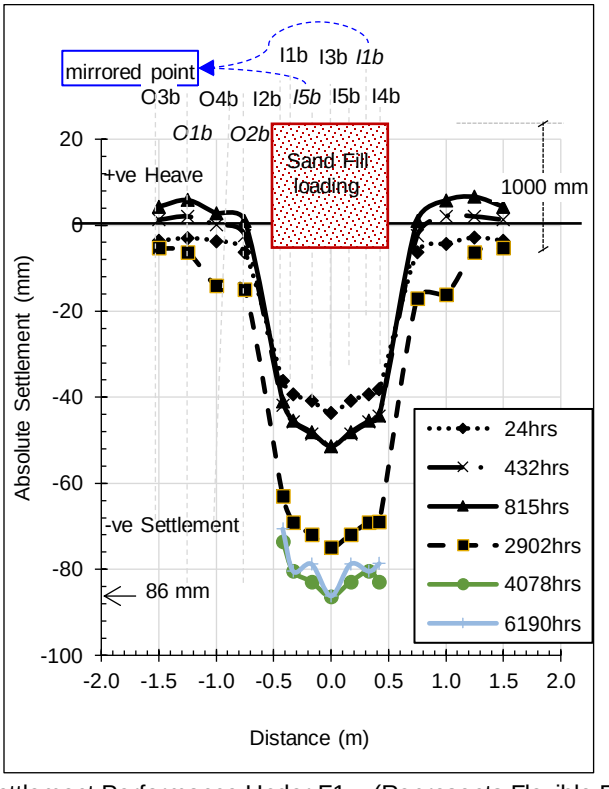


Figure 7. Settlement Performance Under $F1_{SF}$ (Represents Flexible Foundation)

Figure 6 shows the localised differential settlement with respect to the centre of sand fill ($\Delta H_x = \delta_x - \delta_{\mathrm{I3b}}$); the point I3b as a base reference was more than 10 mm for test $F1_{SF}$. The differential settlement at point I2b was only half from what was observed at I4b, even though they were at the same position, which was 2.5/3 mm away from the centre, but on opposite sides of it. These results are due to the non-homogeneity in the peat ground as well as the non-uniform peat depth.

The maximum absolute settlements recorded were 86 and 71 mm observed at the centre (ΔH_3) and edge (ΔH_2) of fill sand loading F2, respectively. This gives a $\Delta H_3/\Delta H_2$ ratio of 1.211. Furthermore, the observation presented in Figure 7 confirms a non-uniform settlement with a sagging profile under sand fill (flexible foundation behaviour).

Settlement Performance Under $F2_{GCM}$

Figure 8 indicates the differential settlement with respect to the centre of GCM fill ($\Delta H_x = \delta_x - \delta_{\mathrm{I3a}}$); the point I3a as a base reference was less than 3 mm for test $F2_{GCM}$. The differential settlement at points I2a and I4a were not the same in this case. However, the difference observed in test $F2_{GCM}$ was only 2 mm. This observation shows that the GCM fill has overcome the non-homogeneity issue on the peat ground.

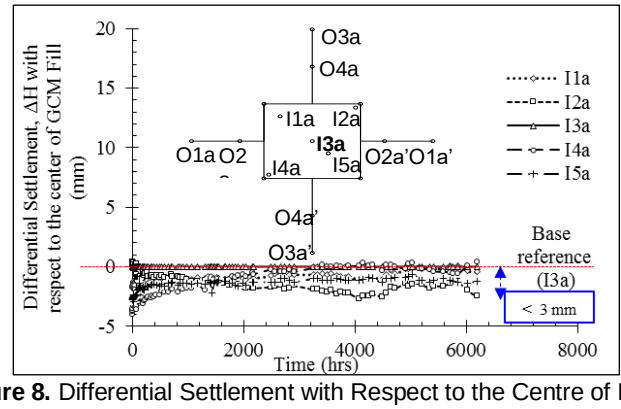


Figure 8. Differential Settlement with Respect to the Centre of $F2_{GCM}$

The maximum absolute settlements recorded were 56 and 55 mm at the centre (ΔH_3) and edge (ΔH_2) of GCM fill loading, respectively at t = 6190 h. The uniformity of the settlement ratio ($\Delta H_3/\Delta H_2$) is 1.018 for this case. Figure 9 clearly shows that the GCM fill has a uniform settlement behaviour similar to that of a rigid foundation.

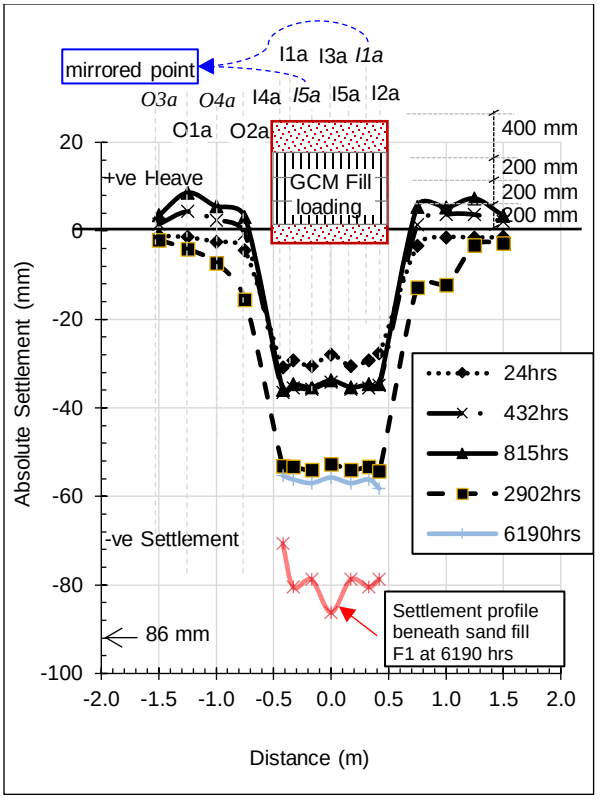


Figure 9. Settlement Performance Under $F2_{GCM}$ (Represents Rigid Foundation)

Field Performance of GCM ($F2_{GCM}$) Versus Sand Fills ($F1_{SF}$)

Figure 7 and Figure 9 show the settlement behaviour under sand fill ($F1_{SF}$) and GCM fill ($F2_{GCM}$), respectively. The maximum absolute settlements recorded were 86 mm at t = 6190 h at the centre of sand fill loading in test $F1_{SF}$ (Figure 7). Compared to test $F1_{SF}$, the maximum absolute settlement (56 mm at t = 6190 h) was reduced by approximately 35% with the application of GCM fill in test $F2_{GCM}$ (Figure 9).

Furthermore, Figure 9 perfectly shows that the GCM fill has a uniform settlement behaviour similar to that of a rigid foundation. Hence, the localised differential settlement with respect to the centre of fill ($\Delta H_x = \delta_x - \delta_{l3b}$) shows that the differential settlement under GCM fil loading $F2_{GCM}$ (Figure 8) was 70% less than that observed within the sand fill loading $F1_{SF}$. Therefore, this research proves that the GCM fill effectively makes the settlement become uniform, even on a varied foundation soil strength. Atkinson (2007) and Das (2011) also claimed that a rigid foundation results in a uniform settlement, and the settlement will vary under flexible foundation.

CONCLUSION

The lightweight fill material has undergone a series of field tests and showed promising results, confirming that the concept of the geo-cellular mat (GCM) is effective in mitigating both excessive and differential settlements. From the field observation, the maximum absolute settlement and differential settlement were reduced by 35% and 70%, respectively with the

adoption of GCM fill compared to sand fill. Moreover, this study shows that the GCM fill have addressed the problem of non-homogeneity on peat ground. GCM fill not only reduces the excessive and differential settlement but also helps accelerate the consolidation settlement within the subgrade through the dissipation of the excess pore water pressure with its open porous cellular structure. The other relevant remarkable features of the GCM are:

- Lightweight (approximately 90 % less than conventional sand fill).
- Excellent mechanical properties, consisting of compressive strength and stiffness more than 2.8 MPa and 100 MPa, respectively.
- Easy to handle, install and transport (reduce time and construction cost).
- Routine field compaction is not required or minimised (reduced construction cost and time).
- Permeable/porous embankment (against rising groundwater table and efficient dissipation of excess pore water pressure reducing the risk of shear failure).
- The viability of this product can also address the environmental concerns through reduction of municipal solid wastes, as they are developed from recycled materials.

ACKNOWLEDGEMENT

The authors would like to thank the Ministry of Education Malaysia for the financial support under Fundamental Research Grant Scheme (FRGS) Vot. No. FRGS/1/2018/TK10/UTHM/03/3 and Universiti Tun Hussein Onn Malaysia for the partial sponsor.

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MONITORING OF NATURAL DAYLIGHT BY USING LIGHT PIPE SYSTEM IN BUILDING

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Abstract
Daylighting is an effective sustainable strategy in illuminating building interiors with natural light during daytime. Light pipe system is one invention that transports daylight efficiently into the core of a building by using highly reflective tubes, 95% to 98%. However, this highly reflecting material costs relatively high. Therefore, this research aims to explore the light transmission of light pipe using reflective materials that are easily available locally at a lower cost. A straight light-pipe using 74% reflective material for the internal surfaces was installed in a test room located in Batu Pahat. Indoor illuminance and concurrent outdoor illuminance were monitored at 5-minutes interval for a month. Results showed that clouds, solar altitudes and external illuminance were affecting the performance of the system. This average internal illuminance level fulfilled the MS 1525:2014 recommendation for application in Malaysian buildings. This indicates that the light pipe system has a potential as a tool for introducing indoor daylight in Malaysia.

Keywords: *Daylight; illuminance; light pipe system; test bed.*

INTRODUCTION

Introduction of natural daylighting system in a building has many advantages and benefits, ranging from aesthetical to physiological and economical aspects. Daylight that is shone into interior building through window can inform people on the changes of external environment. Spaces illuminated by daylight will provide occupants with high satisfaction of visual and thermal comfort along with low energy consumption for lighting, heating and cooling (Edmonds & Pearce, 1999; Li & Lam, 2003; Reinhart & Weissman, 2012). Daylight illumination can come directly from the sun as light diffuses through the atmosphere and reflected from external surfaces. Side lighting from windows is a prominent architectural aspect in building design. Besides lighting purposes, windows offer ventilations, views and fresh air. These help to provide relaxation and inspiration to the occupants. However, the illuminance level from windows will decrease rapidly with distant increase. Therefore, the area which is distant from the windows may tend to look gloomy and requires supplementary artificial lighting. Even though electrical lighting provides sufficient levels of illumination, it cannot provide physiological benefits of natural light (Garcia-Hansen, 2006). Thus, top lighting strategies could be introduced to illuminate building interiors where sunlight cannot penetrate due to the building layout such as terrace buildings. This strategy can also provide satisfactory illuminance as it works more efficiently than windows (Treado, Gillette, & Kusuda, 1984).

Light pipe systems are linear devices that capture, transmit and reflect natural daylight into the interior spaces. The light pipe systems comprise of a collector, a light tube and a diffuser. A collector is usually located at the roof level and it is made of translucent dome to

accept sunlight from the whole sky hemi-sphere. Light tube acts as a light transporter that will guide the light into the rooms through multiple reflections of daylight. A tube with highly reflective internal surfaces, like aluminium sheets with reflectance of about 95 % to 99%, increases the efficiency of transmission (Zhang, Muneer, & Kubie, 2002). Simpler to install, this system is suitable to be incorporate in new buildings or as a retrofitted to an existing one. Installation of the light pipe systems will substantially cost higher compared to the electrical systems and conventional windows, due to its nature as advanced technology implementation for collecting higher amount of daylight (Mayhoub, 2013). Therefore, to ensure economic viability of the system, it is important to use materials that are inexpensive and easily available locally. Polished aluminium with 74% surface reflectivity is found to be the most suitable material to fulfil the photometric performance in Malaysian climate (Ahmed, Zain-Ahmed, Rahman, & Sharif, 2006). However, there is still a lack of information on light transmission efficiency of mirror finished materials in Malaysia.

The performance of light pipe system as natural daylighting system has been reported in a number of studies (Carter, 2005; Marwaee & Carter, 2006; Mohelnikova, 2009; Oakley, Riffat, & Shao, 2000; Petr, Kocifaj, & Kómar, 2018; Shin, Yun, & Kim, 2011). These studies have verified the efficiency of light pipe system as a light source in a building. An experimental study carried out by Oakley et al. (2000) reported that light pipes are proficient devices for introducing daylight into buildings. Surveys conducted by Al-Marwaee and Carter (2006) on 13 buildings have found that light-pipe systems could provide 25% to 50% of the work plane illuminance and tend to reduce lighting energy consumption. In another study, Mohelnikova (2009) revealed that interior illuminance on the working plane could vary depending on the sky condition.

Malaysia is a tropical country located in the equatorial region. The climatic features of Malaysia are classified as hot-humid tropics with uniform temperature, high humidity, copious rain-fall, light winds, and abundant sunshine (Heng et al., 2019; Munaaim, Al-Obaidi, Ismail, & Abdul Rahman, 2014). Table 1 shows three types of sky condition that are commonly found (Lim & Ahmad, 2014).

Table 1. Types of Sky Condition

Sky Condition	Description
Clear Sky	A cloudless or almost cloudless sky, where no more than 30% of the sky dome is obscured by clouds. The external illuminance values range from 50,000 lux to 100,000 lux.
Intermediate Sky	Ranges from 30 to 80% cloud. The illuminance value can vary from 20,000 lx during cloud cover to 100,000 lux when the sunlight is not obstructed by the clouds
Overcast Sky	The sun is covered by the clouds and not visible; at least 80% of the sky dome is obscured by clouds. The illuminance from the overcast sky is more consistent and predictable, which ranges from 5,000 lux to 20,000 lux.

Daylight data is an important factor that needs to be taken into account when designing buildings using daylighting system since the quality of the external illuminance could affect its performance. For daylight availability in Malaysia, a research done by Zain-Ahmed, Sopian, Zainol Abidin, and Othman (2002) showed that the illuminance exceeds 80,000 lux at noon during March when solar irradiation is the highest and even reaches 60,000 lux during December when solar irradiation is less intense. This climate shows that there is a high potential for using light pipe as a daylighting system in Malaysia. The Malaysian Standard 1525: 2014 has outlined illuminance levels recommendation for various tasks and

applications in a building (Department of Standards Malaysia, 2014). Table 2 shows the recommended average illuminance levels for Malaysian building. The aim of this research is to explore the photometric performance of light pipe system using mirror finished material which are easily available in Malaysia by comparing internal illuminance yield in various external conditions, and to test whether the performance of proposed light pipe system complies with MS 1525:2014.

Table 2. Recommended Average Illuminance

Task	Illuminance (lux)	Example of Applications
Lighting for infrequently used area	20	Minimum service illuminance
	100	Interior walkway and car-park
	100	Hotel bedroom
	100	Lift interior
	100	Corridor, passageways, stairs
	150	Escalator, travellator
	100	Entrance and exit
	100	Staff changing room, locker and cleaner room, lavatories, stores
	100	Entrance hall, lobbies, waiting room
	300	Inquiry desk
	200	Gate house
Lighting for working interiors	200	Infrequent reading and writing
	300-400	General offices, shops and stores, reading and writing
	300-400	Drawing office
	150	Restroom
	200	Restaurant, canteen, cafeteria
	150-300	Kitchen
	150	Lounge
	150	Bathroom
	100	Toilet
	100	Bedroom
	300-500	Class room, library
	200-750	Shop/supermarket/department store
	300	Museum and gallery
Localised lighting for exacting task	500	Proof reading
	1000	Exacting drawing
	2000	Detailed and precise work

(Source: MS 1525:2014)

METHODOLOGY

Light pipe system is an approach used to improve daylight penetration as to reduce the dependency on artificial lighting during daytime. It is the easiest way of improving energy efficiency and attaining energy saving. In order to explore the fundamental light pipe transmission efficiency, field measurement using a test bed model with dimensions of 5.4m (L) X 3.0m (W) X 3.0m (H-Ceiling) was built at 1^0 51'N latitude, 103^0 5'E longitude inside the main campus of Universiti Tun Hussein Onn Malaysia as shown in Figure 1.

The test bed was designed to be windowless to achieve total darkness and to obtain accurate results. The indoor walls were finished with white paint. Typical cement rendering screed was chosen as the floor finish, and a set of white plaster boards were used as ceiling. A proposed light pipe model was mounted at the center of the roof to represent the system (Figure 2a).



Figure 1. Actual Location of Experimental Building

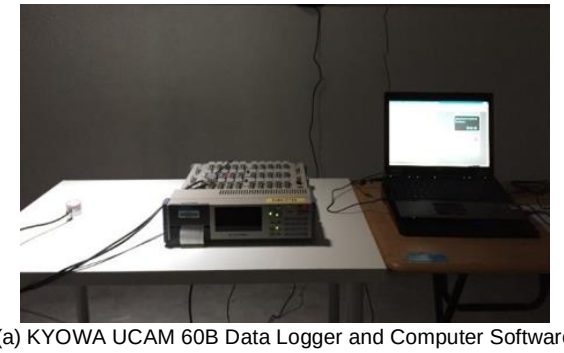
The light pipe was 1.20m height with 250mm diameter. The pipe was made using polish aluminium with 74% reflectance. The top of the light pipe was fitted with a clear acrylic dome and a polycarbonate diffuser was attached to the lower opening of light pipe. The light pipe was 3m from the floor to the bottom of the diffuser (Figure 2b).



(a) Proposed Light Pipe System (b) Indoor Illuminance

Figure 2. Light Pipe System Experiment

The test bed was equipped with measuring equipment. Since this study aimed to explore the light pipe transmission efficiency, the system logged the data into data acquisition system connected to calibrated lux sensors of computer software (Figure 3a). KYOWA UCAM 60B Data Logger was used in this field measurement. Internal and external illuminance was taken concurrently at every 5 minutes in February 2017 from 8:00 am to 5:00 pm.



(a) KYOWA UCAM 60B Data Logger and Computer Software



(b) External Lux Sensor LSI-Lastem ESR003#S



(c) Internal Lux Sensor LSI-Lastem ESR001

Figure 3. Data Logger and Illuminance Sensors

One lux sensor LSI-Lastem ESR003#S was installed on the roof-top to measure the external illuminance. The measurement range of the equipment was 1-150klux (Figure 3b). For indoor illuminance, lux sensor LSI-Lastem ESR001 was placed inside the room (Figure 3c). The measurement range of the equipment was 1-25,000 lux. The positioning of the sensor was at the work plane with the height of 750mm. The sensor was placed in the middle of the room.

FINDINGS

The illuminance level obtained by the light pipe system was in response to the different types of sky condition. The daily time from 8:00am to 5:00pm was selected for data collection in February 2017. This period was chosen because it represented normal office daytime working hours and the behaviour of typical sunrise and sunset times.

External Illuminance Distribution

The external illuminance was measured during various sky conditions. Figure 4 shows the distribution of average external illuminance during the monitoring period. The illuminance steadily increased when the sun rose and gradually decreased in the afternoon. The figure indicates that the reading formed a bell-shaped curved with maximum reading of

80 klux at noon time. Meanwhile, during the morning and afternoon, the average external illuminance was 20 klux and 50 klux respectively.

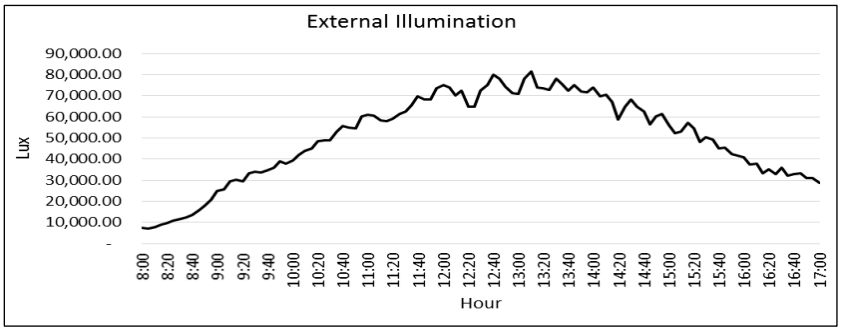


Figure 4. Average External Distribution

Table 3 shows the readings of external illuminance at the highest solar altitude during the monitoring. The highest reading recorded was 119 klux, 117 klux and 128 klux at 77^0, 78^0 and 79^0 respectively. This result corroborated previous studies on tropical daylighting which proved that tropical daylight availability can exceed 100klx at noon time in Kota Kinabalu (Djamila, Ming, & Kumaresan, 2011) and a recorded value of 130 klux in Johor Bahru (Lim et al., 2012). However, illuminance reading recorded during overcast condition at the same solar altitude dropped to 15 klux, 11 klux and 21 klux. This was due to the cloud formation that affected the availability of illuminance (Lim & Ahmad, 2014).

Table 3. External Illuminance Levels at Highest Solar Altitude

Solar Altitude(^{0})	Illuminance (lux)	
77^0	Highest	119,394
	Lowest	15,375
78^0	Highest	117,495
	Lowest	11,295
79^0	Highest	127,753
	Lowest	20,775

Figure 5 shows the distribution of the measured external illuminance during the monitoring time. The distribution of illuminance was divided into three sky conditions of clear, intermediate and overcast. The sky conditions were distinguished on the basis of external illuminance values as described in Table 1. The measurement showed that more than 40% of the monitoring period fell into clear sky condition, 33% for intermediate sky and 22% obtained under overcast sky condition. The average external illuminance was 79,755 lux (Standard Deviation, SD = 20336) for clear sky condition, 34,719 lux (Standard Deviation, SD = 7877) for intermediate sky condition and 10,933 lux (Standard Deviation, SD = 5147) for overcast sky condition. The results showed that there is abundance of daylight in Malaysian tropical sky which is in line with studies conducted by Heng et al. (2019) and Munaaim et al. (2014).

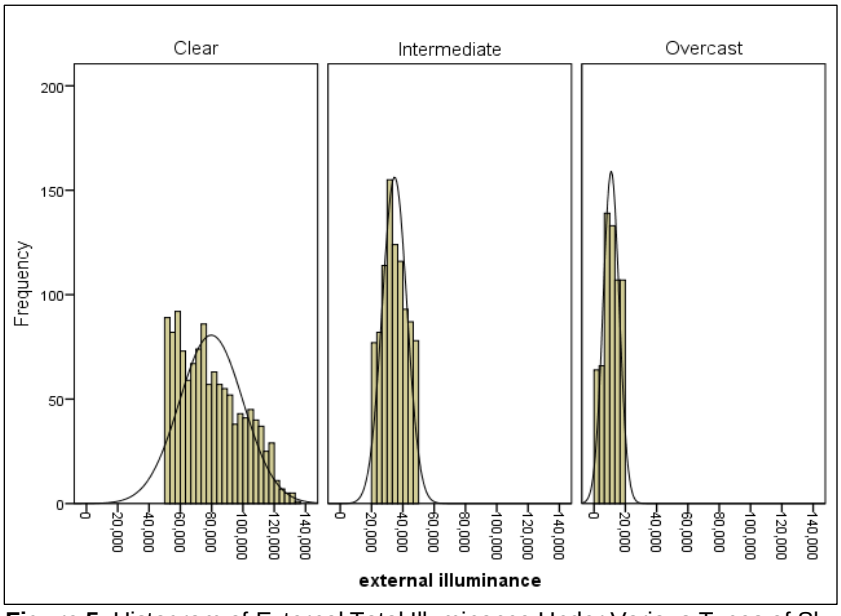


Figure 5. Histogram of External Total Illuminance Under Various Types of Sky

Internal Illuminance versus Solar Altitude

Figure 6 shows the positive relationship between internal illuminance level and solar altitudes. There was a direct relationship between internal illuminance and solar altitude (R^2 = 0.431). In other words, the contribution of solar altitude was 43.10% to internal illuminance. The coefficient of determinant for clear sky was 0.259 (R^2 = 0..259) while the value for intermediate was 0.139 (R^2 = 0.139) and 0.039 (R^2 =0.039) for overcast. This indicates that the type of sky condition is an important factor affecting the relationship between internal illumination and solar altitude.

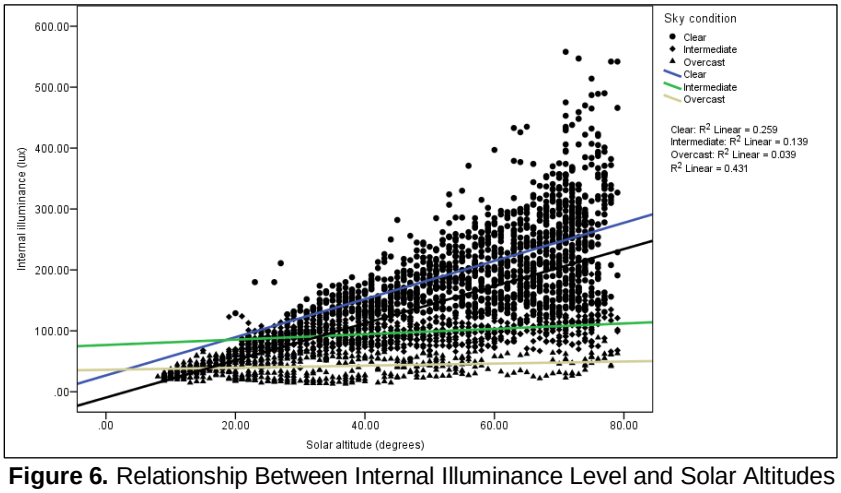


Figure 6. Relationship Between Internal Illuminance Level and Solar Altitudes

Internal Illuminance Versus External Illuminance

Figure 7 illustrates the relationship between internal and external illuminance levels. It indicates there are variations in the strength of relationship between internal and external illuminance levels for different types of sky condition. There is a positive strong linear relationship between external and internal illumination (R^2 = 0.943). In other words, the contribution of external illuminance is 94.30% to internal illuminance.

The result showed that for clear and overcast sky conditions, there were slight variations in the strength of relationship between internal and external illuminance levels. The coefficient of determinant for clear sky was 0.898 (R^2 = 0.898) and the value for overcast was 0.910 (R^2 = 0.910). Comparing the results, it can be seen that the regression coefficient for overcast sky was bigger than clear sky condition. These results indicate that the level of external illuminance is the most important factor in the quality of light in buildings compared to solar altitude.

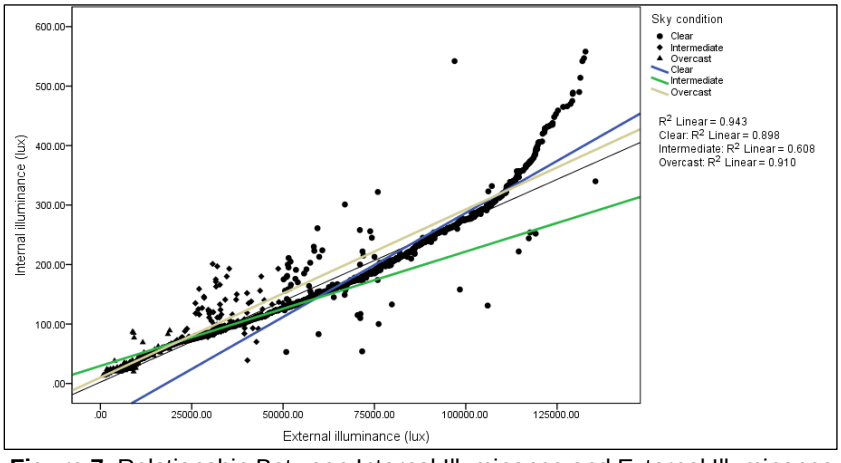


Figure 7. Relationship Between Internal Illuminance and External Illuminance

Internal Illuminance Distribution

Figure 8 shows the distribution of internal illuminances over the monitoring period. The graph showed that internal illuminance was less than 150 lux in the early morning and late evening, but reached a maximum of over 350 lux at noon and early afternoon. Since the sky was mostly covered by cloud, it affected internal illuminance distribution. The highest illuminance level on Day 1 was 400 lux at 1.30 pm, and for the same time, it reached 371 lux, 340 lux, 430 lux and 340 lux on Day 2, Day 3, Day 4 and Day 5 respectively. Starting from 10 am, internal illuminance met the minimum requirement of MS 1525:2014 with 100 lux. These outcomes revealed that the system started to work effectively at about 10am with 40,000 lux.

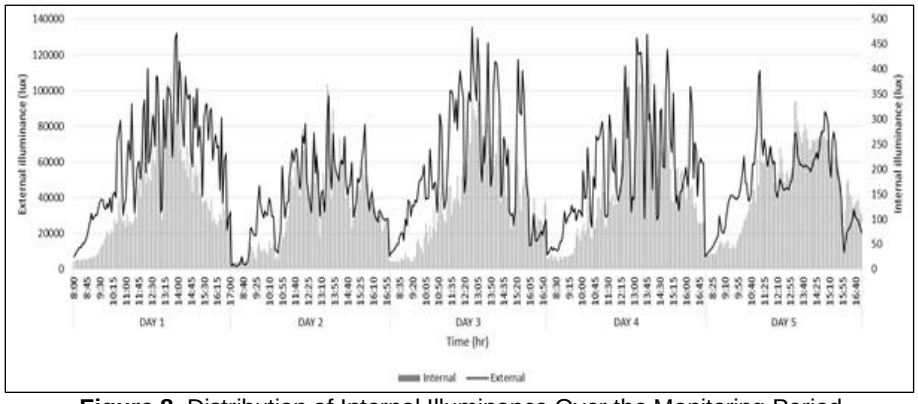


Figure 8. Distribution of Internal Illuminance Over the Monitoring Period

Since the external illuminance was significantly influenced by the sky condition, Figure 9 shows the average variation of workplane illuminance during the monitoring period. In the clear sky condition, the light pipe system was detected start its operation effectively at around 9.30am and gradually increased in the noon, associated with an increasing of solar altitude. The highest average workplane illuminance was around 250lux to 300lux at the same time of highest solar altitude. Even though there is inconsistent reading due to cloud formation, but the workplane illuminance still in the recommended range. During the intermediate sky, the internal illuminance varied from 50 lux to 200lux. Similar to the conditions occur during the clear sky. During the cloudy day, the internal illuminance was varied between 20lux in the early morning increase to 150 lux at 10am. It is expected the highest value will be recorded around the noon unfortunately due to cloud formation the illuminance levels are below 100lux with the external illuminance is around 20,000 lux. On average it can be seen the internal illuminance exceeding 100lx is 80% during the clear sky, about 50% when intermediate sky and around 25% in overcast sky. Since the illuminance levels are inconsistent throughout the day, artificial lighting can be used as a secondary source.

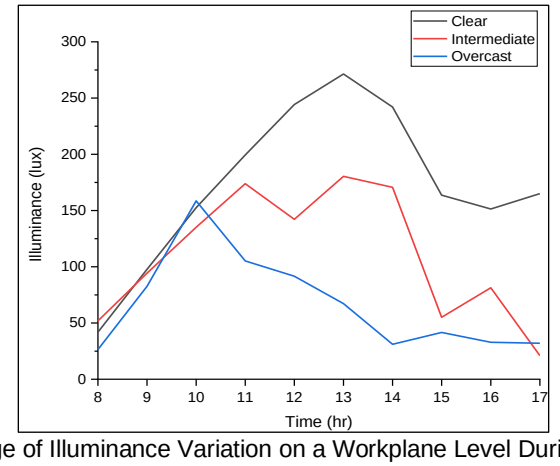


Figure 9. Average of Illuminance Variation on a Workplane Level During the Monitoring

CONCLUSION

In conclusion, this research has indicated that external illuminance influences the performance of the light pipe system. The sky condition is a major factor affecting internal

illuminance that enters the building which should be taken into account when fitting it in the building. This study has identified that the light pipe system could be an effective tool for introducing natural daylight into building interiors. Although the illuminance level from the system is inconsistent throughout the day, the system still shows that good daylighting efficiency can be achieved. The average internal illuminance level offered by the light pipe system under Malaysian climate fulfilled MS 1525:2014 recommendation for application in Malaysian buildings. It is hoped that the findings will contribute significantly to the widespread use of light-pipe systems in Malaysia, thus enhancing the occupant's well-being and health, and contributing to the energy efficiency of buildings.

ACKNOWLEDGEMENT

The research described in this paper was funded by Ministry of Higher Education Malaysia (MOHE) through Research Acculturation Grant Scheme (RAGS), VOT R064. The authors wish to thank the Centre for Diploma Studies, University Tun Hussein Onn Malaysia for providing a platform to carry out this research.

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EXPERIMENTAL STUDY ON STRENGTH PERFORMANCE OF SOIL REINFORCED WITH COCONUT FIBRE FOR PANCHOR–MUAR ROAD

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Abstract

Laterite soil is a typical earthfill material used in road construction base and sub-base layers. Regrettably, such soil becomes soft and saturated under high water content and causes an unsatisfactory performance in withstanding loads from proposed structures. Recently, the use of fibres to improve the soil layer, thereby increasing its geotechnical properties, has been attracting worldwide attention. Therefore, the present study takes a holistic approach to utilise coconut fibre (CF) as sustainable fibre reinforcement to enhance the strength performance of local laterite soil at Panchor - Muar, Johor. It then is used as an earthfill material for construction work. This research focuses mainly on investigating the compaction parameters and bearing capacity of CF–laterite soil. Standard compaction test and California bearing ratio (CBR) test were performed on soil randomly included with three different CF proportions (0%, 0.5%, and 1.0%). Results indicate that the optimum moisture content increased and the maximum dry density decreased as the proportion of CF increased to 1.0%. The CBR value of the reinforced soil recorded a significant increase to 17.42% and 26.14% after being mixed with 0.5% and 1.0% CF proportions, compared with unreinforced soil (7.58%). The coconut fibre-reinforcement makes the stabilised soil ideal as a subbase material for road pavement applications. In addition to enhancing soil strength efficiency, CF reuse would also reduce the amount of agricultural waste disposal to landfills and prevent environmental problems.

Keywords: *California bearing ratio; coconut fibre; laterite soil; soil reinforcement.*

INTRODUCTION

The rapid pace of highway and infrastructure development involving problematic soil presents challenges for engineers; this is attributed to unsatisfactory bearing capacity and high post-construction compressibility arising from changes in the soil's physical properties under wet and dry conditions. The physical properties of the soil are therefore essential from the construction point of view, which caused a change in the soil structure, thereby altering soil strength. Mahmood et al. (2016) also stated that the soil issue due to the physical properties of the soil is closely related to the dispersive soil, collapsible soil, expansive soil, and laterite soil, which are also known as problematic soil (Ali, 2010; Rezaeil et al., 2012). This paper focuses on the issue of collapsible and laterite soils. Such soils are very sensitive to moisture or water. In dry condition, the soil can bear a heavy load without any settlement. However, when the soil becomes wet or comes in contact with moisture, its strength weakens and its volume decreases. This happens because the water fill the voids between the soil particles, resulting in a breakage of the soil structure. This rearrangement of the soil structure results in unnecessary settlement and might cause a structure on it to fail and collapse. Laterite soil under such conditions has poor geotechnical properties such as low strength, low bearing capacity, poor workability, and tendency to retain water content (Kulkarni & Sharma, 2016; Osinubi et al., 2009). Laterite soil, also known as red soil, might be an established material

on site and most of the time, it is commonly used as an earthfill material for construction purposes. Thus, it is necessary to improve the soil's geotechnical properties before it is used to enhance the strength of the soil without shear failure, bearing failure, and massive primary and long-term settlement.

Various alternative technologies including preloading, stone column, pile, vertical drain, lightweight material (EPS and tire chip), fibre-reinforced material (polymer, sugarcane, oil palm fruit fibre, and bamboo), chemical stabilisation, and additive material (cement, fly ash, lime, bitumen, POFA, or a combination of these) have been suggested and adopted for soil improvement especially for construction over problematic ground (Huat et al., 2005; Makusa, 2012; Construction Institute of Malaysia, 2015). However, some of these technologies (preloading, vertical drain, stone column, and pile) are constrained by technical feasibility and space and time limitations (Ismail, 2017).

Global trends today indicate that the marketplace has given more attention to the use of natural fibre, especially by-products from agricultural wastes, due to various societal and economic concerns (Hejazi et al., 2012). Numerous research works have been carried out to improve the physical properties of soil by using various agricultural fibre reinforcement including plant roots (Wood et al., 2016), banana fibre (Sunny & Joy, 2016; Gobinath et al., 2020), bamboo fibre (Ghavami et al., 1999; Asaduzzaman & Islam, 2014), bagasse fibre (Osinubi et al., 2009; Dang et al., 2017), coir fibre (Babu & Vasudevan, 2007; Khan, 2007; Peter, 2016), or a combination of these materials. Soil reinforced with natural fibre is a versatile product, as the fibre is available abundantly throughout the country and is produced at a cheaper cost. As such, it is an economical answer. Babu and Vasudevan (2007) stated that in engineering practice, the idea of soil reinforcement with tensile-resistant materials such as fibre has been generally recognised since the 1990s. Researchers such as Amadi, Eberemu and Momoh (2013), Patel and Singh (2014), Ayininuola and Oladotun (2016), and Anggraini (2016) reported that insertion of fibre into the soil has achieved a varying degree of success and has a significant impact in improving the strength performance of problematic soil to the desired strength required for Standard Specification for Road Works and thus improving the road structure. Kumar and Gupta (2014) also reported that the inclusion of randomly distributed fibre improves the strength properties of soil. Furthermore, fibre also has high tensile strength and good abrasion resistance and is beneficial for supporting tension load from traffics. Moreover, Kulkarni and Sharma (2016) reported that natural fibre is long lasting and can survive in the environment for 4 to 10 years, absorbs water, has high elongation at break, and has high tensile strength under wet condition. Babu and Vasudevan (2007) claimed that natural fibre has the greatest bearing strength and retains this property even in wet conditions to some extent, as it contains lignin and cellulose.

Hence, the current study focuses on investigating the potential of coconut fibre (CF) in enhancing strength performance of reinforced soil via unsoaked CBR test. The tests were subjected to the different CF ratios of 0, 0.5, and 1.0 %, and it has been selected on the basis of the pilot test. Water content, Atterberg limits, particle size distribution, specific gravity of virgin soil, and compaction parameters including optimum moisture content (OMC) and maximum dry density (MDD) values were determined. This study aims to find a sustainable and cost-effective approach to the problems of rutting, cracking, pothole, and settlement at Panchor–Muar road, Johor, Malaysia. Figure 1 shows a road at Panchor–Muar which is in poor condition due to high soil moisture content. Figure 1(b) indicates that after a week of

maintenance, the road starts to fail once again. According to Idrus et al. (2016), the road's base and sub-base layers were constructed from the local soil that was problematic soil, called silty clay. Therefore, the findings from this study will assist in future utilisation of similar materials for construction application.



(a) (b)

Figure 1. Road Failure at Panchor - Muar Road; (a) Cracking and Pothole Failure Occurs on Pavement Surface, and (b) Rutting Occurs After One Week of the Maintenance

MATERIALS AND TEST METHODS

Materials

The soil sample used in this study (Figure 2) was collected from Bandar Universiti Pagoh in the state of Johor in Peninsular Malaysia. It is located at 2°8′4.84″ N, 102°44′4.49″ E as shown in Figure 3. The disturbed sample of soil was obtained by using shovel and hoe at depth of 0.5–1.5 m below the ground surface. The soil is reddish-brow0n in colour. The soil samples were oven-dried at 105 °C for 24 h before being used for further geotechnical and strength tests.



Figure 2. Laterite Soil to be Tested

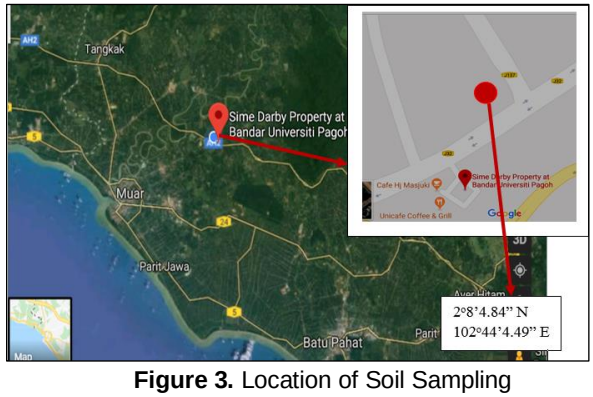


Figure 3. Location of Soil Sampling

Coconut fibre (CF) is one of the by-products from agro-waste extracted from coconut. The CF used in this study (Figure 4) was obtained from T & H Coconut Fibre Sdn. Bhd., Senggarang, Batu Pahat, Johor. The fibres were cut into 30 mm length with diameters in the range of 0.1–0.6 mm. The density of CF is 1.40 g/cm^3, and its specific gravity is 0.87. The CF properties are summarised in Table 1.

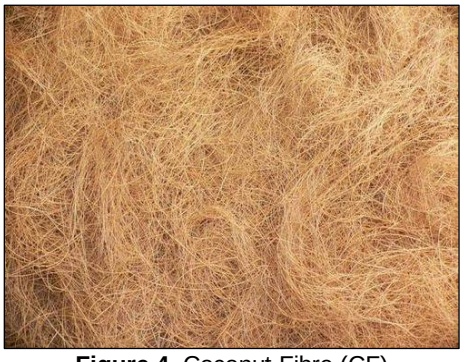


Figure 4. Coconut Fibre (CF)

Table 1. Properties of Coconut Fibre (CF)

Parameter	Unit	Value
Density	g/cm^3	1.4
Specific gravity	-	0.87
Water absorption	%	73.52

Test Methods

The geotechnical properties of soil such as particle size distribution, liquid limit, plastic limit, specific gravity, standard proctor compaction, and California bearing ratio (CBR) were conducted in accordance with British Standard (BS 1377: 1990). The sieve analysis and index properties test including Atterberg limits (liquid limit and plastic limit) and specific gravity determination were performed to classify and identify the type of soil used in this study. The sieve analysis test was carried out to evaluate the percentages of different grain sizes contained within the soil sample. The liquid limit value was determined by the cone penetration method. The soil was classified according to the Unified Soil Classification System (USCS). The specific gravity of soil was determined by using a pycnometer bottle method. Standard proctor compaction test and CBR test are the main testing in this study, which were performed to evaluate the influence of CF on the properties of reinforced soil. The optimum CF was determined based on the load bearing ratio of the CF–soil composition.

Standard Proctor Compaction Test

Standard proctor compaction tests were conducted to obtain the OMC of reinforced soil, the point at which a soil becomes dense and achieves its MDD and subsequently be used to prepare samples for CBR test. Compacted samples were prepared by inserting 0% (control sample, without CF), 0.5%, and 1% of CF randomly into the soil sample by passing it through a 4.75 mm sieve. These CF ratio has been selected on the basis of the pilot test.

The CF must first combine with the soil before the water is applied. The soil sample was initially compacted using 12% water content. Standard proctor tests were conducted in the laboratory by using a cylindrical mould of 100 mm diameter and 127.0 mm height fitted with detachable base and collar with 50 mm height. The soil was compacted in three layers, each receiving 27 blows from a 2.5 kg-weight rammer falling from a 310 mm height. Before placing a new layer, the surface of the previous layer was scratched using a spatula to ensure a uniform distribution of the compaction effect. After the compaction was completed, the

collar was removed and the surface was trimmed using a straight edge. This process was repeated by successively adding 3% water content until the soil weight was reduced. The process was then repeated for five to six times, and the dry densities were determined for each increase in moisture content. The moisture contents were taken at top middle and bottom of the soil sample.

California Bearing Ratio (CBR) Test

CBR tests were performed on unreinforced soil (0% fibre content) and reinforced soil in accordance to BS 1377: Part 4: 1990 (Clause 7). All CBR tests were conducted in unsoaked condition on oven-dried soil at 105 °C which had been passed through No. 4 sieve (4.27 mm). Four kilograms of dry soil was used to prepare the CBR sample and mixed with different percentages of CF at its OMC, which was obtained in the compaction test. The soil sample was then compacted into a 150 mm diameter and 127 mm height cylindrical mould using an automatic compaction machine in three equal layers, each receiving 62 blows from a 2.5 kg-weight rammer. After compaction, the collar was removed, and the surface was trimmed using straight edge. The load applied to penetrate the soil surface and the penetration load value were recorded to the rate of 1.27 mm/min. When the test was finished, about 20–30 g of soil sample was taken to determine the moisture content. The procedure was repeated for soil samples mixed with different percentages of CF (0.5% and 1.0% of the dry mass of soil). The CBR values of soil specimens were assessed on the basis of the higher CBR value measured at 2.5 mm and penetration of 5.0 mm. All tests were conducted in triplicate for reproducibility and accuracy.

RESULTS AND DISCUSSION

Performance of Virgin Soil

The performance of virgin soil, known as unreinforced soil, was observed through its physical and index classification properties. The grain size distribution curve in Figure 5 shows that the soil used in this study is fine-grained and falls under inorganic soil. From a further study through the Unified Soil Classification System (USCS) as illustrated in Figure 6, the soil was classified as high plasticity clay denoted as CH, having a high liquid limit (LL), plastic limit (PL), and plasticity index (PI) of 69.4%, 34.4%, and 35.0%, respectively. The liquid limit of the soil was measured at 20 mm penetration based on penetration and moisture content curve as shown in Figure 7. The physical and index classification properties of the soil are summarised in Table 2. The index properties of the soil clearly show that it should not be directly used for construction purposes as they are below than the required properties as recommended in the Standard Specification for Road Works (Malaysia Public Works Department), 2008), which are:

1) The liquid limit shall be not more than 35%,
2) The plasticity index shall be in the range 4% to 10%, and
3) The material passing the No. 200 sieve shall be not more than 50%.

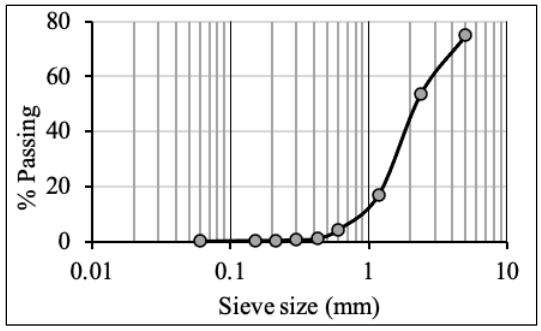


Figure 5. Grain Size Distribution Curve of Soil

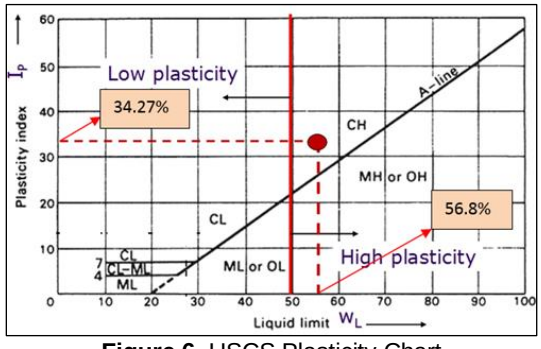


Figure 6. USCS Plasticity Chart

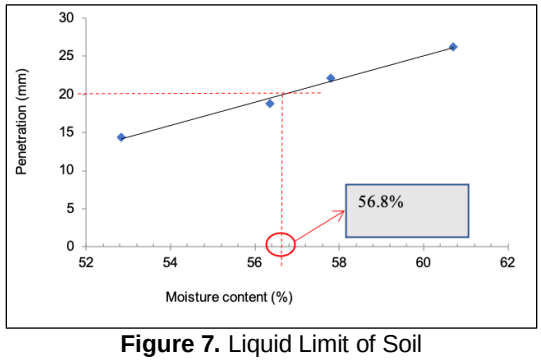


Figure 7. Liquid Limit of Soil

Table 2. Physical and Index Classification Properties of Virgin Soil

Parameter	Unit	Value
Particle size distribution:		
D_{10}	mm	0.9
D_{30}	mm	1.6
D_{60}	mm	2.9
Coefficient of uniformity, C_u	-	3.2
Coefficient of curvature, C_c	-	1.0
Soil classification (USCS)	-	CH
Specific gravity, G_s	-	2.6
Liquid limit, LL	%	69.4
Plastic limit, PL	%	34.4
Plasticity index, PI	%	35.0
W_C/LL	-	0.2

Influence of CF on Compaction Characteristic

The compaction characteristics were measured from the dry density graph to the moisture content corresponding to the maximum dry density and the optimum moisture content as shown in Figure 8. Figure 9 further shows the compaction characteristics, which are the OMC and MDD values of unreinforced (0% CF) and reinforced soil with different proportions of the CF (0.5% and 1.0%). The OMC increased and the MDD decreased with the increment of CF proportion. The OMC values were 18.2%, 20%, and 22%, at 0%, 0.5%, and 1.0% CF, respectively. The increase in the OMC with higher fibre content is because CF has a high tendency to absorb and store water. It was also clarified by Peter et al. (2016) and Sunny and Joy (2016). On the other hand, MDD values decreased from 1.82 to 1.77 g/cm^3 when the CF content was increased to 1.0%. This is because the soil content and the void within the soil particles were replaced with fibre, which has less density compared to the soil. There was little to no gain on the MDD value from the fibres. A similar trend has been reported by Ayininuola and Oladotun (2016), Peter et al. (2016), and Sunny and Joy (2016).

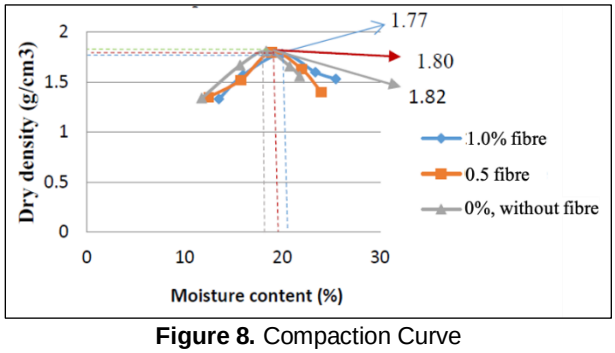


Figure 8. Compaction Curve

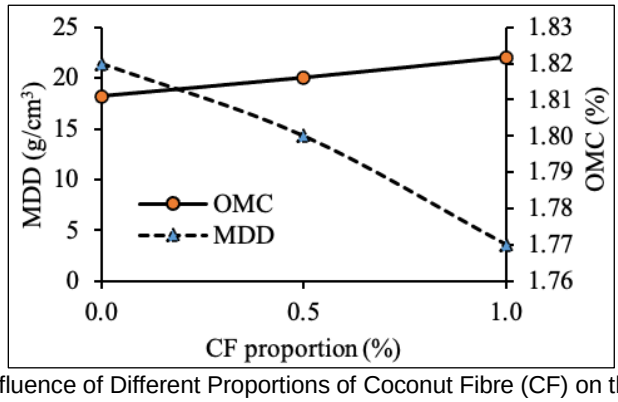


Figure 9. Influence of Different Proportions of Coconut Fibre (CF) on the OMC and MDD Values of Reinforced Soil

Influence of CF on CBR Performance of Reinforced Soil

Unsoaked CBR tests were conducted on unreinforced (without CF) and reinforced soil with different proportions of CF (0.5% and 1.0%). The sample was prepared based on the corresponding OMC value obtained from compaction test. Figure 10 indicates the relationship between the load of plunger and the penetration of randomly distributed coconut fibre-reinforced soil and unreinforced soil. The results show that the random inclusion of CF into

the soil up to 1.0% increased the resistance of the plunger of the CBR penetration and led to an increase in its CBR value. The influence of CF proportions on CBR value is further shown in Table 3 and Figure 11. Unreinforced soil had the smallest CBR value of 7.58%. The CBR values of the reinforced soil increased to 17.42% and 26.14% when CF proportions were increased to 0.5% and 1.0%, respectively. The presence of CF in the soil helps to lock soil particles together and resist the tensile strength from the plunger load. A similar behaviour was also described by Dang et al. (2017). Thus, the reinforced soil meets the CBR requirement as recommended for road sub-base laterite by the Malaysian Public Works Department (JKR), which is 20%.

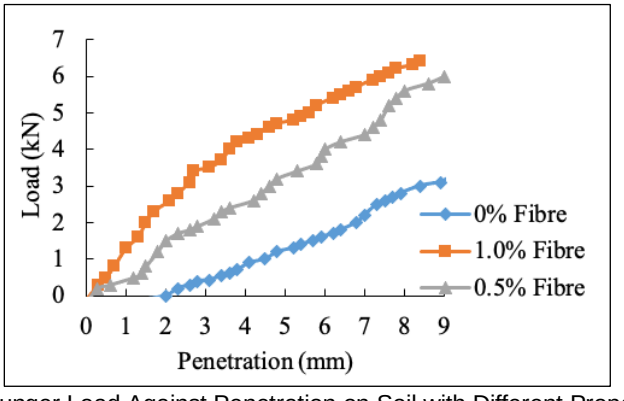


Figure 10. Plunger Load Against Penetration on Soil with Different Proportions of CF

Table 3. CBR Value of Unreinforced and Reinforced Soil Specimens at 2.5 and 5.0 mm Penetration

CF Proportion (%)	Penetration at 2.5 mm	Penetration at 5.0 mm
0	7.58	5.00
0.5	17.42	11.50
1.0	26.14	17.25

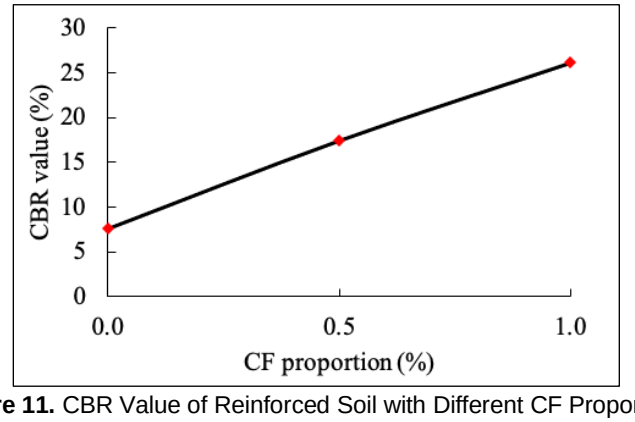


Figure 11. CBR Value of Reinforced Soil with Different CF Proportions

CONCLUSION

In the present study, the potential of using coconut fibre (CF) as a reinforcement material for improving the strength performance of problematic soil was studied. The presence of CF successfully increased the CBR value of the soil from 7.58% (0% CF) to 26.14% (1.0% CF)

with increasing OMC value and decreasing MDD value. By increasing the CBR value to more than 20%, the bearing capacity is increased and the road pavement thickness can be reduced. It can be concluded that CF has potential in improving strength performance of locally available laterite soil at Panchor–Muar, Johor. The findings obtained from this research serve as sustainable and cost-effective solutions to encountered challenges from soft yielding ground.

ACKNOWLEDGEMENT

The authors would like to express gratitude to the laboratory staff of FTK, UTHM and the industrial practitioners who shared their valuable experience with this research. This study was supported by the Universiti Tun Hussein Onn Malaysia (UTHM) under Tier 1 Grant (Vot. H221). The supports are sincerely acknowledged.

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INDUSTRY 4.0 AND CONSTRUCTION PERFORMANCE: FROM LITERATURE REVIEW TO CONCEPTUAL FRAMEWORK

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Abstract
Despite the promising future of Industry 4.0 to be integrated successfully within the construction industry, researchers, stakeholders, and project managers are still struggling with how to link Industry 4.0 with the construction industry. A review of the literature revealed that there is still a lack of research on the benefits of Industry 4.0 in the construction industry context, particularly concerning the impact on performance. Therefore, the main objective of this paper is to develop a conceptual model underpinned by the resource-based view theory and also via applying the CIMO logic. This study seeks to understand the impact of Industry 4.0 technologies as resources, and the mediating role of IT and operations capabilities considering the construction industry complex con-text through the integration challenges.

Keywords: *Conceptual framework; construction industry; IT capability; Industry 4.0; resource-based view.*

INTRODUCTION

Industry 4.0 term was first introduced by Germany as a proposal for a new concept of German economic strategy that relies on modern technologies (Mosconi, 2015). The concept has developed to express the fourth industrial revolution, which is based on multiple technologies, including cyber-physical systems (CPS), the internet of things (IoT), cloud computing, and mobile computing, among many other technologies. Industry 4.0 refers to the growing automation and digitization of the manufacturing environment (Lasi, Fettke, Kemper, Feld, & Hoffmann, 2014). Although Industry 4.0 was initially applied to the manufacturing sector, a literature review has shown that many industries, apart from manufacturing, have benefited from Industry 4.0. For instant, the automotive industry benefits from Industry 4.0 by designing cars with 40% controlled by electronics (Wollschlaeger, Sauter et al., 2017). Also, the financial industry benefits from Industry 4.0 by the integration of blockchain, which reduced cost (Philbeck and Davis, 2019). Similarly, the healthcare industry benefits from Industry 4.0 by the sequencing of genes and development of new diagnostical procedures(da Silveira, Neto et al., 2019). Fundementaly, Industry 4.0 is slowly but strongly influencing and changing the construction sector (Dallasega et al., 2018).

However, the construction industry, which is one of the significant contributors to the countries' gross domestic product (GDP), has not managed to integrate Industry 4.0 technologies (Oesterreich & Teuteberg, 2016). The construction sector has a more complex environment compared to the manufacturing sector. For an instant, having a large number of processes, sub-processes, and participants acting at different stages and locations increases the complexity of the construction industry (Arayici & Coates, 2012). Osunsanmi, Aigbavboa, and Oke (2018a) asserted that the evaluation of the construction into a fully digital sector is difficult due to the segmented nature, site-based activity, and resistance to change of the construction industry.

Moreover, construction project are dynamic structures that could not be conceived as serial goods (Kohler et al., 2014). Every building is planned and constructed in compliance with unique requirements and stakeholder decisions, making Industry 4.0 more complex to implement than other industries For instance, Building Information Modeling (BIM) is becoming the center of design, engineering and construction information management that enables all stakeholders to share and manage information during the life cycle of a construction project. Yet the pace of BIM adoption in the construction sector was much slower than anticipated (Hossain & Nadeem, 2019).

Many studies have indicated that Industry 4.0 could create positive impacts and improve construction performance and competitiveness (Hossain & Nadeem, 2019; Oesterreich & Teuteberg, 2016; Osunsanmi et al., 2018a). Although empirical studies on Industry 4.0 are in abundance in the construction literature, there is nevertheless inadequately empirical evidence relating to how information and communication technologie (ICT) improves construction performance. Therefore, there is a need to develop a comprehensive framework for analyzing the affects of internal and external factors as modertaors in the relationship between Industry 4.0 and performance. Against this background, the main objective of this paper then is to propose a conceptual model for the Industry 4.0 integration within the construction industry by applying the Resource-Based View (RBV) theory and CIMO logic.

LITERATURE REVIEW

Industry 4.0 Overview

Industry 4.0 is a term/concept used to relate to the idea of the fourth industrial revolution. The main objective of Industry 4.0 is to integrate modern technologies with the production process. One of its assumptions is that the product can be customized with the individual needs of the user, for example, individually customized sneakers, cars, or furniture. Industry 4.0 grants the possibility to produce customized products with excellent quality and reasonable prices. Smart, visualization, simulation, and digitization are the technical basis of Industry 4.0. Moreover, Industry 4.0 influence the entire life cycle of a product; development, manufacturing, use, and maintenance (Alaloul et al., 2018).

Germany proposed term Industry 4.0 in 2011, German government developed Industry 4.0 as a concept to define coherent policy framework to maintain competitiveness on the global market, then was formally put forward as a concept at the Hannover Industrial Expo in April 2013 (J. Li & Yang, 2017). Since then, the research efforts to examine Industry 4.0 technologies, applications, benefits, and challenges started.

Essentially, Rylnikova, Radchenko, and Klebanov (2017) indicated that the main components of Industry 4.0 are: cyber-physical systems (CPS), the internet of things (IoT), the internet of service (IoS), and smart units. Nevertheless, communication between machines (M2M) and smart products are not considered as a self-determining portion. They indicated that M2M is viewed as a part of IoT and smart products as a part of CPS.

Industry 4.0 and Construction Industry

The complex nature of the construction industry, site-based, large number of participants, and low IT investment capabilities, leads to difficult integration of Industry 4.0. Oesterreich and Teuteberg (2016) reported that the lack of innovation and technology implementation in the construction industry is reasoned by the low investment in research and development (R&D). However, despite this complexity, Figure 1 shows how technologies are evolved through various stages of construction through history.

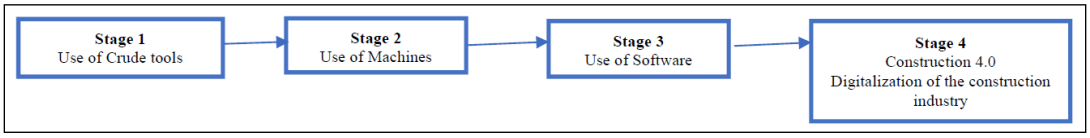


Figure 1. Evolution of Construction Technologies (Source: Osunsanmi et al., 2018a)

Recently, the term construction 4.0 has been used by researchers to indicate the integration of Industry 4.0 to the construction industry (García de Soto et al., 2019). A review of the literature shows many research efforts to examine Industry 4.0 in construction. In particular, Oesterreich and Teuteberg (2016), who were among the first to review Industry 4.0 concept, application, state of the art, state of practice, benefits, and challenges among construction companies. Subsequently, Osunsanmi et al. (2018a) and Osunsanmi et al. (2018b) provided a framework for construction 4.0 based on the results of Oesterreich and Teuteberg (2016) (Figure 2).

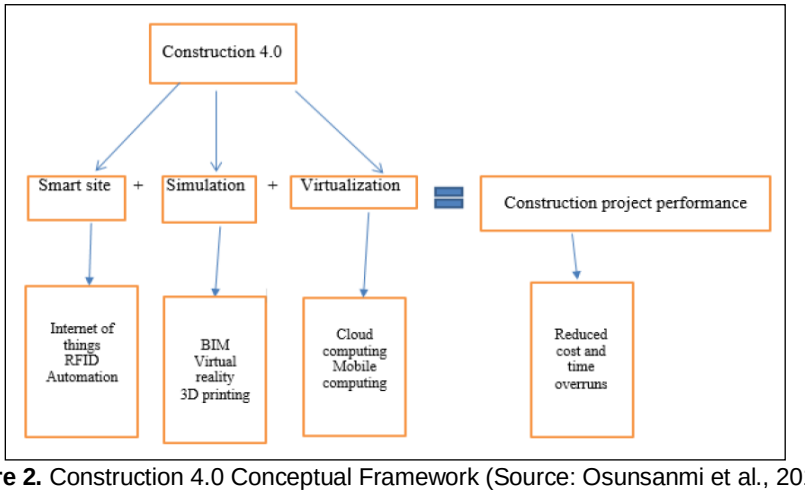


Figure 2. Construction 4.0 Conceptual Framework (Source: Osunsanmi et al., 2018a)

Oesterreich and Teuteberg (2016) were the first to establish the term Industry 4.0 to the construction literature. They conducted research to review the existing research efforts in the field of Industry 4.0 and construction. As their efforts end without results, it becomes clear that scientific publications still did not exist in this domain. A year after, Nowotarski and Paslawski (2017) conducted a structural literature review (SLR) with "Industry 4.0" and "construction" as keyword phrases, ending with only 8 publications found. Likewise, Dallasega, Rauch et al. (2018) conducted SLR with the same keywords and ended up with 39 publications. Those results implicate that in the construction idustry the idea of Industry 4.0 is randomly raised, which gave the area for further research in this subject.

In 2017, Hexagon PPM, which is a company that creates software solutions that enable the design, construction, and operation of industrial assets, raised the term "Construction 4.0" in one of its marketing seminars for an application that relays on Industry 4.0 technologies (Hexagon, 2017). Since then construction 4.0 is used to refer to counterpart of Industry 4.0 in the construction industry (Osunsanmi, Aigbavboa et al., 2018a, Osunsanmi, Aigbavboa et al., 2018b, García de Soto, Agustí-Juan et al., 2019).

Generally, construction 4.0 research mainly focuses on applications of Industry 4.0 in construction, Industry 4.0 readiness, and construction industry challenges (García de Soto et al., 2019; Maskuriy et al., 2019). However, some researchers were interested in the benefits of construction 4.0 and its role in enhancing performance. They argued that the integration of Industry 4.0 technologies with the construction industry would enhance construction project performance. Among previous studies focusing on Industry 4.0 benefits to the construction industry are the following:

Oesterreich and Teuteberg (2016) conducted a review to understand the implication of industry 4.0 for the construction industry. Using the PESTEL framework (political, economic, social, environmental, and legal) to point out the benefits of its adoption, they found its potential benefits in terms of improvements in productivity and quality. Osunsanmi et al. (2018a) research focused on Industry 4.0 adoption among the construction industry in South Africa by using a questionnaire to evaluate the awareness and readiness of construction professionals towards construction 4.0. The findings indicated that the adoption of construction 4.0 would enhance the performance regarding cost and time saving and also create sustainable buildings.

UNDERLYING THEORY AND LOGIC

Resource-Based View (RBV)

RBV theory has been highlighted in the literature to link the firm's resources with performance. RBV theory was introduced by Werenerfelt in 1984, stating that the competitive advantage/ performance of any organization relies mainly on the use and adoption of tangible and intangible resources (Miller, 2019). Tangible resources are physical assets such as equipment and machines, while intangible resources are the ones with no physical presence such as skills and reputation. Consequently, the company chooses their strategies based on their resources (Barney & Hesterly, 2012). Competitive advantages were defined as having more economic value than competitors in the product market (Miller, 2019).

The main argument of the RBV theory is that firm/organization holds resources to achieve competitive advantage and enhance performance (Barney & Arikan, 2001). RBV assumes that resources should be economically valuable, relatively scare, difficult to imitate, and immobile across companies; to be a valuable resource. RVB has three primary constructs; firm performance, resources, and capabilities, where the dependent construct is firm performance. RBV argues that a firm's performance can be enhanced by the proper use of resources (Miller, 2019).

Daowd (2016) refers to resources as something that firms could employ to achieve its goals and capabilities as subsets of the resources. Resources were classified into four

categories: physical, financial, human, and organizational (Barney & Hesterly, 2012). Consequently, RBV theory was applied in various sectors, including human resources, marketing, IT management, and construction management (Miller, 2019).

Dynamic Capability view (DCV) was derived from RBV. This view argues that firms' resources need other factors to create a significant impact on performance. Teece, Pisano, and Shuen (1997) referred to those factors as 'Dynamic Capabilities', defined as "the firm's ability to integrate, build, and reconfigure internal and external competences to address rapidly changing environments". According to this approach, the relationship between resources and performance must be mediated by capabilities (Carrick, 2016). The rationale of that approach is that the firm's resources can enhance a specific firm's capabilities through the implementation and integration of these resources (Liang & You, 2009).

CIMO Logic

Based on Bunge's rule, "In Context, use Intervention to invoke generative Mechanisms that produces Outcome" (Denyer et al., 2008), the CIMO (Context, Intervention, Mechanism, and Outcome) logic was developed. Subsequently, based on the CIMO logic, the framework of this research was developed. The CIMO approach has been adopted recently in construction management research to evaluate the relationship between technology and performance (da Silva, Floriani, & Hein, 2018). The context of the model is the complexity in the construction environment and the challenges of Industry 4.0 integration. Industry 4.0 technologies are considered as an intervention. Outcomes of Industry 4.0 integration are sustainable competitive advantage (SCA) and performance. The mechanisms that mediate these impacts are IT and operational capabilities. Table 1 summarizes the CIMO approach implementation.

Table 1. CIMO Logic Implementation

Component	Construct in this study	Explanation
Context	Integration challenges	"The surrounding and human nature factors that influence the behavior change, such as experience, age, organizational politics, firm nature, stability, and system. Interventions are affected by four contextual layers (individual, relationships, institutional setting, and infrastructure system)" *
Interventions	Industry 4.0 technologies	"The interventions managers have at their disposal to influence behavior. Such as planning, control systems, training, leadership style. It is important to examine the nature of the intervention and how it is implemented. Also, hypotheses are carried by interventions" *
Mechanisms	IT and operations capabilities	"The mechanism that is triggered by the interventions in a certain context" *
Outcome	Performance and SCA	"The outcome of the intervention in its several aspects, such as performance improvement, cost reduction or low error rates" *

* Source: (Denyer et al., 2008)

METHODOLOGY

The research method followed in the current study is to propose the conceptual framework by signifying the impact of Industry 4.0 technologies on the firm performance through the IT firm's capabilities as a mediation variables.

Further, the current study highlighted the need to study firm performance especially in the construction sector. To build the proposed conceptual framework, the researchers considered the practical gap and also have gone through extensive literature review of the given constructs and come up with the proposed conceptual framewo which should be studied at organizational level. Moreover, the underpinning theoretical background namely, resource-based view (RBV) theory, and CIMO logic also underpinned the proposed conceptual model.

Most of the research methodology and research design was conducted following the procedures of existing literarure (Ahmed et al., 2017; Bhatti et al., 2016; Casanueva et al., 2015; Che Ghani, binti Taju Rahim, bin Zainuddin, Wan Hamzah, & Alias, 2016; Jensen, Cobbs, & Turner, 2016; Minai, Raza, bin Hashim, Zain, & Tariq, 2018; Raza, Minai, ul Haq, Ismail, & Zain, 2018; Salleh & Hussin, 2016; Silva et al., 2016; Yang et al., 2019; Zhao & Priporas, 2017).

Proposed Research Model

Currently, there are several studies relating the RBV theory to construction management research and IT use. For example, Olanipekun and Xia (2017) review found that RBV is the most used model in construction management research between 2005-2014. Razak, Othman, and Pandiyan (2015) used RBV as the underpinning theory to explain the relationship between IT and operational performance in the Malaysian construction industry. Similarly, Y. Li, Ding, and Sun (2019) and Hazarika and Zhang (2019) adopted RBV in their research in the construction management field.

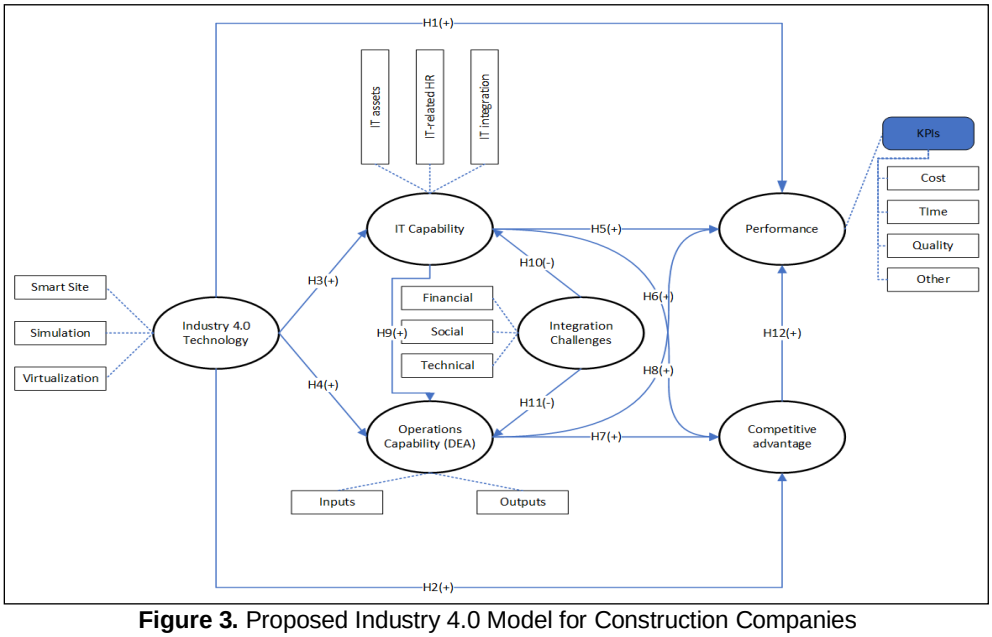


Figure 3. Proposed Industry 4.0 Model for Construction Companies

Based on the RBV, this research model is derived to understand the relationship between Industry 4.0 technologies and construction performance. This model has been adapted to suit the construction industry context, where new variables such as IT capabilities, and operations capabilities are included. The model consists of six key variables, namely: 1) Industry 4.0

technologies; 2) IT Capabilities; 3) operation capability (DEA); 4) performance; 5) competitive advantage; and 6) integration challenges. This model is shown in Figure 3.

This research identifies several constructs that are considered relevant to the research problem and construction context:

- Dependent variable: construction performance.
- Mediating variables: IT capabilities, Operations capabilities, and Competitive advantage.
- Independent variable: Industry 4.0 technologies.

The following section will describe each variable in detail.

HYPOTHESES DEVELOPMENT

To advance the literature on the impact of Industry 4.0 of construction companies, the research framework was developed, based on RBV and CIMO logic. Consequently, research hypotheses are proposed in this section. Relative studies are also provided to support hypotheses.

Impact of Industry 4.0 Technology

The integration of Industry 4.0 in the construction industry have been actively discussed by researchers(Alaloul et al., 2018; García de Soto et al., 2019; Nowotarski & Paslawski, 2017; Oesterreich & Teuteberg, 2016; Osunsanmi et al., 2018a). Even various industries have benefited from Industry 4.0 technologies, construction companies have found Industry 4.0 integration difficult or may also fail to integrate (Osunsanmi et al., 2018a). This issue could be a result of the misunderstanding of the benefits of Industry 4.0 integrations and the mechanisms in which Industry 4.0 makes an impact.

Many researchers have addressed industry 4.0 technologies' implications and benefits to construction companies. Construction companies could benefit from Industry 4.0 in terms of cost savings; labor costs are reduced through the use of robotics (Pan et al., 2018), and material costs could be reduced through the use of RFID technology to track equipment and materials (Niu et al., 2018). Moreover, construction companies could benefit from the time saving provided by Industry 4.0; construction time is reduced through the use of prefabrication and additive manufacturing (Bamana et al., 2019). Similarly, Industry 4.0 improves quality for construction companies; simulating the whole construction process and the use of BIM leads to avoid errors (Kelly & Ilozor, 2019), decision making is enhanced due to big data analytics (Dubey et al., 2019).

Likewise, Industry 4.0 implementation in construction companies leads to communication and collaboration enhancement; through the use of cloud, BIM, and social media apps among the high amount of project participants (Ahmad et al., 2019; Chen et al., 2019). Moreover, Alizadehsalehi et al. (2019) indicated that the satisfaction of project stakeholders would be improved through the application of VR, AR, and mobile computing. Notably, the performance indicators are impacted by Industry 4.0 technologies. Hence the following hypothesis was proposed.

H1: Industry 4.0 Technologies Have a Positive Impact on Firm Performance

The RBV theory suggests that integrating Industry 4.0 as physical resources enhances the competitive advantage of the construction companies (Barney & Hesterly, 2012). In addition to the economic impact by increasing productivity, performance, and efficiency, the integration of Industry 4.0 in construction companies can improve safety and sustainability, which leads to enhancing the image of the construction industry in the long run. The implementation of Industry 4.0 technologies transfers the construction industry to a technology-driven industry that can keep up with other sectors in terms of competitive advantage (Oesterreich & Teuteberg, 2016). Hence the following hypotheses were proposed.

H1a: Sustainable Competitive Advantage Mediates the Relationship Between Industry 4.0 and Performance

H2: Industry 4.0 Technologies Have a Positive Impact on Sustained Competitive Advantage

Bipat and Sneller (2015) conducted a review to conceptualize IT capabilities' constructs. In their research, IT technical resources were one of the IT capabilities defined construct, which supports (Bharadwaj, 2000) classification. Similarly, Ross, Beath, and Goodhue (1996) emphasized the role of IT resources in their explanation of IT capabilities (Zhao & Priporas, 2017). Hence, and based on RBV and DCV, the following hypotheses were proposed.

H1b: IT Capabilities Mediates the Relationship Between Industry 4.0 and Performance

H3: Industry 4.0 Technologies Have a Positive Impact on the Firm's IT Capabilities

Operations capability is a critical dynamic capability in both RBV and DCV (Yu et al., 2018). Data envelopment analysis (DEA) is applied to assess the relative efficiency of decision-making units (DMU). DMU is defined as the organization responsible for converting inputs (i.e., people, cash, resources) into outputs (i.e., satisfaction, revenues, performance metrics) and whose performance is to be examined (Emrouznejad & Yang, 2018). According to RBV, firms diversify for using resources to generate synergy, that is, to minimize inputs and maximize outputs for higher performance, which could be analyzed by DEA (Jiang, Yang, Chen, & Liang, 2019). Thus, in this research, DEA will be deployed to measure operational capabilities by using Industry 4.0 as the input following Yu et al. (2018). Hence the following hypotheses were proposed.

H1c: Operational Capabilities Mediates the Relationship Between Industry 4.0 and Performance

H4: Industry 4.0 Technologies Have a Positive Impact on Operational Capabilities (DEA)

Impact of IT Capabilities

IT capabilities, defined as the firm's ability to integrate and manage IT, serve as a source of SCA when combined with other resources and capabilities (Bharadwaj, 2000). Many scholars have discussed IT capabilities and its constructs (Zhao & Priporas, 2017). Following Ross et al. (1996) research, IT capabilities were classified into three categories; IT assets, IT-related human resources, and IT integration.

IT assets are defined as hardware and software resources that are possessed by the firm. Moreover, since IT assets do not function by itself, IT-related human resources are defined as technical and managerial skills needed to build and maintain IT applications using the available technology. Similarly, without a system and infrastructure that supports IT integration, IT will not be distinctive enough to create SCA (Zhao & Priporas, 2017). Hence, by considering IT capabilities as a combination of tangible (hardware, software, and infrastructure) and intangible (Skills, knowledge, and system) resources, and by applying the RBV, the following hypotheses were proposed.

H5: IT Capabilities Have a Positive Impact on Firm Performance
H6: IT Capabilities Have a Positive Impact on Sustained Competitive Advantage

Impact of Operations Capabilities

Operations capabilities are defined as the synergy of tasks a firm deploys to enhance its output (service or product) by efficiently using the available technology and process (Brandon-Jones et al., 2016). Consequently, RBV and DCV asserted that operations capabilities could achieve an SCA through the efficient management of knowledge and IT (Yu et al., 2018). Additionally, as mentioned, DEA is used to assess the relative efficiency of organizations (Emrouznejad & Yang, 2018). In recent efforts, researchers are using DEA combined with theories to explore the organization's operations capabilities (Tsai & Chang, 2018). However, the level of integration related to establishing relative efficiency can be identified by DEA; then, different strategies could be compared to improve efficiency (Cioaca et al., 2017). Hence the following hypotheses were proposed.

H7: Operations Capabilities (DEA) Have a Positive Impact on SCA
H8: Operations Capabilities (DEA) Have a Positive Impact on Firm Performance

Bharadwaj (2000), Bipat & Sneller (2015), Salleh & Hussin (2016), and Zhao & Priporas (2017) indicated that IT capabilities should be combined with other resources and capabilities to enhance the performance and create an SCA. Hence the following hypothesis was proposed.

H9: Operational Capabilities Mediate the Relationship Between IT Capabilities and Firms Performance

Impact of Integration Challenges

There are financial, technical, and social challenges that must be considered before the transformation of Industry 4.0 (such as the implementation cost, the organization changes,

technology acceptance, data security) (Oesterreich & Teuteberg, 2016). The complex nature and environment of the construction industry result in many challenges that slow and impact the implementation of Industry 4.0, thus lowering both IT and operational capabilities (García de Soto et al., 2019). Hence the following hypotheses were proposed.

H10: Integration Challenges Have a Negative Impact on IT Capabilities
H11: Integration Challenges Have a Negative Impact on Operational Capabilities

Impact of Sustained Competitive Advantage

RBV theory significantly explains how sustained competitive advantage can enhance performance (Barney & Hesterly, 2012). Most of the studies applying RVB have investigated the impact of sustained competitive advantage on a firm's performance (Piboonrungroj, 2012).

In a recent effort, Oyewobi, Windapo, Jimoh, and J.O.B. (2019) concluded that technological resources in construction companies do not have a significant direct impact on companies' performance. On the other hand, when technical resources are mediated by competitive strategy, a significant relationship with performance was indicated.

Hence the following hypothesis was proposed.

H12: Sustained Competitive Advantage Has a Positive Impact on Firm Performance

Table 2. Summary of the Developed Hypotheses

No	Independent	Dependent	Impact	Hypothesis
1	Industry 4.0	Firm performance	Positive	H1: Industry 4.0 technologies have a positive impact on firm performance
2	Industry 4.0	SCA	Positive	H2: Industry 4.0 technologies have a positive impact on Sustained Competitive Advantage.
3	Industry 4.0	IT capabilities	Positive	H3: Industry 4.0 technologies have a positive impact on the firm's IT capabilities.
4	Industry 4.0	Opereations capabilities	Positive	H4: Industry 4.0 technologies have a positive impact on Operational Capabilities (DEA).
5	IT capabilities	Firm performance	Positive	H5: IT Capabilities have a positive impact on firm performance.
6	IT capabilities	SCA	Positive	H6: IT Capabilities have a positive impact on Sustained Competitive Advantage.
7	Operations capablilities	SCA	Positive	H7: Operations Capabilities (DEA) have a positive impact on SCA.
8	Operations capabilities	Firm performance	Positive	H8: Operations Capabilities (DEA) have a positive impact on firm performance.
9	Operations capabilities	IT capabilities and firm performance	Positive	H9: Operational capabilities mediate the relationship between IT capabilities and firms performance
10	Integration challenges	IT capabilities	Negative	H10: Integration Challenges have a negative impact on IT capabilities.
11	Integration challenges	Operations capabilities	Negative	H11: Integration Challenges have a negative impact on operational capabilities.
12	SCA	Firm performance	Positive	H12: Sustained Competitive Advantage has a positive impact on Firm performance.

Table 2 summarizes the developed hypothese. Recalling the purpose of the current study was to propose the conceptual model by signifying the impact of Industry 4.0 technologies on the construction firm's performance through the operations and IT capabilities as a mediators.

CONCLUSION

The underlying philosophy of the proposed conceptual framewrork was to identify the adoption and implemenataion of the proposed construct in the firm and to measure them at organizational level. Furthermore, the proposed framework works as a basic step to identify the affecting strategic factors and how construction management deals with them. For instance, thefirma capabilitis depicts the internal factors, while the integration challahenges indicates the external factors.

This paper has a favorable implication both for academics as well as the construction industry professionals, which were resulting from a robust theoretical framework, which is underpinned by the RBV theory and CIMO logic. This model is multi-dimensional, which utilizes resources and capabilities that affect Industry 4.0 impacts on construction performance. For construction professionals, the findings will increase their awareness, knowledge, and strategies to achieve a proper integration of Industry 4.0 technologies in their companies.

The proposed model has both practical and theoretical implications. For researchers, The framework proposed the types of variables to be used in future empirical tests of the relationship between Industry 4.0 and firm performance, especially in the construction industry. Consequently, the model expands understanding of the moderating significant of firms capabilities such as IT and operations capabilities on the relationship between Industry 4.0 and fim performance.

At the other hand, practitioners may use the framework to refine their thought on the Industry 4.0 and the strategic tools of their firms. This paper's key contribution was to be considered as a benchmark to understand the relationship with respect to Industry 4.0, firm's capabilities, SCA, and perfomance.

Research hypotheses are developed based on the suggested model and previous studies. The next step of this study is to develop a survey that will be applied in Malaysia's construction industry for pilot study data collection. We 're going to try to do research using RBV approaches. Therefore, both researchers and practitioners will have more empirical evidence associated with the fundamental theory behind the RBV, thereby enhancing understanding of Industry 4.0 relationships and firm performance as well as the moderating effects of firm's capabilities.

ACKNOWLEDGMENT

The corresponding author would like to express very profound gratitude to Prof Tahseen Mansour; for providing him with unfailing support and continuous encouragement throughout his years of study, and also through the process of research and writing this article.

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STAKEHOLDER'S PERCEPTIONS ON THE SIGNIFICANT FACTORS AFFECTING SAFETY MANAGEMENT IMPLEMENTATION AT CONSTRUCTION SITES

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Abstract
One of the factors of high number of accidents in construction industry is due to the lack of safety management that is closely related to the poor safety awareness among stakeholders. Previous studies had emphasized the role of stakeholders in affecting entire project performance. This paper aims to investigate the significant factors affecting safety management implementation at construction sites based on the perceptions of different construction stakeholders. A total of 112 questionnaires was received from respondents of various construction stakeholders such as consultants and contractors who are working at construction sites in Johor Bahru. The factors affecting safety performance that formed the questionnaire are commonly accepted physical safety management performance factors that were derived from previous studies involving accident rates and the satisfaction of construction stakeholders. An average mean index analysis was conducted, and the result indicated that proper handling of equipment, proper installation and dismantling of heavy devices, and site inspection and supervision conducted by supervisors closely contribute to the factors affecting safety management at the construction sites. Further analysis on the similarity/dissimilarity perceptions on the factors affecting safety management at construction sites for within (Intraclass-Correlation Coefficient (ICC)) and between (Kruskal-Wallis) groups of respondents were analysed. The findings of the present study revealed a high similarity of OSH risk perception within and among stakeholder groups. This paper offers an important contribution by providing significant information that benefits the construction stakeholders to further facilitate the benchmarking process and various initiatives that can improve the safety performance of construction industry as a whole.

Keywords: *Construction; safety performance; safety management; safety performance factor*.

INTRODUCTION

Construction is a major industry that has become the main focus today but remains plagued with constant challenges, such as accidents and death. Laws and regulations have been introduced to establish a good quality of safety management in many countries (Mehta and Agnew, 2010). However, the number of accidents is still high in construction industry, which has resulted in negative impacts in terms of reputation and safety performance.

Various construction projects are being undertaken at an increasing rate and complexity, which influence workplace safety management. In 2000–2009, the Social Security Organisation (SOCSO) reported a total of 42,775 accidents in the construction industry in Malaysia, with an average annual mean incident of 19.3 per 100,000 by sector (Ganesh and Krishnan, 2016). The most critical causes of accidents in the construction industry are falls, specifically being hit by falling or moving objects (17%) and falling from height (28%) (Ganesh & Krishnan, 2016).

The construction sector has earned a negative reputation due to its high accident rates. The rates of fatal and non-fatal injuries and illnesses are relatively high and have not dropped significantly during the past 10 years (Abudayyeh, Frederick, Butt & Shaar, 2006). These problems in safety management are affected by the 'poor safety awareness among stakeholders, lack technical guidance and lack of supervision to monitor the reckless operations' (De Silva, Rajakaruna & Bandara, 2008).

Stakeholder management has a considerable influence on construction projects (Loosemore, 2006). Ng, Cheng & Skitmore (2005) stated that it is an extra advantage for stakeholders to involve in the small-scale construction projects, which require low-quality standards, low cost and minimum time, thereby regarding safety as the lowest priority. The factors listed in this research are human, equipment, management and technical factors, which strongly prove that stakeholders have a poor record in safety management.

Factors affecting safety management have been researched by Kanachana and Karunasena (2018), Yiu, Sze and Chan (2018), Zhao, Kazemi, Liu and Zhang (2018) and Tam, Zeng and Deng (2004), but their studies were conducted in countries other than Malaysia. Several studies have been published in the form of journals, theses, books and online data indicating the safety factors involved. In Malaysia, there is a lack of research in this field, and the perceptions of construction stakeholders have not been examined despite the several case studies that have been made.

Stakeholders substantially influence project complexity and interactions. According to Salancik and Pfeffer (1974), stakeholders are a major factor affecting the entirety of project performance. This statement was underlined by Yang and Shen (2014): stakeholders have the power of compromise and adaption in project strategies. Project outcomes can indicate the influence of the stakeholders in project making (Aaltonen & Sivonen, 2009) and the project phases (Herazo & Lizarralde, 2016).

Therefore, this study intends to investigate the significant factors affecting safety management at construction sites from construction stakeholders' perspectives. Furthermore, the collected data were examined for any similarity in perception about these factors.

LITERATURE REVIEW

Construction Safety Management

Construction safety management deals with the actions of the management in all organisational levels to create a structured organisation platform with which employees are going to be trained and motivated to perform safe and productive construction work (Hassan et al., 2009; Levitt & Samelson, 1993). In addition, safety management is an effective system of removing and minimising the forces that injure workers and damage facilities and equipment at construction sites. Therefore, safety management is an organisational function that identifies safety risks to ensure safety (Jokkaw & Tongthong, 2016).

Project site safety is related to the process of safety management in terms of controlling safety policies, procedures and practices. According to Cheng, Li, Fang and Xie (2004), the desired goals can be achieved through a dynamic process. However, Ming (1994) concluded

that safety, health and environments should first be managed in all aspects of a construction business to manage safety management systems, such as policies, organisations, management controls and resources.

Moreover, the actions of engineers, contractors and subcontractors influence jobs at construction sites. Among these important personnel, engineers should be the most ethical; they should prioritise safety and health at construction sites. According to Hinze and Raboud (1988), the intervention of top management can help reduce accident rates. This finding supports the view of Heinrich's Domino Theory, in that 98% of accidents can be prevented through the involvement of safety management. Heinrich's theory can be the platform on which monitoring systems can be started and feedback on effective safety management can be provided (Chi & Han, 2013).

Contractors and subcontractors in the construction industry tend to ignore the safety management requirements stated in standards produced by OSHA and legislations (Bakri, Zin, Misnan & Hakim, 2006). This is supported by Abdullah and Chai (2010) who suggested that it was the employer's negligence that was responsible for the occurrence of accidents within the construction sector. According to Lingard and Rowlinson (1997), subcontractors do not prioritise the importance of worker safety and violate safety practices to ensure adherence to project schedules. Although the subcontractors in Lingard and Rowlinson's study are from small to medium sized company, some subcontractors do not implement safety management at construction sites to stay within the budget of the contracting company. In addition, they also found that subcontractors have poor safety awareness and do not communicate sufficiently with the workers at their construction sites due to the unavailability of safety resources on these sites.

A successful safety management plan of action must be underpinned by the commitment of top management; such support is essential for reducing the accident rates at construction sites. In addition, management systems have been introduced into industrial undertakings (Bakri et al., 2006). The application of safety management systems should be clear to identify, understand and control injury hazards and reduce incidents. The implementation of safety management requires employers to conduct daily inspection to create awareness and show the advantages of safety management systems. Therefore, this study adopted the framework of Zhao et al. (2018), whose findings outlined safety management supervision under the five significant factors.

Factors Affecting Safety Management

The factors affecting safety management implementation at construction site are discussed by category.

Human Factors

Human factors include the human activity involved in the operation, production or construction of equipment and systems. According to Garrett and Teizer (2009), *'human has the strong relationship influence each other in the same work of environment situation'.* This statement is supported by the finding that 88% of accidents are caused by human factors (Heinrich & Peterson, 1931). Catastrophic accidents are caused by human factors (Bridges &

Tew, 2016). The deficiencies in these factors include the safety attitudes of workers (Hassan, Chew & Chandrakantan, 2009; Siu, Philips & Leung, 2003) and supervisors (Fang, Wu & Wu, 2015; Kanachana & Karunasena, 2018) and lack of safety training received by workers (Olutuase, 2014), effective communication and cooperation (Zin & Ismail, 2012; Zohar (2002), safety experience and skills of contractors and supervisors (Ghani, hamid, Zain, Rahim, Kamar & Rahman, 2010; Wong, Chan, Tse & Love, 2000) and site safety inspection and supervision conducted by supervisors (Fang et al., 2015; Choudry & Fang, 2008).

Equipment Factors

Equipment factors are dangerous because they involve human beings working with machines to complete tasks. According to the National Bureau of Statistics, mechanical equipment causes five major injuries to humans, namely, falling injuries, crane machine accidents, object strikes, electric shocks and collapse. The key factors that must be properly selected and used according to standard operation procedures to reduce life-threating situations are personal protective equipment (PPE) (Zin & Ismail, 2012; Guo, Li, Chan & Skitmore, 2012), proper installation and dismantling of heavy devices (Beavers, Moore, Shapira & Lyachin, 2009) and proper handling of equipment (Teo, Ling & Chong, 2005).

Environmental Factors

Environmental factors influence the performance of a project and involve the lack of good knowledge about successful safety management. Akanni, Oke and Akpomiemie (2014) stated that the environmental factors in safety management should be taken into account in managing the development of construction sites to reduce unpredictability and add potential effects. This is also supported by Kartam, Flood and Koushki (2000) that adverse weather condition is one of the causes that forbid construction stakeholders to actively pursue effective ways to maximize the achievement of safety goal.

Management Factors

Management factors refer to the failure of the management in all organisational levels to promote workplace safety performance through safety management. Any organisation management that fails to promote safety management might contribute to management factors. The key elements influencing the management factors of safety management are safety meetings (Tam, Fung & Chan, 2001), safety management commitment (Michael, Evans, Jansen & Haight, 2005; Baxendale & Jones, 2000), health and safety files (Baxendale and Jones, 2000), safety regulation and plan enforcement (Baxendale & Jones, 2000; Ning, Wang & Ni, 2010; Tan and Nadeera, 2104 2014?), budget allocated for safety management (Tan & Nadeera, 2104) and safety management systems that adhere to legislations, codes and standards (Bakri et al., 2006; Kartam, Flood & Koushki, 2000).

Technical Factors

Technical factors refer to the improper handling of equipment and the use of wrong techniques by workers at construction sites (Abdul Hamid, Yusuf & Singh, 2008). The key elements of the technical factors influencing safety management are safety risk identification and analysis (Malekitabar, Ardeshir, Sebt & Stouffs, 2016; Choe and Leite, 2016), first aid

training and emergency preparedness (Ali et al., 2012), complexity type of construction (Suraji et al., 2001) and behaviour-based safety techniques (Oostakhan, 2012; Tavakol & Dennick, 2011).

METHODOLOGY

The factors affecting safety management implementation were obtained through a literature review of published journals, theses, books and internet articles published from the year 2000 to 2019. Next, the obtained data that relevant were selected and collated on the basis of the identification of the factors affecting safety management implementation at construction sites. The researcher designed the questionnaire by incorporating the five factors influencing safety management at construction site, i.e. human, technical, management, environmental and equipment factors.

Next, the questionnaire was reviewed by 3 panels of expert to validate and check the content reliability of its design. The experts were selected among those who had more than 10 years experience in managing safety and health at construction sites. The questionnaire was revised several times regarding the importance of the collated attributes before being distributed to the selected stakeholders. The authors solicited responses from several construction sites throughout Johor Bahru. The data collected from the questionnaire were analysed using analyses of the average mean index, intragroup (intraclass correlation coefficient) and intergroup comparison (Kruskal–Wallis) to measure the five factors that are significantly related to construction safety management. Conclusions and recommendations were made on the basis of the analysis findings.

Questionnaire Development

The data on the perceptions of multiple stakeholders at construction sites were collected through questionnaires to enable an objective and cost-effective investigation. First, a comprehensive list of safety factors was screened through a pilot study to ensure that all items in the questionnaire were valid, reliable and significant. . The questionnaire contained two sections. The first section explored the backgrounds of the respondents, and the second section consisted of the 23 safety factors listed in Table 1. These are the factors that could affect safety management at construction sites. The questionnaire adopted Likert scales ranging from 1 (absolutely disagree) to 5 (absolutely agree).

Table 1. Factors Affecting Safety Management

Factors	Item	Key Items
Human Factors	HF1	Safety attitude of workers
	HF2	Safety training received by workers
	HF3	Safety experience and skills of contractors and supervisors
	HF4	Safety attitude of supervisors
	HF5	Site safety inspection and supervision conducted by supervisors
	HF6	Effective communication and cooperation
Equipment Factors	EQF1	PPE
	EQF2	Proper installation and dismantling of heavy devices
	EQF3	Proper handling of equipment
Environmental Factors	ENF1	Complexity of geology and hydrology
	ENF2	Frequency of adverse weather

Factors	Item	Key Items
Management Factors	MF1	Safety meetings
	MF2	Safety management commitment
	MF3	Health and safety files
	MF4	Safety regulation and plan enforcement
	MF5	Safety and health awareness
	MF6	Budget allocated for safety management
	MF7	Safety management system with adherence to legislations, codes and standards
	MF8	Safety behaviour toward training and education by top management
Technical Factors	TF1	Safety risk identification and analysis
	TF2	First aid training and emergency preparedness
	TF3	Complexity type of construction
	TF4	Behaviour-based safety techniques

Population and Sampling

The participants were the construction stakeholders who were fully involved in the construction sites in the Johor Bahru area. The respondents were labelled 'unknown population sample' due to the unknown exact number of them working in Johor. The questionnaire was answered in a voluntary and anonymous manner with the strong support and coordination of the project owners. In total, 120 questionnaires were distributed, and 112 valid questionnaires were retrieved (93.33% response rate). The 6.3% invalid questionnaires either had incomplete answers or were not filled out by the respondents they were sent to. Figure 1 shows pie charts of the respondents' stakeholder type, designation type and working experience.

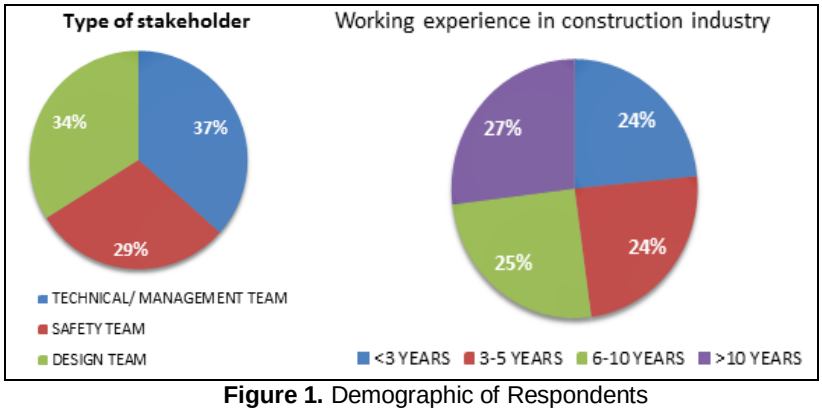


Figure 1. Demographic of Respondents

Measures for Data Analysis

After the data collection, non-parametric methods were used to analyse the safety factors affecting safety management implementation at construction sites. The data were analysed with spreadsheets to estimate the mean value for the factors. The software Statistical Package for the Social Sciences (SPSS) was utilised for the following techniques: (1) reliability test, (2) intragroup comparison using the intraclass correlation coefficient and (3) intergroup comparison using the Kruskal–Wallis test.

Instrument for Reliability Test

A reliability test was conducted to check the reliability of the results and the overall consistency of the entire survey. Cronbach's alpha was used to measure the instrument's reliability (i.e. internal consistency). Factor analysis theory (Tavakol & Dennick, 2011) indicates that an alpha value greater than 0.6 shows acceptable reliability. According to Koo and Li (2016), good internal consistency is expressed by an alpha value of at least 0.70.

Average Index Analysis

Average index analysis was used to compute the mean scores of the significant factors affecting safety management. The rating groups of the average indices that represented the appropriate classification suggested by Majid & McCafer (1997) are shown in Table 2.

Table 2. Average Index Rating Group

Likert Scale	Score Value
Strongly Disagree	1.00 ≤ Average Value ≤ 1.50
Disagree	1.50 ≤ Average Value ≤ 2.50
Neutral	2.50 ≤ Average Value ≤ 3.50
Agree	3.50 ≤ Average Value ≤ 4.50
Strongly Agree	4.50 ≤ Average Value ≤ 5.00

Intragroup Comparison

An intragroup comparison was performed using the ICC test to identify any similarity or difference within the groups regarding the safety factor in safety management. Specifically, intraclass correlation measures the reliability of scales or measurements for clusters of data collected or sorted into groups. As stated by Koo and Li (2016), an ICC less than 0.5 signifies low similarity among the same group, 0.5 to 0.75 indicates moderate similarity, 0.75 to 0.9 indicates high similarity, and 0.9 indicates the most excellent similarity among the same group.

Inter-Group Comparison

The purpose of inter-group comparison was to check whether there are similarities or differences between and among the groups of stakeholders regarding the risk perceptions related to all of the cases. The Kruskal-Wallis test was employed in the present study with the aim of determining whether all of the four stakeholder groups as a whole had any disagreements on risk perceptions. Next, the post-hoc test (Anova 1-way test) was carried out to test the pairwise comparisons between stakeholder groups regarding any significant discordance of risk perception regarding each scenario.

RESULTS AND DISCUSSION

The data were analysed using Microsoft Excel and SPSS. The outcomes from the software provided the results for the reliability test, average mean index and intragroup and intergroup comparisons, which are presented here in the form of tables.

Reliability of the Survey Instrument

A reliability test was done to ensure good internal consistency. The stakeholders involved in this reliability test were technical/management, safety and design teams involved in the construction sites. Table 3 shows the results of the reliability test for all groups of respondents, which is 0.952. Therefore, the degree of agreement is uniformly reliable (high reliability) (Cronbach & Shavelson, 2004, Koo & Li, 2016).

Table 3. Reliability Test Results

Groups	Cronbach's Alpha	Number of item	Number of particpants
Technical/ Management team Safety team Design team	0.952	23	112

Average Index Analysis of the Survey Results

An average index analysis was performed to determine the significant factors affecting safety management that were gathered through the given Likert scale. The results in Table 3 show that the design team has a higher mean rating score compared with the safety team. This finding is in line with Zhao, McCoy, Kleiner, Mills and Lingard (2016), who stated that the *'design team have lack of knowledge about hazards awareness involved at the construction site'.* Moreover, Ning et al. (2010) postulated that design teams have little responsibility on the on-site practice and management, hence unconvinced about the factors. Meanwhile, the safety team has the lowest mean rating score, probably because its members have a good understanding of their responsibilities and ability to control the crucial safety factors occurring at their construction sites. This result is in line with that of Zhao et al. (2016), who stated that whoever controls the working environment directly influences the perception of existing factors.

Table 4. Mean Values Indicated by Groups of Construction Stakeholders

Factor ID	Technical/ Management Team	Safety Team	Design Team	Mean for All Groups
HF1	4.22	4.12*	4.26**	4.20
HF2	4.24	4.15*	4.32**	4.25
HF3	4.29	4.18*	4.34**	4.28
HF4	4.15*	4.36	4.37**	4.30
HF5	4.17*	4.42	4.32**	4.31
HF6	4.37**	4.21*	4.26	4.29
EQF1	4.32	4.15*	4.45**	4.30
EQF2	4.32	4.21*	4.45**	4.31
EQF3	4.27*	4.27*	4.47**	4.32
ENF1	3.76	3.55*	3.84**	3.72
ENF2	3.73*	3.73*	3.82**	3.74
MF1	3.85*	4.03	4.13**	4.02
MF2	4.12*	4.18	4.24**	4.19
MF3	3.88	3.85*	3.97**	3.91
MF4	4.15*	4.18	4.24**	4.19
MF5	4.24	4.15	4.11*	4.18
MF6	4.07	3.97*	4.21**	4.07

Factor ID	Technical/ Management Team	Safety Team	Design Team	Mean for All Groups
MF7	4.12*	4.24	4.42**	4.28
MF8	4.24	4.03	4.32**	4.21
TF1	4.12*	4.12*	4.16**	4.15
TF2	4.20	4.09*	4.32**	4.23
TF3	4.12**	4.00*	4.11	4.10
TF4	4.05	3.91*	4.16**	4.04

**Indicates lowest mean perception among the teams.*
***Indicates highest mean perception among the teams.*

From Table 4, it could be seen that the most significant factor affecting safety management implementation at construction site is EQF3 (proper handling of equipment), followed by HF5, EQF2, HF6, HF3, MF7 and TF2. Meanwhile, the least factors perceived by the stakeholders are ENF1, ENF2 and MF3. Design team tend to perceive most of the factors (20 out of 23 factors) the highest compared to other stakeholder groups, whilst safety team tend to perceive most of the factors (15 out of 23 factors) the lowest compared to other groups. The findings also suggested that the mean of design team was greater than the mean of all participants for all factors, except HF6 (effective communication and cooperation). Meanwhile, the perceived factors of safety team was lower than the mean of all participants for all factors.

It is interesting to note that all stakeholder groups tend to perceive 2 key items under 'Equipment' factor, which are EQF2 (proper installation and dismantling of heavy devices) and EQF3 (proper handling of equipment), among the top 5 factors that influencing safety management implementation at construction sites. According to Teo et al.(2005), proper handling of equipment and other equipment-related factors are related to the process of carrying out works by workers that may eventually be harmful to their safety and health. Some organisations and individuals had committed to unsafe act unknowingly while carrying out their tasks (Dedobbeler & Beland (1991), which may result in the difficulty in implementing safety management at construction site. Therefore, all plant, tools and equipment should be handled, maintained and repaired by qualified personnel (Teo et al., 2005).

Intragroup Comparison

The pairwise intragroup comparison among the technical/management, safety and design teams was analysed using the ICC. Table 5 shows the results. The safety team has the highest ICC regarding the factors affecting safety management (ICC = 0.958, $p < 0.001$). The ICC of the technical/management team is 0.943, and the lowest ICC among these groups is that of the design team, which is 0.940. Although the design team has the lowest value of ICC, its value still indicates an excellent similarity (the value of the ICC demonstrates homogeneity within a group). These results indicate a high mean perception among the teams.

Table 5. Mean Values Indicated by Groups of Construction Stakeholders

Groups	ICC	Significance Value
Technical / Management Team	0.943	< 0.001
Safety Team	0.958	< 0.001
Design Team	0.940	< 0.001

Intergroup Comparison

The intergroup comparison was analysed using the Kruskal–Wallis test to measure the disagreement (if any) among these three groups of construction stakeholders as a whole regarding the significant factors. The results of the pairwise intergroup comparison from the Kruskal–Wallis test demonstrate that all of the stakeholders perceive the factors homogenously ($p < 0.001$). Meanwhile, a post hoc test (one-way ANOVA) was not needed because no contradiction existed between the groups.

CONCLUSION

The current research presents an experimental study that investigated construction stakeholders' (technical/management, safety and design teams) perceptions of the factors affecting safety management at construction sites. Twenty-three safety factors (human, equipment, environment, management and technical factors) were investigated, and the similarities and differences in the stakeholder groups' perceptions were measured. The design team had the highest mean score among all groups.

Furthermore, all of the stakeholders produced high, significant values (more than 0.005), indicating that all of them perceive the factors homogeneously. Meanwhile, the SPSS findings revealed that the design team was likely to give higher ratings compared with the other groups of stakeholders because, as stated by Zhao et al. (2016), *'design team focusing point is to ensure the satisfactory of building performance in their design and calculation often lead to uncertainties'.*

The safety team had the lowest mean rating score, possibly because its members had a good understanding of their responsibilities and ability to control the crucial safety factors occurring at their construction sites. This is in line with Krallis and Csontos (2006), who stated that *'who control over the working environment will directly influence to perception of the existing factors'.*

The present study contributes to the existing body of knowledge in construction safety by providing evidence of the similarities and differences in stakeholders' perceptions, particularly those who are site-based. The method employed in this study refers to previous works but provides a new approach to investigating the factors affecting safety management at construction sites.

In view of this, future research can be improved by detailing out the reason why each of the stakeholder groups chose the provided rating for each factor. The goal is to identify significant factors precisely, thus enhancing the safety management factors. Finally, future studies should widen the research scope and increase the number of respondents by covering different states in Malaysia.

ACKNOWLEDGEMENT

The authors would like to thank the Ministry of Education Malaysia for supporting this research under Fundamental Research Grant Scheme Vot No. FRGS/1/2016/SS03/UTHM/03/2 and partially sponsored by University Tun Hussein Onn

Malaysia. The author would also like to thank all industry individuals who participated in the survey.

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CONCEPTUAL AND PROTOTYPE DEVELOPMENT OF A COMPOSITE, LIGHT-WEIGHT AND SUSTAINABLE FILL MAT TO REDUCE DIFFERENTIAL SETTLEMENT

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Abstract
In civil engineering, the lightweight construction technology is adopted to describe particular methods for increasing economic and structural efficiency in reducing the weight of building structural constructions together with costs and construction lead time with the use of special construction materials. Highway 'failure' occurs due to settlement of subgrade affected by dead load, and not necessarily live load, which is momentary and has less effect on subgrade settlement, the awareness to use lightweight fill technologies as a means to develop more sustainable highways is crucial. Suitable for soft soil such as clay that tends to settle unevenly due to its geological history or in man-made situations such as back filled inactive mining sites. This paper discusses the conceptual development and the laboratory testing of a prototype which would be a composite recyclable material that would meet the need of a strong, stiff, permeable, lightweight mat as an alternative fill material. The combination of polyurethane and waste plastic bottles meet the criteria of such an alternative fill. The paper discusses the compressive strength of both materials separately as well as its composite. Other products with similar concept such as the Geofoam was used as comparison and tested. The findings show that the PUPB block with a yield strength of 130kPa has about 2.6 times higher compressive strength when compared to the Geofoam which is also the well-known lightweight field material in the industry. This implicates that the PUPB block is not only greener but also stiffer and more suitable to be used as an alternative lightweight fill to mitigate hazardous settlement of constructions built on soft soil.

Keywords: *Alternative lightweight fill; sustainable material; settlement; waste material.*

INTRODUCTION

All development projects desire uniform ground conditions that are both adequate in terms of bearing capacity and stiffness. However such sites do not exist in practice, making it inevitable to construct on less favorable soils, such as soft soils or even cavernous rocks.

A local and current case study area is in Parit Yaani at a site where the approach fill embankments to a bridge as shown in Figure 1, built by Jabatan Kerja Raya (JKR) in 2010 has often undergone settlements under the embankments where the soil largely consist of marine clay After the completion of the bridge's construction and during its liability period, the edge/slant which is supported by the reinforced concrete bridge had undergone movement due to settlement from the horizontal direction and the resulting hazardous road surface deformations have had to be repaired at a considerable road repair cost. This is because the bridge is highly used by vehicles, as it is the shortcut route from Sri Gading's Industrial area to the highway and also roads leading to adjacent states. Thus it is normally used non-stop during the day and night by normal vehicles and heavy transportations (trucks and buses). To solve the settlement problem, JKR has repaired the embankments by adding fill materials on the settled area to make it even again, however this solution has not been effective as the

replaced soil fill induces further settlement of the subgrade and the embankment becomes uneven again. The repair work has been done every 6 months ever since it was built, and has been monitored by UTHM researchers since 2015, Ganason (2016).

Thus by studying the settlement behaviour of this bridge embankment, the conceptual development of an innovative composite mat is deemed to provide a more long lasting solution for this and other cases where infrastructure is built on such soft ground conditions.



Figure 1. Parit Yaani Bridge Embankment Shows Its Settlement Pattern as Indicated by the Arrows (Picture Taken As Recent 17 December 2019)

The first function of the lightweight material is to act as weight fill replacement since the conventional fill material is much heavier in comparison and poses a problem especially for highway constructions on soft soil. Bamboo grids, bamboo cells and coir fiber reinforced polymer composites, can increase the homogeneity of load spreading in soil, just like how geotextiles function, Hegde & Sitharam. (2015) Libo, Chouw, Huang & Kasal (2016) and . In Malaysia, bamboo logs are still currently being used as a lightweight construction material in soft soils. According to Construction Research Institute of Malaysia (CREAM), Geotextile bamboo fascine mattresses have been used to enable filling over wet peat where drainage cannot be carried out. This technique was first used for reclamation of slime ponds in Kuala Lumpur, Construction Research Institute of Malaysia, (CREAM, 2015). Figure 2 shows the utilization of bamboo grid as part of a bridge repair work situated in the highway interchange of Puchong and Cyberjaya. As shown, the bamboo grid is crushed upon compaction of the upper soil layer, due to the low bearing capacity of the bamboo, thus highlighting that the lightweight material also needs sufficient bearing capacity, at least higher than compacted soil.



Figure 2. The Bamboo Logs are Crushed Due to Compaction of Soil

Other lightweight fill materials such as foamed recycled glass which due to its low strength, is more suitable to be used as for non-structural fill purposes, Arulrajah (2015) whereas processed blast furnace slag aggregates are suitable and proposed to be used as replacement fill material for sub-grade, sub-base or base materials, Australasian Slag Association (ASA, 2002).

Due to different testing procedures and parameters used to analyze the materials, it is difficult to compare the suitability of the materials to act as an alternative fill material accurately. However these researches have paved the way in the exploration of various alternative materials for highway construction. Moving forward, field test on suitable models besides laboratory experiments should be conducted for understanding its practical application and the feasibility of such products. For this research, appropriate materials are investigated in order to be included for the product development of a lightweight, composite, and cost effective lightweight fill material. There are two types of materials which would be used for the research product development as discussed in 3.1.

PRODUCT CONCEPT

To meet the requirements of a fill material which can be used for a long period in the construction industry to reduce hazardous settlement, the following elements should be considered and be present in the development of the prototype.

Mat Fill Vs Aggregates

Cellular mat structures present in the minimum, triple benefits in being lightweight, load sharing and minimising non-uniform deformation. The combination of mat structure and use of recycled waste material is a desirable development in manufacturing, Hasanah et al. (2014). The mat fills acts like a set of Lego set such as depicted in Figure 3, which can be assembled and dissembled at site according to its needs. The additional characteristics which is appealing to the mat fill concept is designs ability to create interlocking blocks. This is commonly used in brick works as well. However, since the design product for this research is to be lightweight fill, thus interlocking concept would help to strengthen the integrity of all the blocks to acts as one whole.

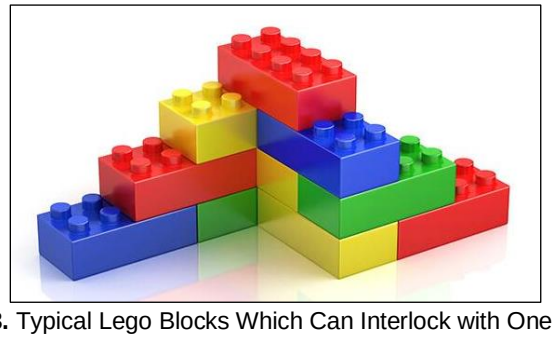


Figure 3. Typical Lego Blocks Which Can Interlock with One Another

Workability

The size and shape of the product should be determined based on easy handling during the manufacturing, storage and construction stages. Efforts to construct should be minimised without the need to use elaborate handling techniques/ machineries and easily carried out by a general construction worker. The usage of machineries would naturally increase the cost of the construction. This research which aims to mitigate settlements especially in rural/ sub urban roads which typically has one lane per direction, as shown in figure 1, aims to reduce the need for professional labour or heavy machinery to a significant extent. The lightweight blocks can be manoeuvred easily by human strength and simple standard of procedure to ensure the integrity of the layout.

Lightweight

In comparison to the density of an average backfill soil of almost 2000kg/m^3, the prototype should be as light as possible without losing its rigidity, stiffness and strength. As per the previous point, for workers to easily lift the blocks and place it during the construction process, requires it to not exceed the human capacity. In this research the lightweight blocks density would not exceed 50kg/m^3. The prototype of this research has a density of not more than 48 kg/m^3.

Cost Effective

If the cost to manufacture and apply the method is too high, construction industries would not seek to use it in their designs, thus the raw materials should be easily accessible at a significant cost in comparison with the conventional methods, and usually it would prove to be cost effective if the developed product is able to solve construction issues for a long term period. As such, this research aims to utilise waste products, usually one off usage items such as plastic bottles. Since most of the roads in Malaysia are under the jurisdiction of the State or Federal government, cost is a major consideration for contracts and designs to be considered. The end of this research aims to obtain a working patent so that any designs for actual roadworks can be then then proposed to the Jabatan Kerja Raya for consideration.

Sustainable

Civil engineering processes are both resource and fuel intensive. Sustainable methods in construction are a broad area of study and thus some fundamental concepts required in each phases of a project, critically the planning and design phase would be highlighted.

It is necessary to plot practical steps and carry out detail planning for each phase. For a sustainable approach, a detail planning is very crucial in reducing as many errors and saves time and cost in the long run towards project completion.

More than 100 million plastic bottles are used up in the world every day and only 1 out of 5 bottles end up being recycled, The World Counts 2014 Bottled Water Waste Facts (2014). Thus developing ways to put in use waste material to be used as a construction fill material reduces the need to use natural resources.

PROTOTYPE DEVELOPMENT

Primary Materials

As per concept, suitable materials needed to be identified and tested for the prototype. For the material to be light and yet stiff and rigid at the same time, the polyurethane was identified and made in composite with waste plastic bottles.

Polyurethanes (PU)

PU are usually obtained from reaction between polyol polyether or polyol polyester and poly-isocyanate forming urethane linkages. PU foams are widely used in applications such as furniture industry, packing, coatings, decorating, building construction, insulation, shoe industry and transportation, Wang, Zhang, Li, Yin, Wu & Mao (2008). The mixing ratio of the chemicals affects the density of PU foam made thus the related test to determine the control and identify the best mixing for the required density is crucial.

Polyurethane has already been used in the field of geotechnical and structural engineering. For problems related to the settlement of shallow foundations, which in turn deforms existing buildings, polyurethane expanding resins, are injected into the subsoil beneath the base of a shallow foundation to lift the foundation (Valentino, Romeo & Stevanoni, 2014).

Not only settlement for buildings, but settlement issues on both concrete and asphalt roadways has also injection of polyurethane grout which would raise sunken concrete roadway, improve geo-material performance, increase load bearing capacity, stabilize soil conditions and prevent piping and soil erosion (Figure 4). Similarly, rapidly expanding polyurethane foam (fully expanded in 30-60 seconds) is injected into the ground. The foam fills voids in the immediate area and compacts the surrounding soil using expansive pressure. The bearing capacity of the soil is increased as a result of the placement of the high strength foam. The slab can be lifted due to the hydraulic force created by the expanding foam. This procedure requires specialized equipment and well-trained technicians. Typically, the contractor may need to use multiple injection points to evenly distribute the lift. It may be necessary to inject holes multiple times by re-drilling the previously injected holes. It is also important to locate any buried utilities and check for obstructions before construction, Yu, Wang, & Skirrow (2013).

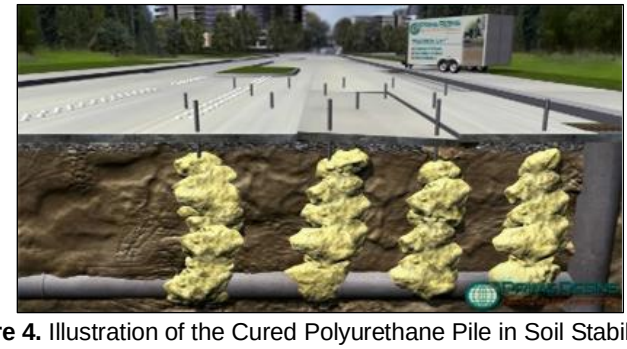


Figure 4. Illustration of the Cured Polyurethane Pile in Soil Stabilization

Plastic Bottles

The easiest and most common plastics to recycle are made of polyethylene terephthalate (PETE). Examples include soda and water bottles, medicine containers, and many other common consumer product containers. Irwan et al (2013). Malaysia is a global player in the plastic industry with currently about 1,300 plastic manufacturers. As of 2016, our exports amounted to RM30 billion which saw a 2.26 million metric tonnes of resin utilised to produce plastics. Environmental problems related to plastic waste have become a major problem in Malaysia where it has been ranked as 8th among the top ten countries with mismanaged plastic waste in the world. A study estimated that Malaysia had produced 0.94 million tons of mismanaged plastic wastes, of which 0.14 to 0.37 million tons may have been washed into the oceans, Tibbetts (2015).

Laboratory Testing on Polyurethane (PU) Cubes

Polyurethane is the product of the reaction mixture of two chemicals which is polyol and isocyanate. The mix ratio that has been used for the polyurethane-plastic bottle composite block (PUPB) is 1:1.5, where one part of polyol is mixed with 1.5 part of isocyanate. Through a volume ratio test as shown in Table 1, the 1:1.5 ratio has the maximum strength while being able to retain its expansion volume (Figure 5). Increasing the ratio of isocyanate shows that the volume of polyurethane obtained is significantly lesser and reduced the strength of polyurethane too. As shown in Table 1, the 1:1.5 ratio has the highest elastic modulus, secant modulus and yield strength compared to the other mix ratios while maintaining the expansion volume of 1:20, meaning for one part of chemical mixture of polyol and isocyanate, a volume expansion of 20 times happens to produce polyurethane.

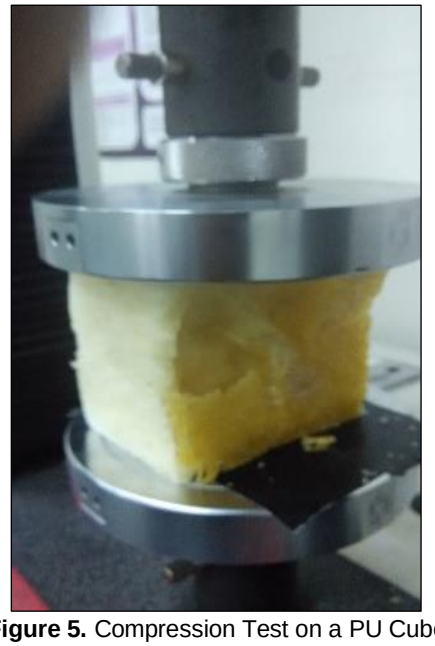


Figure 5. Compression Test on a PU Cube

Table 1. Compression Test on Various Ratio Mix of Polyurethane Cubes

Mix Ratio	1:1	1:1.1	1:1.2	1:1.5	1:2
Max. Load (N)	1510	1610	1800	2650	1900
Yield Def (mm)	14	4.8	11	7	8
Yield Strength (N/mm^2)	0.144	0.139	0.18	0.27	0.21
Yield Strain (%)	15.4	5	11.5	7.5	4.5
Young's Modulus (N/mm^2)	1.455	5.33	4.25	7.5	6.82
Secant Modulus (N/mm^2)	0.935	2.76	1.65	3.62	3.05

Test on Plastic Bottles

A similar test was conducted to measure the yield strength of plastic bottles. The test was to see if the bottles are better with cap (PE1 & PE2) or without cap (PEN1). The test also includes plastic bottles filled with water and capped (PW1) against plastic bottles filled with water without cap (PWN1). Lastly there is also plastic bottles that is filled with sand, (PS1). As shown is Table 2, the results shows that yield strength for a bottle without cap is lower than a bottle with cap. The reasoning for this can be that the stress applied on a bottle without cap is directly absorbed by the air or water initially, very little acting on the bottle itself. Whereas for a bottle with cap, the applied load acts on the whole bottle first before being absorbed by the air or water in the bottle, thus having more resistance and a higher yielding strength. Bottle filled with water or sand (PW1/PWN1/PS1) has higher elastic modulus, secant modulus and yield strength when compared with an empty bottle. Naturally, water and sand are able to absorb and resist more load compared to air. Bottle with water is 5.35 times stronger of that of an empty bottle whereas a bottle with sand is 6.35 times stronger compared to an empty bottle.

Table 2. Compression Test on Plastic Bottles

Type	PE1	PE2	PEN 1	PW1	PWN1	PS1
Max. Load (N)	53	56	44.375	538.91	35.94	462.5
Yield Def (mm)	8.1	6.92	9.02	10.43	6.28	14.7
Yield Strength (N/mm^2)	0.017	0.0195	0.008	0.091	0.01	0.108
Yield Strain (%)	3.68	3.15	4.1	4.74	2.85	6.68
Young's Modulus (N/mm^2)	0.412	0.563	3.0	4.439	3.0	2.625
Secant Modulus (N/mm^2)	0.459	0.629	1.0	4.75	1.0	1.612
Ultimate Strength (N/mm^2)	0.0276 @15%	0.0254 @8.48	0.0157 @5.73	0.191 @4.74	0.013 @4.49	0.164 @9%
Strength/ Weight	0.101	-	-	0.0171	-	0.012

Thus, it is better to use bottles with cap in designing the product and water or sand can be used to manipulate the density and strength of the product. In order to create the lightest possible block, naturally empty bottles are preferable.

Polyurethane-Plastic Bottle (PUPB) Blocks

In comparison to the density of an average backfill soil of almost 2000kg/m3, the PUPB block has a density of only 47.6 kg/m3. When used as an alternative fill in the construction of

embankments, the weight reduction is about up to 40 times. The PUPB blocks, unlike additives such as cement, lime, fly ash and various lightweight aggregates can be retrieved for the purpose of re-use/ recycle as needed, thus eliminating the possibility of becoming a waste. Figure 6 shows the arrangements of 64 bottles in a acrylic mould, and the filler polyurethane used to make a PUPB block.

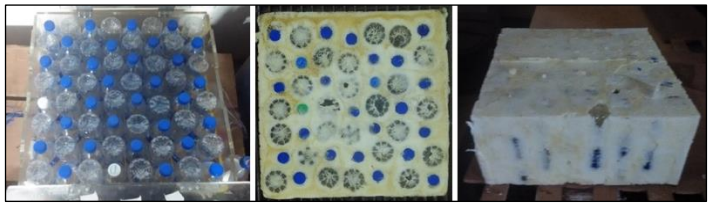


Figure 6. PUPB Block with Bottle Arrangements Before and After the PU Chemical is Poured

COMPRESSION CHARACTERISTICS OF PUPB AND OTHER COMPETITIVE PRODUCTS

Conventional Laterite Soil

Based on the JKR standard, weak sub-grade materials shall be replaced with selected materials or stabilized material up to depth of at least 300 mm below sub-grade level when it is required and specified. Based on Table 3, the minimum CBR of 5% is recommended for pavements that support traffic volumes corresponding to the Equivalent Standard Axle Load (ESAL) of Traffic Classes T1 through T3. For road pavements designed for large volumes of traffic (Traffic Classes T4 and T5), a minimum subgrade strength corresponding to CBR of 12% is recommended, Jabatan Kerja Raya (2013).

Table 3. Minimum CBR Value of Subgrade Based on Traffic Category

Traffic Category	Design Traffic (ESAL x 10^6)	Minimum CBR value (%)
T1	<1.0	5
T2	1.1 to 2.0	5
T3	2.1 to 10.0	5
T4	10.1 to 30.0	12
T5	>30.0	12

In accordance with the size the area of the product being made, a soil block of the size 500mm x 500mm was made with a height of 184mm. Laterite soil which is to be used as fill material from a construction site in Parit Raja was taken for testing. The soil block was compacted in a mould and then wrapped with a plastic film around its sides to avoid spillage upon failure after the testing as shown in Figure 7.

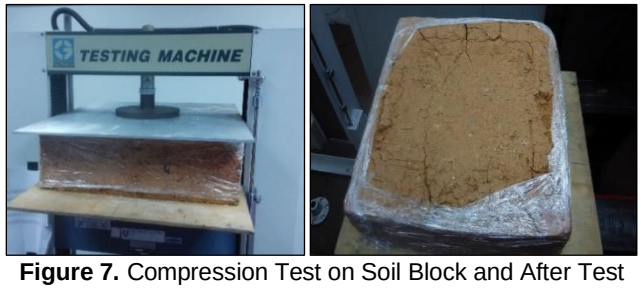


Figure 7. Compression Test on Soil Block and After Test

The purpose of making a soil block is to emulate the rigidity and stiffness characteristics of the innovated lightweight blocks and to be tested under similar conditions so that comparison between strength can be made. Soil compaction is done at construction sites to mechanically increase the dry density of the soil by rapidly expelling air out primarily. As it can be seen in Figure 7, cracks becomes visible and the blocks begins to disintegrate thus reaching a failure point.

Geofoam

Geofoam or the Expanded Polystyrene (EPS) as shown in Figure 8 is a lightweight product that has been used as an alternative fill in the construction industry for many years and thus would be a good comparative product to be used for this research.



Figure 8. Geofoam (500mm x 500mm x 200mm)

The practical application of Geofoam includes a minimum of two layers of blocks should be used for lightweight fills beneath roads because a single layer of blocks can shift under traffic loads and lead to premature failure. Block thicknesses typically range between 610 mm (24 in.) to 1000 mm (39 in.). Therefore, it is recommended that a minimum of two EPS blocks with a thickness of 610 mm (24 in.) each or a total initial height of 1.2 m (4 ft) be considered for the EPS block height to determine the preliminary embankment arrangement during the design process. The preliminary height of conventional soil fill is the total embankment height required based on the background investigation less the preliminary pavement system thickness of 600 mm (24 in.) and the thickness of two EPS blocks of 1.2 m (4 ft), Stark (2004). Figure 9 shows the practical application of geofoam in an embankment.

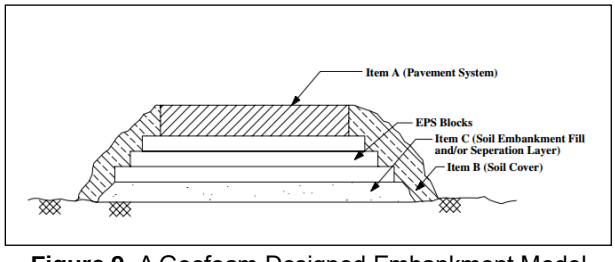


Figure 9. A Geofoam Designed Embankment Model

Figure 10 shows a typical graph of the stress-strain relationship for the Geofoam, Beju & Mandal (2017). EPS behaves as a linear elastic material up to a strain of about 1% as shown in the figure above that depict the stress-strain response of EPS. As a result, the design

recommendation for EPS geofoam is to limit loading to the compressive resistance at 1% strain. The stress at a compressive strain of 1% is called the elastic limit stress and is measured in a standard rapid-loading compression test. Except for special compressible applications, higher compressive strain, e.g., 5 or 10%, is not used to estimate the EPS strength because these strains are past the yield strength of the EPS and this may lead to undesirable permanent strains.

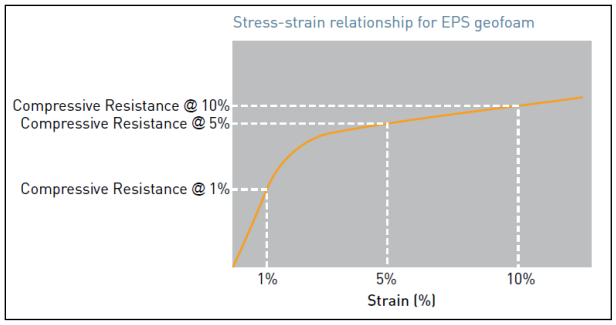


Figure 10. Typical Graph of Stress-Strain Relationship for Geofoam

Polyurethane Plastic Bottle (PUPB) Blocks

The PUPB block was made with different numbers of bottles throughout its designing process. During the initial stages, various bottle arrangements, as shown in Figure 12, were considered to determine the most optimum design in terms of strength and cost, and it was eventually found out that even though PU is the major contributor to the PUPB blocks strength as can be seen from the yield strength of PU and plastic bottle from Table 1, Table 2 and Figure 11 respectively, there is not a significant change in strength or reduction in rigidity when more bottles are used. Whereas using more bottles in the design reduces the cost of PUPB blocks drastically, as bottles are very cheap in comparison with PU.

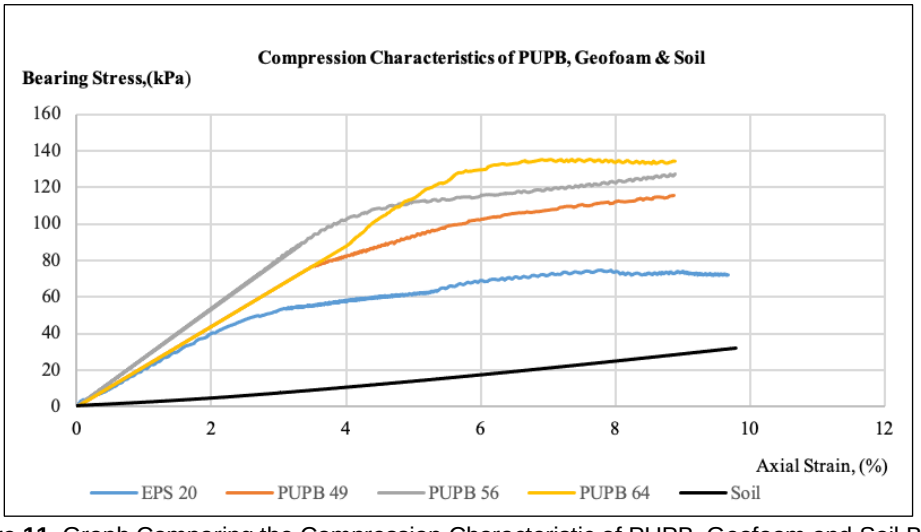


Figure 11. Graph Comparing the Compression Characteristic of PUPB, Geofoam and Soil Blocks

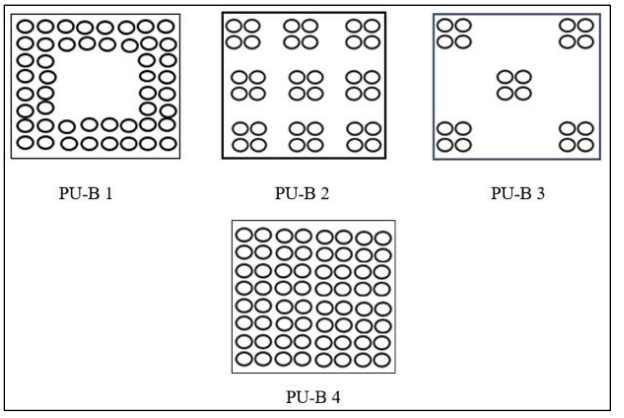


Figure 12. Various Designs of PUPB Blocks

The advantages of utilising plastic bottles in the PUPB blocks makes it easy to manipulate the density of the blocks by adding water or sand inside the blocks. Thus, besides manipulating the bottle arrangements, tests were conducted with blocks made with bottles filled with water as well, and Table 4 summarizes the results of the compression test. The number of bottles used in a PUPB block is indicated accordingly, for example a PUPb64 contains 64 bottles whereas PUPB49 has 49 bottles in it. As shown in Table 4, using more empty bottles, PUPB 64 E results in a higher strength compared to blocks with lower number of empty bottles such as PUPB49 and PUPB56. Using bottles filled with water, PUPB56W and PUPB49W, has a higher compressive strength but a much lower strength/weight ratio, when compared to blocks made with empty bottles. The strength/ratio parameter is a significant one because when the PUPB blocks are made with empty bottles, the ratio is over 1000 times stronger than its weight comparatively, whereas this value falls down to over a 100 times when bottles filled with water are used instead. Even though compressive strength of PUPB-W is higher however returning to the main aim of the research of a lightweight block and to promote easy workability and installation at site without needing heavy machineries for lifting purposes, the PUPB-E, blocks with empty bottles are much more preferable.

Table 4. Compressive Strength of Different PUPB Designs

No.	Block Type	Mass (kg)	Max. Load (kN) @15mm def.	Strength/ Weight ratio	Elastic Modulus (Mpa)
1	PUPB 56-E	2.55	31.06	1241	4.686
		2.55	41.56	1661	1.884
2	PUPB 56-E2	2.62	29.3	1140	3.339
3	PUPB 49-E	2.6	26.6	1043	1.166
4	PUPB 64-E	2.57	33.59	1332	3.672
5	PUPB 56-W	31.9	50.04 @ 14mm def.	159.9	3.713
6	PUPB 49-W	26.1	50.23	196.18	3.215
		26.1	38.83@ 10.68mm def		3.461

Comparison Between PUPB, Geofoam And Soil Block Strength

From the graph in Figure 10, it can be seen that the soil block has the lowest strength and reaches yield strength at about 45 kPa. The Geofoam which is EPS with a density of 20 kg/m^3, is the next stronger material with a yield strength of about 50 kPa. The PUPB blocks which are of three kinds shows that as more bottles are used, the strength of the blocks

increases, with the PUPB64E having the highest yield strength at about 120 kPa, thus being twotimes more stronger than the soil block and the Geofoam. The soil block despite being heavier about 40 times in comparison with Geofoam and the PUPB blocks, loses its integrity under pressure and begins to cracks, unlike the rigid mat concepts of the blocks.

The PUPB with an increasing amount of bottles used shows a higher compressive strength, which proves that plastic bottles are a good reinforcement when used together with polyurethane which is the filler. The higher the number of bottles used, the stiffer would the PUPB block be made. Using more plastic bottles for this purpose does not only recycle waste material but also has the added benefit of reducing the production cost of a PUPB because the cost of plastic bottles is almost insignificant when compared to the cost polyurethane. The Geofoam has been labelled as EPS20 because it has a density of 20kg/m^3, and through this research has been proven to be less stiffer than the PUPB, however if a much denser Geofoam is used, the compressive strength would definitely be higher, but since this research advocates the use of waste materials, thus Geofoam falls short of such condition.

CONCLUSION

As a conclusion, moving forward from the conventional fill method which forms a heavy layer of dead load on top of the subgrade soft soil, it is more appropriate to think of a lightweight fill layer, yet superior in terms of rigidity, strength and stiffness. The PUPB block, in addition to conforming to these terms, also has added value by utilising waste plastic bottles and this is beneficial to the environment, increases the stiffness of the block and also directly reduces the cost of manufacturing these blocks. As the construction industry grows, sustainability is a highly pursued theme and its significance can never be underestimated, thus this prototype is an effort towards such direction. As seen from the results, the PUPB-64 does not only maximises the use of plastic bottles in its design, it also has the lowest manufacturing cost and has a higher compressive strength compared to the soil and Geofoam blocks. Since this is part of an ongoing research, thus the prototype would have to be tested with other similar lightweight concept products in a field test to observe and analyse its efficiency in reducing settlement under load (infrastructure).

ACKNOWLEDGEMENT

The author would like to express his gratitude to all his supervisors for their constant guidance throughout the research period. The project is also funded by a FRGS grant U-1626 and a MyBrain scholarship for the postgraduate researcher.

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CORROSION OF STEEL UNDER SIMULATED CONDITION OF OFFSHORE SEABED SEDIMENT

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Abstract
Steel pipelines subjected to corrosion at any time when placed in adverse environments. Marine sediments are corrosive in nature and can allow severe damage to steel facilities underneath. There had been incidents to indicate the potential of oil and gas pipelines that carry hazardous materials to cause financial damage, environmental issues and pollution. This paper studies the behaviour of corrosion in pipelines steel when placed in seabed sediment environment. Simula-tions on sediment of different characteristics have been carried out to examine their effects on steel coupons buried in them. Consequently, the corrosion rate of the steel specimens were determined via weight loss technique. The results show that metal weight loss increases as the duration of exposure to seabed sediment environment becomes longer. The corrosion order for types of sediment existing in seabed simulated environment is found as follows; fine sand < medium sand < clay < coarse sand. In conclusion, the indoor simulation of seabed sediment can be conducted and is found relatively reliable to explain some corrosion phenomena for a short period studies.

Keywords: *Corrosion; pipelines; offshore; seabed sediment; steel.*

INTRODUCTION

Corrosion of marine steels much discussed and known in widespread number of publications in the past years (Platt and Harries, 2018; Ikeuba et al. 2019) and (Refait et al. 2020). Pipelines play a pivotal part all over the world as mean of carrying gases and oil from plants or refineries and finally to the consumers. In petroleum manufacturing industry, steel pipelines such as grade X70, X65, X42, etc., which usually used to transport oil has corrosion problem. Corrosion happens on pipelines located on the land and below the seabed. Around 20-40% of pipelines' incidents recorded due to corrosion (Palmer and King, 2008). Corrosion issue is important as it may disrupt or discontinue the operation of pipelines systems, with the fact saying that only 60% of the globe's inventory of oil/gas pipelines is not yet at their nominal design life (Palmer and King, 2008). Therefore, with this high percentage of pipeline systems, which have exceeded their designed life span, the risk of failure due to corrosion is high. On overall, corrosion progression in pipeline is an electro-chemical process (Peabody, 1967).

Disastrous incidence that occurred due to pipeline failure could be as severe as the one that reported death of twelve people, and destroying three vehicles, when a natural gas transmission pipeline exploded. Furthermore, the estimated direct cost of corrosion for industries, analysed in 2001 was about 27% of the infrastructures related to gas and liquid transmission pipelines (Pierre, 2007). The internal or external corrosion defects is due to environmental factors. Corrosion that occurs on aging pipeline reduces its structural integrity before failure (Teixeira et al. 2008). Consequently, growing quantity of aging pipeline used

in operation may considerably intensify the amount of accidents (Yang et al 2015). External corrosion is the focus of the present research due to the concerns highlighted by Gomes et al. (2013). Shokrzadeh and Sohrabi (2016) and Biezma et al. (2018). Hence, a detailed study is required to explain the behaviour of pipeline metal loss in order to improve the monitoring systems and limit the accidents due to corrosion. However, some forms of corrosion expected to occur in pipelines summarized in the following paragraphs.

In the past, the forms of corrosion in materials were categorized by which the corrosion manifests itself. Corrosion are divided into three main categories (Czichos et al., 2006). The first category includes forms of corrosion, which can be identified by visual observation, such as; general, localized and galvanic corrosion. The second category deals with those forms that need supplementary means of examination. The third category is those, which require microscopic or scanning electron microscope for a more detailed investigation. Usually, corrosion in pipelines are general or localised type. However, pipelines could face many other different types of corrosion (Cosham and Hopkins, 2004). Therefore, this article aims to investigate the influence of the surrounding marine factors on the steel coupons corrosion placed in the sediment bed.

METHODOLOGY

In the literature, some studies have been conducted indoor, but for short period of time (Duan et al., 2002) and Li et al. 2000). On the other hand, long term researches have been conducted outdoor in the real environment (Beavers, and Thompson, 2006), (Bermudez et al., 2005) and (Brossia, and Cragnolino, 2000). This study looks into an indoor investigation that has been conducted to simulate the data from the site. The main difference between indoor and site is the controlled environment in the laboratory as compared to actual site. However, the simulation has been targeted to reproduce similar results to those collected from site. All steel coupons used in this experimental study were prepared from unused steel pipes. The pipes were cut into smaller segments to facilitate transportation work of the steel material to the laboratory. The steel then underwent preparation process to produce coupons to suit the progress of the experimental work.

Preparation of Coupons

The preparation of coupons for the experimental work was done according to ASTM G162-99. Firstly, sections of the pipe in the laboratory were cut into small plates sized of 200 x 350 mm. The plates were further cut into two sizes, as required for the experimental work. The coupon size of (15 x 15 x 5 mm) was used in the laboratory works for weight loss determination. Several researchers (Sun, 2011), (Li et al. 2000) and (Zhao et al., 2007) have used different sizes of specimens for corrosion testing. Then, all the coupons were milled to produce uniform surface. Towards the end, the coupons were cleaned, dried and stored in desiccators.

Experimental Method

In the past, the use of large steel plates required much manpower and material resources. In fact, it led to loss of samples caused by offshore currents, thus test data as well. Therefore, simulation method was designed to resolve problems mentioned above, and would be able to

closely replicate the universal corrosion law for steel structure placed in the sea environment (Li, and Hou, 1999). Subsequently as indoor simulation can simulate the seabed sediment condition with experimental equipment (Wang et al., 2005), this method was applied to measure the corrosion rate of steel coupons. Hence, this method adopted to study the behaviour of steel coupons at the laboratory and under controlled conditions.

Samples of Seabed Sediment

Samples of sediments were selected carefully to have different particle sizes. The container's size determined by the amount of soil necessary for the test. At least 40 cm3 sediment was considered for every 1 cm2 of metal's exposed surface area calculated according to ASTM G162-99. The number of scheduled periodic coupons removals during the test should include two and, preferably, three specimens at any given test period to control the variability in the corrosion behaviour. Therefore, 4 coupons were scheduled for retrieval every month. The quantities of soil were calculated for 21 coupons used, in each vessel for weight loss determination. Altogether 84 coupons were used for four types of sediment. Along with sediment samples, seawater and river water were collected as fresh medium to simulate the actual site condition in the laboratory.

Corrosion Testing Vessels

Vessels or containers used to simulate the site where the sediment fully immersed in seawater all the time. The water used in this case was from the same site where the sediment was collected. However, in this case the steel coupons retrieved much more frequently than the coupons from the site. That was one of the result of adopting the simulation method, as it increases the number of retrievals to enable more data acquisition. Besides the available data in the field, the results gathered from this experiment can be also used to compare with site work results. Figure 1 shows the vessel that has been provided with a control valve to allow water to flow out. Frequent replacement of fresh water was carried out to provide similar condition to the ones at site.

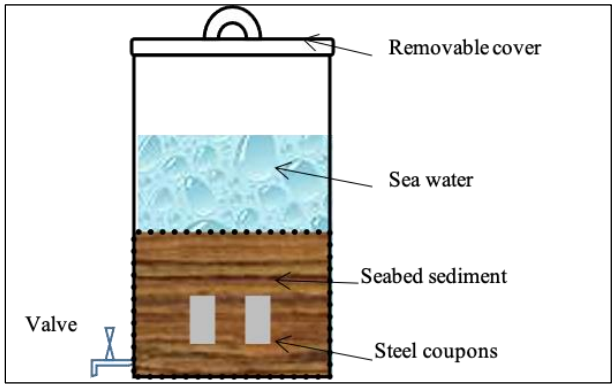


Figure 1. Corrosion Testing Vessels for Offshore Samples

Cleaning Procedure

Weight loss method was implemented to calculate the coupons' metal loss, considering the function of time and corrosion rate progress. After the retrieval of coupons which had

been subjected to sediment environment, coupons underwent the cleaning process. distilled water used to take out adherent sediment. Then, chemical cleaning procedure ASTM G1-03 applied to eliminate the remaining impurities leaving plain coupons, as shown in Figure 2. The standard covers procedures for preparing bare steel coupons for tests, for removing corrosion products after the test, and for evaluating the corrosion damage that has resulted. Then the coupons weighed repeatedly to arrive at its constant weight.

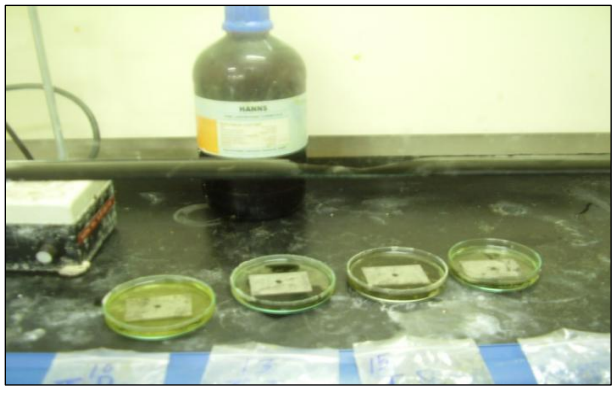


Figure 2. Chemical Cleaning of Coupons at the Laboratory

RESULTS AND DISCUSSION

A simulation of offshore environment focusing on the same types of soil used at offshore actual sites was carried out to monitor the behaviour of metal loss at the laboratory. As mentioned earlier, soils of different types and grain sizes were used in the laboratory to study the metal loss behaviour. The purpose of this simulation was to find a way to study and monitor the coupons with closer interval than site and to enrich the data on top of the data collected from site. This study continued the research conducted by authors regarding results under simulated conditions that had good correlation with actual (Budiea, 2015). The results presented in Figure 3 shows the metal loss value for each type of sediment. Coupons placed in coarse sand has experienced the highest metal loss as compared with other types of sediments. This explained by the fact that, coarse sand allows more water and dissolved oxygen (DO) to penetrate into it, and this makes it more accessible to the coupons surface. Whereas other types of sediments have smaller and closer particles, which make it difficult for corroding factors, such as, DO to reach the steel coupons.

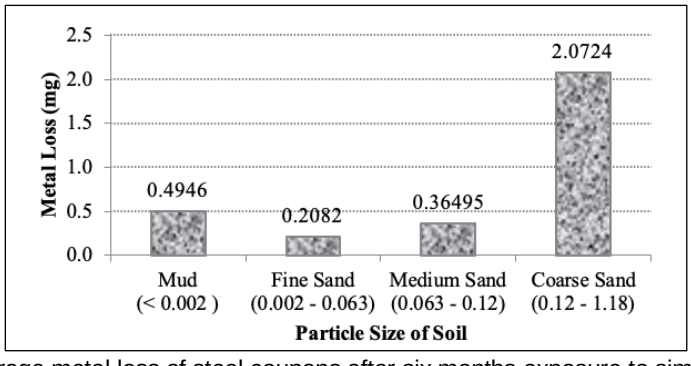


Figure 3. Average metal loss of steel coupons after six months exposure to simulated seabed sediment of various sediment grain size at laboratory

The metal loss of steel coupons exposed to simulated offshore sediment environments at the laboratory are presented in the graphs of Figure 4 - Figure 7. Sediments of various grain sizes, such as, fine sand, medium sand, coarse sand and mud were used. Generally, results indicate that metal loss increases with time for all types of sediments, though with different level of severity. In Figure 4, the metal loss against exposure time for mud presented, shows that the first month incurs the lowest metal loss of 0.028 g. Then it increases to 0.089 g in the second month, but slightly drops to 0.074 g in the third month. It then increases in the next three months with 0.154 g, 0.177 g and 0.169 g in the fourth, fifth and sixth month, respectively. Figure 5 shows that the metal loss against exposure time for fine sand starts with 0.035 g and gradually increases to 0.045 g in the second month, 0.108 g in the fourth month, but decreases slightly to 0.107 g in the fifth month. At the end of the sixth month, it increases to 0.208 g. Figure 6 presents the metal loss against exposure time for medium sand that begins with 0.036 g and increases to 0.084 g in the second month, 0.104 g in the third month and 0.194 g in the fourth month. Then, there is a drastic increase in the fifth month when it shows 0.282 g metal loss. However, it ends at 0.365 g in the sixth month. The metal loss against exposure time for coarse sand shown in Figure 7 where it reads 0.281 g, 0.465 g, 0.670 g, for the first three respective months, before the rate increases to 1.010 g, 1.229 g during fourth and fifth month respectively and drastically shoots up to 2.072 g after the sixth month.

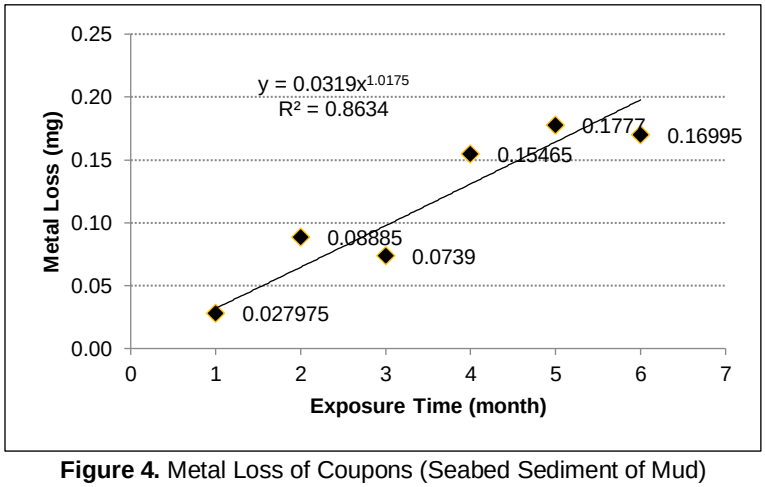


Figure 4. Metal Loss of Coupons (Seabed Sediment of Mud)

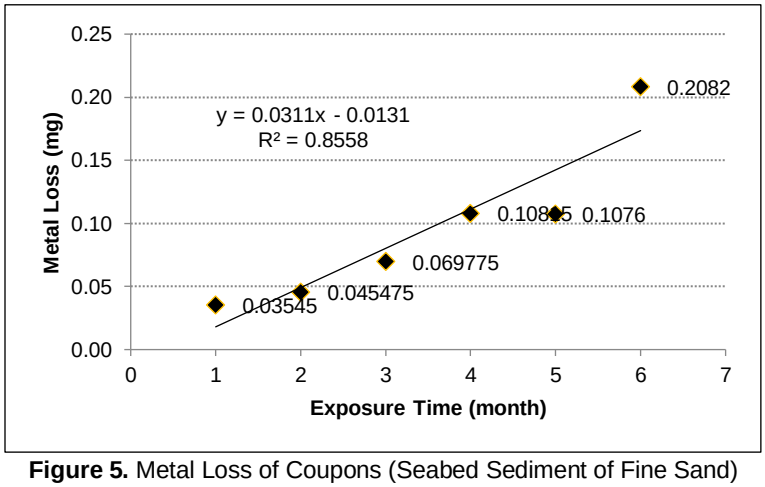


Figure 5. Metal Loss of Coupons (Seabed Sediment of Fine Sand)

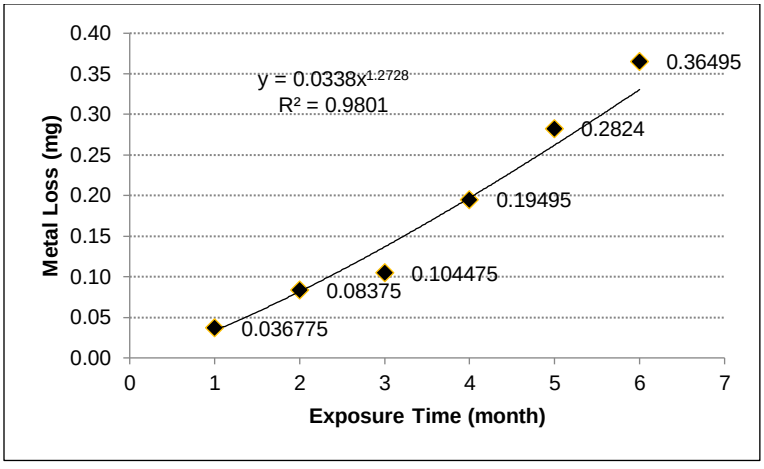


Figure 6. Metal Loss of Coupons (Seabed Sediment of Medium Sand)

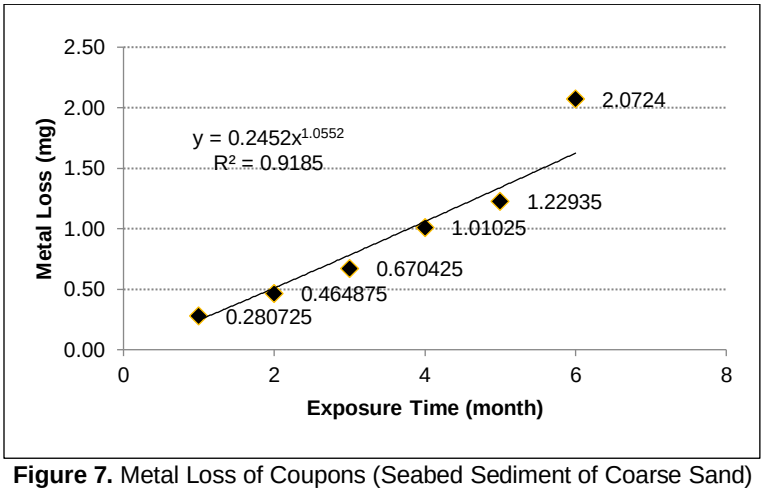


Figure 7. Metal Loss of Coupons (Seabed Sediment of Coarse Sand)

Some positive relationships between soil texture and its corrosiveness have been reported in the literature, with corrosion rate in mud for three types of steels to be less aggressive than sand (Wang et al. 2005). On the other hand, Oguzie et al. (2004) highlighted the mud to be more corrosive than loam and sand. However, findings of current study provide support and explanation for some of the previous research. When corrosion rate in sea mud appeared to be higher, this is possibly because of organic and microbial influences, which were present and more active at site, than in the laboratory environment. Microbial influences hard to predict by short-term laboratory observations (Melchers and Jeffrey, 2008). This statement could provide further support to the results of the current study. The differences may be explained by considering that the mud is attached closely to each other and to the steel coupons. Besides, the depth of water makes it difficult for oxygen to break through to the steel. That could lead to lower corrosion rate as compared with sand. Nevertheless, it is known that bacteria and some of organisms become active in the absence of oxygen hence dissolved oxygen could prevent bacteria to grow Zhang et al. (2001). Therefore, seamud with the absence of dissolved oxygen, which is the reason for bacteria that cause corrosion to be active, and therefore metal loss is expected to be higher than with other types of sediment. Corrosion under anaerobic environment can even make marine pipes perforate (Wang et al. 2005). The

presence of sulphide indicates the existence of sulphate reducing bacteria. Yuzwa (1991), who stated that the sulphate when reduced by bacterial enzymes would produce sulphide, specifically hydrogen sulphide, supports that.

CONCLUSION

The conclusions drawn from this study are as follows.

i. Simulation conducted is relatively reliable to explain the corrosion phenomena lasting for short period. Hence, for long-term purpose, it may be less accurate. Besides the difficulties facing laboratory simulation, the survival of organic and bacteria in the laboratory is rather limited, and may not represent the corrosion in seabed sediment, especially those located in tropical climate.

ii. This may be a result of its complexity of behaviour, and the difference in sediment characteristics and parameters, such as, the electro-chemical factors and chemical constituents, and other microbiological factors.

iii. Based on this study, metal weight loss increases as the duration of exposure to seabed sediment environment is longer as have been noticed for all four testing conditions.

iv. The corrosivity order for soil types existing in this offshore simulated environment is found to be in the following order, i.e., fine sand < medium sand < clay < coarse sand.

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WORK-LIFE BALANCE PRACTICES AMONG PROFESSIONALS IN MALAYSIAN CONSTRUCTION INDUSTRY

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Abstract
The construction industry is known for its dynamic working hours that require workers to work until midnight. The unhealthy work environment of the construction industry is often overlooked and lacks proper research on the work environment that is burdensome to workers. As a result of this unhealthy work environment, there is a lack of employee awareness in the construction industry itself. The concept of work-life balance has become the prominence in ensuring stability and balance between employees' work and personal life. This study examines work-life balance in the Malaysian construction industry, to determine the trends of work-life balance, analyzing the main factors which affect work-life balance and the best initiative that can be taken in practicing work-life balance practices among workers. A set of questionnaires was distributed among the employees, particularly professionals who work at the construction site. After the data was successfully collected, it was analyzed by using the 'mean scores' analysis. The result shows a value of the mean score of 3.01, which means the employee only had a medium work-life balance and it can either be dropped into worst or can be improved. It also shows the item 'employee feels satisfied when they can give their best performance to the company even they need to sacrifice their time' is the top factor under the career satisfaction factor with the mean score value 3.60. The best initiatives that most preferable by the employees to keep a good work-life balance is 'easily getting approval for unpaid leaves for taking care of their dependents' with a mean score value of 4.08. Work-life balance is a good practice in improving employees' work performance and makes employees feel more comfortable to focus at work. The factors affecting work-life balance can either bring a good or bad implication depending on how employees face the challenge and their working environment.

Keywords: *Construction, wellbeing in construction, occupational stress, work-life balance.*

INTRODUCTION

Work-life balance issue has brought attention to many people due to its negative impacts on employees, regardless of the types of workplace or industry. It can be seen the work-life balance matter seems to be neglected by the company, organization, employers and even employees due to necessity or job requirements, and also prioritization for work (Bolden-Barret, 2019). It is becoming a serious issue nowadays as employees are willing to ignore the poor work-life balance over tedious work requirements to chase after dreams like owning a physical property like lands, houses, and cars while neglecting the quality of their lives and well-being (Harris, 2014).

In the construction industry, even among construction workers, employees fail to recognize whether the work environment is good or bad because it looks normal to them. Due to job demands, employees often have to stay back for long hours to keep up with work

progress. The work ethic of people involved in the construction industry is strongly influenced by the culture of long working hours even during the weekend (Townsend, 2015). Adding up to that, Lingard, Brown, Bradley, Bailey, and Townsend (2007) stated that long and inflexible working hours were the most consistent predictor of work-life conflicts among construction employees, particularly those working on-site or in a project office. This is also supported by Panday and Bhagat (2018) and Holden and Sunindijo (2018). Poor work-life balance has an impact on the employees which indirectly resulted in work performance. All of these will result in a conflict between employers and employees (Cao, Liu, Wu, Zhao, & Jiang, 2020). It can also cause the loss of trust between the employers and the employees and can become the factor for poor health due to depression and stress. For example, it is considered 'normal' for the site engineers to be on-site for more than their designated working hours. Sometimes they have to stay on-site up to the next early morning of the next day, and yet still need to get on-site again for another working day to perform the designated job. These are some of the causes that lead employees to a bad lifestyle. It is not due to the failure of time management, but the ignorance of their rights in the workplace that makes them think that work is their only priority.

The work-life balance among construction industry workers is critical especially for site-based employees who usually work for long hours and required to work for 6 days per week. The work-life balance is an important issue for the construction industry in terms of both organizational and effectiveness, and occupational health (Brough & O'Driscoll, 2010). Jaafar, Loh, Nuruddin, Jalali and Raslim (2016) studied the work-life balance satisfaction among professional quantity surveyors in Malaysia, and found that the respondents were satisfied with their current career and family life. However, the level of satisfaction and work-life balance among other construction professionals in Malaysia is still doubtful, as there is a lack of study about the work-life balance trend in the construction industry. Therefore, this study seeks to investigate the work-life balance scenario and practices among construction workers in the Malaysian construction industry.

ISSUES OF WORK-LIFE BALANCE IN CONSTRUCTION INDUSTRY

Work-life balance has also been defined by Bradley and Mc Donald (2005) as the level of engagement or 'fit' between the various roles in one's life and the specifications for one's work and personal life. According to Naithini (2010), work-life balance refers to the effective management of multiple responsibilities at work, at home and in other aspects of life. Clark (2001) defines work-family balance as the satisfaction and good functioning at work and home, with a minimum of role conflict. The employer often expects that by working long hours, the outcome of work will be productive and better. They should also expect that there will be several times that the employee may reach the limits. The outcome of work at that moment may not be perfect because it was done half-heartedly by the employees. In other words, the workers' focus had lost or no longer about work (Thompson, 1999). Work-life strategies can be proposed by superior or employer by providing the best flexible program for their employees and family members such as maternity and paternity leave, flexible work scheduling, eldercare assistance, childhood health programs and gymnasium (Lingard & Francis, 2004; Malone & Issa, 2013).

The construction industry always exposed to the non-conducive environment such as a dirty, dangerous and difficult working area. The employees have to face the risk of work-

stress due to excessive workloads, time constraints and deadlines (Love et al., 2010; Lingard & Francis, 2004). Lingard and Francis (2004) stated that the workers who work with construction companies were likely to expose to a higher level of work stress compared to those who work with consultants and clients.

The construction industry is also well known for its ups and downs and its workplace has experienced substantial transformation in recent years. High-pressure project-based nature work increases the potential for imbalances in work-life commitments (Leung, Liang, & Olomolaiye, 2016). Construction professionals are often expected to work extra hours. Recent research suggests that working these long hours can be linked to increasing work-life conflict and can increase the risk of developing mental and physical health issues (Lingard, Francis, & Turner, 2015, Sunindijo and Kamardeen, 2017). According to Glosser and Golden (1997), overtime work hours become a trend and perceived as normal to the community since the 1970s. In Australia for instance, most Australian construction employees are working on average hours of at least 62.5 hours each week (Lingard and Francis, 2004).

Factors Influencing Work-life Balance in Construction Industry

Technology

Information and Communication Technologies (ICT) can be classified under one big theme which is sociological theories. ICT's is having their capacity for humans to reorganize their time and space (Nam, 2014; Wajcman, Rose, Brown, & Bittman, 2010). The smartphones nowadays come with the package of the internet and can keep the people in touch with one another. Unfortunately, this advantage is also had destructed the boundary between work and family quality time. Most of the employees nowadays hoping that there is an existence of a fine line between the work environment and also outside the work environment (Nurisman & Sampurna, 2019; Demerouti and Cropanzano, 2010). The use of ICTs such as the internet, email, laptops and communication devices such as smartphones and tabs allows workers access 24/7 without interruption, making these two aspects of work and the outside work environment difficult to define (Ferguson et al., 2016; Leung, 2011). Adetayo, Olaoye and Awolaja (2011) emphasize that ICT makes communication easier but affects work-life balance that requires employees to keep working even after office hours.

Business Culture

Most of the big companies usually provide their guidelines and regulations for their workers during the hiring process but often neglected and subsidized on their benefits and incentives (Demerouti and Cropanzano, 2010). Without realizing it, companies will suffer more losses than they think have been saved, especially as they begin to encourage employees to work longer. As the intensity increases day by day, the workers will somehow reach their limits, mentally exhausted and eventually experience less productive days. This can lead to less creativity and less idea in solving problems at work even in small problems. Implementing work guidelines that have been agreed upon by management and employees is very important. Human resources must take action and provide a way for workers to avoid overtime.

Holden and Sunindijo (2018) found that long work hours (which is a norm culture in the construction industry) have a very negative correlation with work-life balance. The management should restrict the implementation of extra working hours per day or month (Leung, 2011). Shift working hours can be one of the options for consideration when dealing with critical time to meet the project's deadline. Having a 'family day' program is also a good way for stress reliever for the employees (Panojan, Perera & Dilakshan., 2019).

Commute Time

According to Panojan et al (2019), commute time is vital for the work-life balance of the construction workers, particularly professionals who have to balance their free time against travel time. Commuting can be defined by the terms of distance or time traveled (Koslowsky and Krausz, 1996) which indicates the regular travel between home and place of work. Wachs, Ong, Levme and Taylor (1991) mentioned that the growth of the workforce had contributed more to the growth in local traffic congestion that had lengthening of the work trip over time. The automobile remains the dominant mode of travel between home and works for the employees, and the choice of the residential locations was determined based upon many factors in addition to the home-work separation such as quality of neighborhood and schools and guaranteed safety living region. Turcotte (2011) found that longer commuting time was associated with worse work-life balance. Moreover, commuters who have longer commuting time tend to experiences more difficulties in fulfilling family responsibilities. Apart from that, the employer who spends long hours in commuting time tends to sleep less and have fewer family time (Mukhtar, 2012).

Salary Compensation

High salary compensation is like a bonus and a gift for the workers. Research has found that there is a positive and significant effect of compensation on job satisfaction (Mabaso and Dlamini, 2017). It is one of the reasons why the workers choose to stay even the work feels burdensome for them (Wachs et al., 1991). However, a high salary alone is not enough to positively impact the work-life balance of workers (Coyle, Van Leer, Schroeder, & Fulcher, 2015). This is because most construction sites operate on a six-day week basis, and professionals and managers work many hours of unpaid overtime. This is proven in Holden and Sunindijo's (2018) found that there is a significant correlation between being adequately compensated when working longer hours and greater work-life balance. In their study, the respondents who are the site-based construction professionals felt inadequately compensated when they receive the same amount of salary even though they work more than designated hours.

Career Satisfaction

The concept of work-life balance has been abstracted from the career satisfaction level of employees to provide a good work-life balance and productivity levels of employees (Mukhtar, 2012). McMahan (2018) identified that the career satisfaction of employees was related to the environmental condition, departmental climate, and demographics of the organization in which employees used to work. Career satisfaction is a pleasurable and emotional state resulting from the perception of one's job as fulfilling or fulfillment of one's career values (Calvo-Salguero, Carrasco-González, & Salinas-Martínez de Lecea, 2010). It

can also be elaborated as the employees' perceptions of their working environment, the relationship among colleagues, earnings and promotion opportunities (Belias and Koustelios, 2014). Oshagbemi (2003) stated that career satisfaction is an affective reaction to a career that results from the comparison of actual outcomes with those that are desired. The recognition of organizational was linked with the work-life balance, which leads to employee satisfaction and the effects of employee contribution and affects the levels of satisfaction (Sageer, Rafat & Agarwal, 2012). There is evidence that job satisfaction can positively increase employee's performance (Abdirahman, Najeemdeen, Abidemi & Ahmad, 2018).

Strategies to Improve Work-life Balance

Work-life balance strategies are also reported to improve organizations' competitiveness by increasing their ability to attract employees (Sheppard, 2016; Shagvaliyeva & Yazdanifard, 2014); induce employees to exercise discretionary effort in performing their work (Chang, Hsieh, Lan & Chen, 2019; Konrad & John, 2010); and help employees to be more productive (Jackson & Fransman, 2018). Arthur (2003) reported a positive relationship between the announcement of organizational work-life balance initiatives and shareholder returns, indicating that investors view family-friendly firms more favorably. Grover and Crooker (1995) found that employees in companies with organizational work-life balance benefits had higher levels of organizational commitment to the organization and expressed lower turnover intentions, regardless of whether the employee individually benefited from the policy. They postulate that work-life balance benefits have a positive influence on employees' attachment to the organization because they signify corporate concern for employees and their families.

Arif and Farooqi (2014) and Francis (2004) had reported a link between an organization's commitment to work-life balance initiatives and the employee's commitment to the organization. Sun (2011) in his study of civil engineers reported higher commitment, job satisfaction, and lower turnover intention when the employees' perceived their organization to be supportive of work-life balance initiatives. These results provide evidence that work-life balance initiatives are linked to organizational performance in the Australian construction industry.

METHODOLOGY

Questionnaire Development

A questionnaire survey was used to attain respondents' level of work-life balance, their opinions and perceptions on the factors that affect the work-life balance, and strategies to improve work-life balance. The survey instrument was established by adapting the work by Holden and Sunindijo (2018). A pilot test was conducted with 10 panels of experts that had more than 10 years of experience in the construction industry, to ensure content validity and reliability of the questionnaire. Next, the amended questionnaires were distributed to employees who are working at the construction site in Klang Valley. The selected respondents include the site/project engineers, site/project managers, quantity surveyors, safety personnel and others. The questionnaire consists of 4 parts, as follows:

i) Part I – Respondents were required to provide their background information such as gender, job position, years of experience, working duration, and type of current project;
ii) Part B – The respondents were requested to rate statements regarding work-life balance trend, based on a 5-point Likert Scale, ranging from 1 (totally disagree) to 5 (totally agree);
iii) Part C – The respondents were asked to rate statements regarding the significant factors affecting work-life balance such as technology, business culture, commute time, salary compensation and career satisfaction based on a 5-point Likert Scale, ranging from 1 (totally disagree) to 5 (totally agree); and
iv) Part D – The respondents were requested to rate statements regarding significant strategies to improve work-life balance, based on a 5-point Likert Scale, ranging from 1 (very insignificant) to 5 (very significant).

Measures for Data Analysis

The analysis was carried out by using SPSS 25.0 software and further divided into two tests, namely instrument reliability test and mean score analysis.

Instrument for Reliability Test

The purpose of conducting the reliability test was to check the reliability of the result and the overall consistency of measures for the entire survey. The Cronbach's Alpha statistic was employed to measure the instrument's reliability (i.e., internal consistency). Generally, an Alpha value greater than 0.6 indicates acceptable reliability, while a value greater than 0.7 signifies good reliability (Cronbach and Shavelson, 2004). The results of the reliability test revealed that the Cronbach's Alpha values for questionnaire variables are more than 0.7, which indicates good reliability and internal consistency.

Mean Score Analysis

The data gathered from the survey was analyzed using the mean score analysis. The mean value or score of a certain set of data is equal to the sum of all values in the data divided by the total number of values (Holden and Sunindijo, 2018).

Participants

The survey form was distributed to the participants in the form of a self-administered questionnaire and online platform (google form). The participants of the survey were the employees working at construction sites in Klang Valley. A total of 95 returned questionnaires were received and analysed. The summary of the participants is depicted in Figure 1. From the figure, it could be seen that majority of the respondents work 6 days per week, work more than 61 working hours in a week and involved in infrastructure projects.

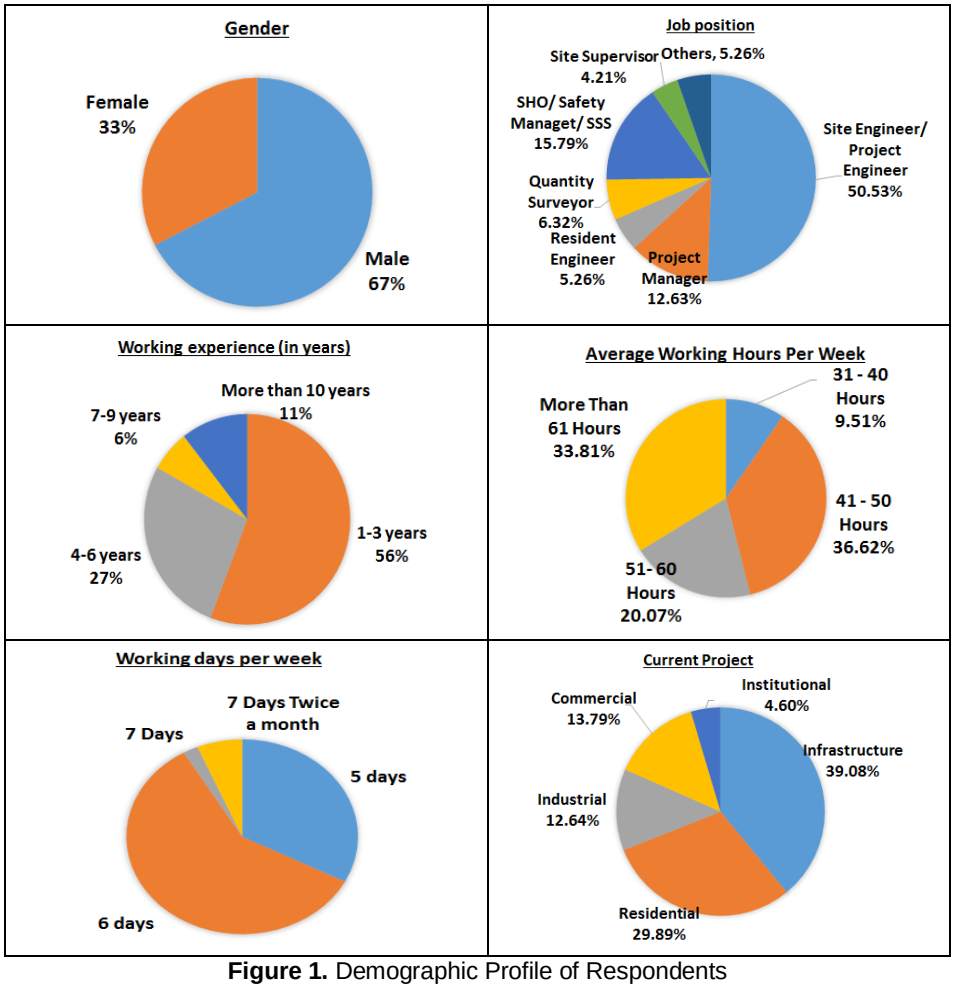


Figure 1. Demographic Profile of Respondents

RESULTS AND DISCUSSION

Work-Life Balance Trends

Table 1 summarises the work-life balance trend scored by respondents. The ranking of all items had been sorted out in descending order, with the average mean of 3.01, indicating an average level of work-life balance. In general, the findings from this study are relatively similar to the previous study by Holden and Sunindijo (2018). Item 9 and 10 were the two lowest items, denoting how the employees were struggling to fit-in or juggle between the multiple roles in working and personal life (Bradley and Mc Donald, 2005; Naithani, 2010; Clark, 2001); and effectively control the work-life conflicts while completing the task (Holden and Sunindijo, 2018; Baker, 2012). The long working hours seem normal to construction industry employees in recent years, therefore employees tend to set works and responsibilities towards the company as the upmost priority until the family time and self-comfort tend to be neglected.

The positive aspects of the work-life balance of the selected respondents are conveyed by items 1, 2, 3 and 4 where the mean scores are more than 3. Most of the respondents could

manage their temper and emotion at work while having relatively high job satisfaction, despite experiencing a hectic work schedule. The finding is in agreement with Beautell (2010) that measuring the quality of work-life balance arrangements should not be only focusing on the time spent completing work and non-work, but also must take the consideration in terms of satisfaction in the task given. Apart from that, the finding is also in-line with Holden and Sunndijo (2018) that "despite the indication of work-life conflict (item 9), employees still managed to participate in social or family events regardless of their work commitments", as seen in item 3. The contradicting result may be due to the initiatives taken by the respondents themselves or respondents' organizations that emphasize family time (De Cieri, Holmes, Abbott and Pettit, 2005).

Table 1. Respondents' Level of Work-Life Balance

No.	Items	Mean Score	Ranking
1	I am able to maintain my temper at work	3.43	1
2	I feel satisfied with my current work	3.32	2
3	I manage to participate in social or family events despite my work commitments	3.27	3
4	I have adequate control over my work-life balance	3.12	4
5	I normally don't work through my lunch break on a weekly basis	2.91	5
6	I make time to relax on weekdays	2.89	6
7	I have adequate control over my work-life balance	2.87	7
8	I normally don't think about work after office hour	2.79	8
9	My family and friend are happy with my working hours	2.77	9
10	I feel relax while at work	2.74	10
	Average	3.01	

Factors Affecting Work-Life Balance

As described earlier, there are five categories of factors that influence work-life balance in the construction industry. Table 2 shows the results of the mean score analysis on the variables for each factor category, as perceived by the respondents.

In general, the results indicate low to above average agreement on factors that influence work-life balance, indicating that this is common among construction workers. Most scores are contrary to the findings of previous studies by Holden and Sunindijo (2018), which were implemented in Australia and showed higher scores. However, respondents felt that they wanted to quit their job as shown in T1. In a research report on the effects of technology (Holden and Sunindijo, 2018), it was stated that workers find it difficult to control the number of hours spent on mobile devices and that workers prefer such employers to avoid excessive use of technology.

Among the highest points scored by respondents for factors influencing work-life balance were related to career satisfaction. While it is clear that workers face high levels of exposure to mental stress such as traffic jammed along the way to work (C1) and work more than 5 days a week (B2), their perception of career satisfaction is high (CS1). This study shows that achieving goals and completing work on time is more important than having leisure time. It is supported by the second position among the career satisfaction factors that employees have mentally prepared for this dynamic working hour.

Table 2. Factors Affecting Work Life-Balance

ID	Items	Mean score	Std. Dev.
	Technology		
T1	I tend to mute and refuse to check any updates from my work chat group	3.28	1.19
T2	Superior and co-worker keep on asking and calling even when I am on leave or vacation	3.15	1.23
T3	Implementation of using Information and communication Technology (ICT) is disturbance	3.00	1.15
T4	At least three hours are spent using ICT on work during weekend	2.97	1.26
T5	I could not differentiate an exact timeline that I need to perform on work	2.94	1.05
T6	ICT had violated my privacy life	2.81	1.02
T7	It is hard using ICT as a medium to discuss regarding work	2.62	1.05
T8	I usually turn off my internet connection when I arrive home	2.45	1.10
	Business Culture		
B1	My company has own policies regarding office hour	3.54	1.13
B2	I work more than five days per week	3.46	1.37
B3	I enjoy with this kind of working environment	3.33	0.92
B4	I am fine with working overtime	3.12	1.08
B5	My company implementing working hour based on the Employment Act 1955	3.02	1.05
B6	I feel at ease during weekend, no need to think about work	2.97	1.18
B7	I often work until midnight	2.92	1.18
	Commute Time		
C1	I have to face traffic jammed everyday	3.44	1.37
C2	I have to supervise more than one site	3.06	1.38
C3	I take more than half an hour from my residential area to workplace	2.86	1.34
C4	I often get the task to work outstation	2.72	1.06
C5	The distance between my site and office are near	2.67	1.27
	Salary Compensation		
S1	I stay in this company because I need to have a stable salary	3.38	1.13
S2	I am willing to work extra time in order to get extra payment	3.33	1.24
S3	My current salary and workload are unreasonable	2.92	1.18
S4	My current salary is according to my demand and above expectation	2.83	1.26
S5	I am satisfied with the salary offered by the company	2.81	1.02
S6	I get extra payment when working overtime	2.33	1.28
	Career Satisfaction		
CS1	I feel satisfied when I give my best performance to company even though it need to sacrifice my time	3.60	1.00
CS2	I have mentally prepared for dynamic working hours in this profession	3.58	1.06
CS3	I manage to learn new thing everyday	3.57	1.03
CS4	I feel like time spending at site is valuable time for me	3.51	1.02
CS5	I love to work for a long hour as long as my task completed on time	3.02	1.23

In addition, they are mentally prepared for the dynamic working hours of their profession (CS2). These can be related to the salary compensation in which the respondents feel that their salary is adequate (S1) and that they are willing to work longer hours and face the difficulty of construction work (S2). This result is supported by the findings from Mabaso and Dlamini (2017), who stated that wage compensation positively affects overall job satisfaction, motivation and leads to less absenteeism and turnover behavior. And in the last rank among salary compensation factors, even though employees do not earn extra pay by working overtime, they are still willing to stay at the company because of the demand for

today's life. Arif and Farooqi (2014), found that when employees were satisfied with their careers and commitment to the organization, they improve their work-life balance.

Strategies to Improve Work-Life Balance

Table 3 shows the strategies perceived by respondents to improve work-life balance. The results show that the most important strategy is the speed of employers to approve leave applications. This indicates that respondents are still committed to their lives and families and appear to be demanding more free time with their families. This is in-line with Jaafar et al's (2016) study that most of the respondents (professional women quantity surveyor) preferred their companies to implement written family-friendly policies in supporting work-life balance.

In addition, respondents also prefer to allocate more employees on a larger project scale. Pongpanich and Socorro (2019) emphasize that peer support, organizational support and supervision are powerful initiatives to improve work-life balance. Therefore, by providing more employers in larger-scale projects, it will help speed up project completion and employees will begin to have a good work-life balance. The results also showed that respondents were less concerned about other initiatives by employers who emphasized reducing work stress.

Table 3. Strategies in Work- Life Balance

No.	Items	Mean score	Std. dev.
1	Unpaid special leaves to care for dependents easily getting approved	4.08	0.96
2	Unpaid special leaves for other purposes easily getting approved	3.82	1.01
3	Allocation more workers and professionals on bigger task project	3.81	0.96
4	Paid maternity leaves	3.77	1.17
5	Allocate extra days for unpaid paternity leaves	3.72	1.01
6	Time-off to attend non-work events (dinner, company event)	3.65	1.04
7	Allowing adequate break during working hours (pray and lunch)	3.62	1.20
8	Providing a panel parental care home	3.60	1.18
9	Providing a playground and childcare at office	3.42	1.08
10	Providing health and well-being facilities such as gymnasium	3.40	1.27
11	Flexible work scheduling	3.27	0.97
12	Conduct a psychological well-being program (handling stress, anger management)	2.93	1.16

CONCLUSION AND LIMITATION

The issue of work-life balance has attracted many people because of the negative impact it has on workers, whether at work or in the industry. It can be seen that work-life balance seems to be neglected by companies, organizations, employers and even employees due to necessity or job requirements, and also prioritization for work. This is a serious problem as workers are willing to overlook the poor work-life balance over tedious work requirements to pursue dreams such as owning physical assets such as land, homes, and cars while neglecting the quality of their lives and well-being. Factors such as technology, business culture, travel time, wage compensation and job satisfaction also affect the work-life balance of the employees in the construction industry. Career satisfaction is the most significant factor affecting work-life balance in the Malaysian construction industry.

Work-life balance can be considered as an individual's perception of how well employees are managing their daily lives. The conceptualization of individuals subjectively measures the balance between their work and their life expectancy (Guest, 2002). At work, employees generally do not realize that they have practiced a bad work-life balance that ultimately affects their physical and mental health. Recently, it is common for most employees to work over-time, which is a bad example of everyday life and leads to adverse effects and causes of mental illness.

To address this, several work-life balance initiatives or strategies have been introduced to reduce work-life conflict between work and personal life and to better work-life balance (Lingard, Francis & Turner, 2012). Work-life balance initiative strategies can improve organizational competitiveness by increasing the ability to attract workers to make more effort while working and helping employees become more productive (Lingard et al., 2012).

This study has some limitations as it only covers respondents in the Klang Valley region and a small number of respondents do not represent the entire population. Further research should be conducted to expand the research in other states and to increase the number of respondents. In addition, it is recommended that future research be conducted to examine the correlation between work-life balance and potential predictors, such as job satisfaction, job performance and productivity, etc.

ACKNOWLEDGEMENT

The authors would like to thank the Ministry of Education Malaysia for supporting this research under the Fundamental Research Grant Scheme Vot No. FRGS/1/2016/SS03/UTHM/03/2 and partially sponsored by University Tun Hussein Onn Malaysia. The author would also like to thank all the industry individuals who participated in the survey.

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A STUDY AT A NEW CONCEPT OF LIGHTWEIGHT FILL MATERIAL CONSTRUCTION IN EMBANKMENT USING ARCH STRUCTURE

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Abstract
Construction of road embankment over marine clay can cause significant settlements that contribute to the failure of road embankment. This is due to the properties of marine clay which are low shear strength, high water content and high compressibility. Such properties are quite difficult and challenging to work with especially in building road structures such as embankments. One of the techniques in improving this kind of soil is to use expanded polystyrene (EPS) as lightweight filler materials in road embankments. This is one of the initiatives to improve the soil bearing performance and reduce the settlements of soil. Due to the irregularities in the design and an increase in demand and cost for EPS, this study proposes a new design of using an arch structure concept in lightweight fill design. A small-scale model of the road embankment will be used to simulate where settlement will be measured on the model using EPS as the filler material. This research will focus on the different models in the concept of arrangement of fill materials and the pattern of failures from settlement analysis. The shape, orientation and size of fill materials that forms an arch structure are the important elements to be studied in this research as this contributes to the structural condition of the construction. The suitable design and arrangement of fill material will be obtained according to the observation and mathematical analysis. It is found that the arch structure provides less settlement while maintaining using limited EPS materials.

Keywords: *Road embankment; EPS; marine clay; settlements; lightweight fill.*

INTRODUCTION

Road constructions on marine clay may have difficulty for engineers and poised great danger for road users. There are many infrastructure projects and also coastal high-rise buildings where foundations are often supported by such soils of low shear strength and high compressibility.

According to Hassan (2011), the construction projects on soft soils such as clay can lead to a costly foundation system, because of the soft soil properties that causes high settlements. Marine clay creates many problems to buildings and roads due to uneven settlements where buildings will see cracks and roads to be unstable for road users.

There are few methods that can be applied to reduce settlement problems and damages that occur in road constructions like replacing the compressible soils with lightweight fill materials. One of the fill materials used is the expanded polystyrene (EPS) geofoam which can be used to replace marine clay or in place of heavy fill materials to prevent sudden settlement on the underlying soils and adjacent structures. EPS geofoam has a unit density that can be roughly as low as one pound per square foot, it is an extremely lightweight

alternative to soil, nearly 100 times lighter than most soils. This lightweigh characteristics is perfect to be used in reducing the self-weight of the embankment constructed on soft soils.

This study is done with the use of a small scale physical modelling to measure the performance of an improved design of EPS geofoam blocks for filler materals in order to reduce settlements of soil soil under an embankement construction. Theimproved design applies an arch structure concept that provides equally good stability by using less materials, and having better flow of underground water. Hence, this study will compare settlement behaviour of soft clay soils of a control embankment structure, use of conventional EPS geofaom embankment structure and the arch structure EPS geofoam concept.

CRITICAL REVIEW ON MARINE CLAY AND GEOFOAM

The dirt found in the sea bed is named marine clay and it can also be typically found in coastal areas and inland areas. The properties of marine clay depend altogether on its underlying conditions. The properties of soaked marine clay contrast essentially from wet soil and dry soil. The mud stores comprise of profoundly cracked and broken dirt and silty muds which have a high swelling potential. The depth of the regular high-water table in marine clay varies that relies upon soil stratification.Water is frequently roosted above gradually porous subsurface strata. According to Basack & Purkayastha (2009), depth to hard bedrock for most marine clays is at least 50 feet.

Physical Characteristics of Marine Clay

Marine clay or mud is described scientifically as a low penetrability and has the capacity in constriction of inorganic contaminants. According to Jones et al. (1993) the hydraulic conductivity of marine clays is low and this will provide high water content in the soil.

Benson and Trast (1995) performed water driven conductivity tests on thirteen compacted clay mud tests under controlled conditions. They found that the soil expand and consolidate significantly. Different behavior of clay particles with respect to another constituent is dependent on the different ratio exist between the solid surface and their specific surface. The bonding properties of clays and association with a particular chemical structure in swelling characteristic is related to the size and to the classification of their minerals. It refers to a constituent of a soil that is smaller than 0.002 mm for size determination (Das, 2006).

Expanded Polystyrene (EPS) Geofoam

Expanded polystyrene (EPS) geofoam is a super-lightweight unbending cell plastic frothed polymeric geosynthetic. As a lightweight fill, EPS geofoam diminishes the loads imposed on adjacent and underlying soils and structures. EPS geofoam is additionally accessible in various types of physical properties that can be picked by the an engineerin for a particular application.

EPS geofoam is innately multi-practical, which makes it compelling to use in a wide variety of utilizations. EPS geofoam is available in different material types. Thorough knowledge and understanding of the type being used on an EPS geofoam project is essential. The material properties of various EPS materials are shown in Table 1.

Table 1. Physical Properties of EPS Geofoam

ASTM D6817 Physical Property Requirements of EPS Goefoam							
Density min kg/m^3	11.2	14.4	18.4	21.6	28.8	38.4	45.7
Compressive Ressitance min, kPa at 1%	15	25	40	50	75	103	128
Flexural Strength, min, kPa	69	172	207	240	345	414	517
Oxygen, min, Volume %	24.0	24.0	24.0	24.0	24.0	24.0	24.0

Construction of Embankment with EPS Geofoam

Here stated the conventional method that used for the construction of embankment using EPS Geofoam. A typical road development, from base to top, is as per the following: compact a layer of sand at the base of the roadway excavation to give a level and free depleting development surface byplacing the EPS geofoam to the desired height, staggering the vertical joints in each course so as not to make constant vertical creases.

Whenever required, a partition layer might be set between the top of the EPS geofoam and the overlying asphalt framework. A detachment layer can have two capacities that is to upgrade the general execution and life of the asphalt framework by giving reinforcement, partition or potentially filtration and to improve the sturdiness of the EPS geofoam both amid and after construction. Choices for the partition layer include resistant geomembrane, geogrid, geocell with soil fill, soil cement, pozzolanic stabilized materials or a reinforced concrete slab.

For instance, if assurance against fuel spills is wanted, a hydrocarbon safe geomembrane cover can be put over the EPS block course to protect from conceivable hydrocarbon assault. Furthermore, a reinforced concrete load distribution slab can be utilized to protect the EPS geofoam from hydrocarbon assault and from potential overemphasizing coming from traffic loads.

EPS geofoam is innately multi-practical, which makes it compelling to use in a wide variety of utilizations. EPS geofoam is available in different material types. Thorough knowledge and understanding of the type being used on an EPS geofoam project is essential (Ratha, et. al. 2018).

Using geofoam as fill material in embankments can be up to 50 times less massive than other lightweight fills, and can give the accompanying focal points to transportation framework improvement as maximizing available right-of-way, accelerating the construction schedule, promoting clean construction adjacent to waterways, reducing construction labour, and enabling winter construction. Hence, it can be a perfect material for the usage of road contruction especially on embankements as showns in Figure 1.

EPS geofoam can also be use as refill retaining wall and covered structures likewise restricts the flat powers that can create during seismic tremors. In retaining wall applications, satisfactory channels should be given to keep the advancement of hydrostatic weight and inspire because of lightness for locales with shallow groundwater and free soils as showns in Figure 2.

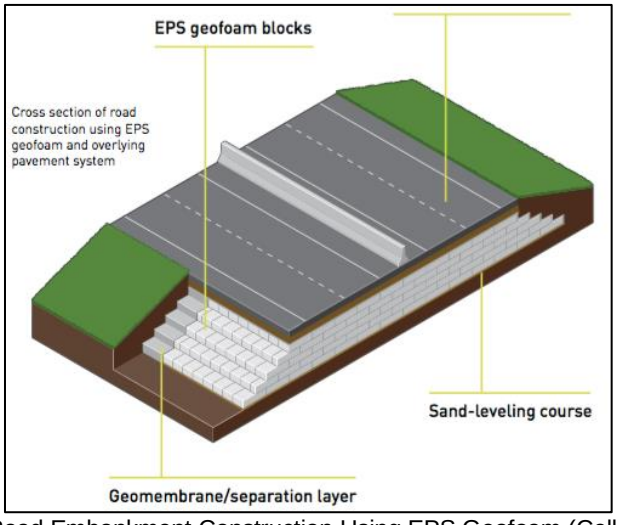


Figure 1. Road Embankment Construction Using EPS Geofoam (Cellofoam, n.d)

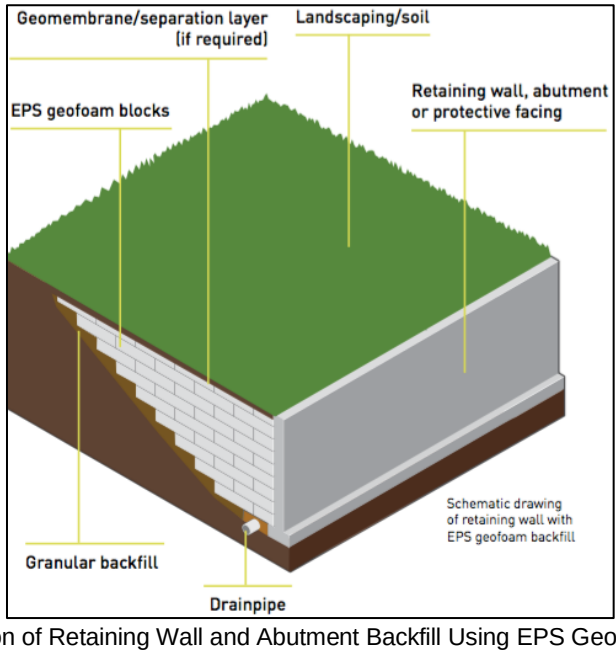


Figure 2. Construction of Retaining Wall and Abutment Backfill Using EPS Geofoam (Cellofoam, n.d)

EPS geofoam can be utilized to build railroad embankments that not over-burden the current soils. As a fill material, EPS geofoam is sufficiently solid to help railroad loads as shown in Figure 3. It's utilized as a part of the extension of the light-rail framework.

Ultimately, unstable slopes can be remediated by expelling a part of the current soil and supplanting it with lightweight EPS geofoam. EPS geofoam additionally can be utilized for slope stabilization and repair in both soil and rock slopes as shown sin Figure 4.

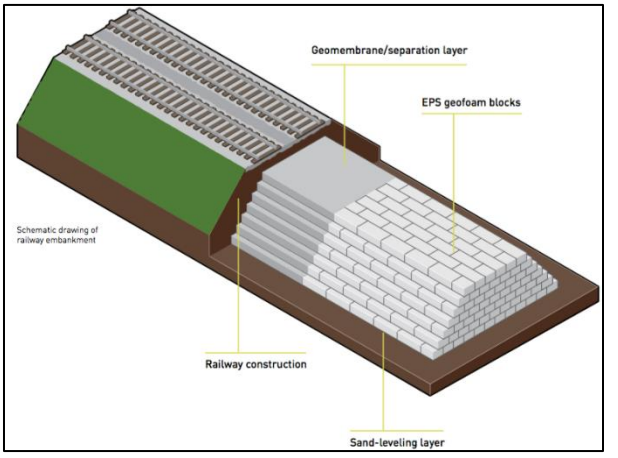


Figure 3. Construction of Rail Embankment Using EPS Geofoam (Cellofoam, n.d)

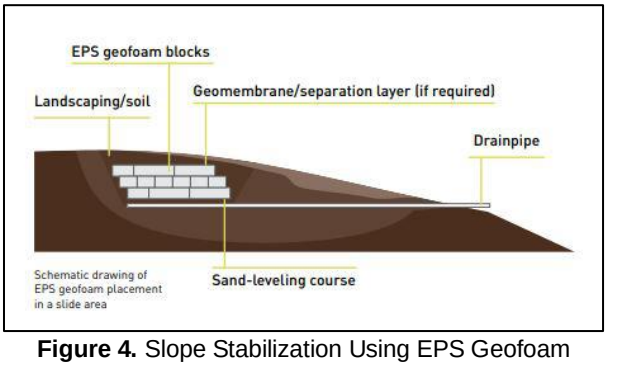


Figure 4. Slope Stabilization Using EPS Geofoam

Construction of Arch Structures

In masonry construction, arches have a few extraordinary focal points over horizontal beams, or lintels. The span have more extensive openings since they can be produced using small, effortlessly conveyed pieces of block or stone, rather than a monstrous, and solid stone lintel. This shape and the circular arch had extraordinary incentive in bridge engineering since they allowed common help by a row of arches, conveying the sidelong push to the projections at either end of a bridge.

Utilizing an arch as the essential structure when there is a requirement for spreading over a long separation is exceedingly helpful. A arch can traverse between two purposes of vertical support than a straight beam. The curved or arched design enables the structure to uproot or spread the weight equally. Its establishments are settled solidly on the ground yet its arched element helps the even dislodging of power over the arch structures (Siang et al., 2019).

The arch structure that incorporates the EPS geofoam as a filler material for road embankment provides an improved design to the coventional construction method. The arch structure concept maintains the lightweigh property of the materials that helps reduce the self weight of the soil and provide better flow of groundwater under the embankment. This will help in reducing dislodging of the EPS blocks due the rising water table that ocasionally

occurs in clay soils. The design also helps to reduce the amount of EPS used by maintain the staibliyt of the whole structure.

MATERIALS AND METHODS

A small-scale embankment model was made to simulate 3 types of testing conditions. The first test consists of a controlled test that simulates a road embankment of using sand overlying a soft clay soil. The second test consists a road conventional road embankment with EPS geofoam as the filler material. The third test is to design the arch structure EPS geofoam embankment overlying a clay soil. The performance of each of this model was done by studying the settlement levels with strategically placed indicators on the model boxes.

The embankment model was designed according to JKR road standard specification and the preparation of the soft clay will be according to the technique done by Liaki et al. (2010), Azhar et al. (2017), Basac and Purkayastha (2009), Ganasan et al. (2016), Siang et al. (2012). The sample preparation technique in the model box is also in accordance to the compaction method as stated in the ASTM standard Madun et al. (2016), Izzati et al. (2017). The real dimension of road is scaled down to 1:10. The proposed size of test box is 75 cm (L) x 25 cm (W) x 60 cm (D) as shown in Figure 5. The sandy soil and marine clay was taken from the Research Centre for Soft Soils (RECESS) in Batu Pahat, Universit Tun Hussein Malaysia (UTHM). The slotted weight is used to simulate vehicle loading for the model. A maximum load of 100 kg will used in this study, as this is a scaled load which is a typical load for light weight vehicles. Moreover, the proposed size of EPS Geofoam for this study is 12.5 cm (L) x 6.6 cm (W) x 2 cm (T).

In order to prepare the soil samples in the model box, a proctor compaction test was done to obtain optimum moisture content and dry density of the marine clay. The soil conditions will follow the optimum moisture content to simulate conventional soil conditions. To maintain a consistent condition for all samples, the sample preparation of the soil in the soil box was done by compacting the marine clay by dividing it into three levels, where each level has a height of 8.34 cm (H) and is marked with a straight line using an indicator of a fine sandy soil. The loading test will be first carried out with a control test without using EPS blocks. The settlement is recorded by image analysis from indicators on the model box at these 3 levels of compaction as shown in Figure 5.

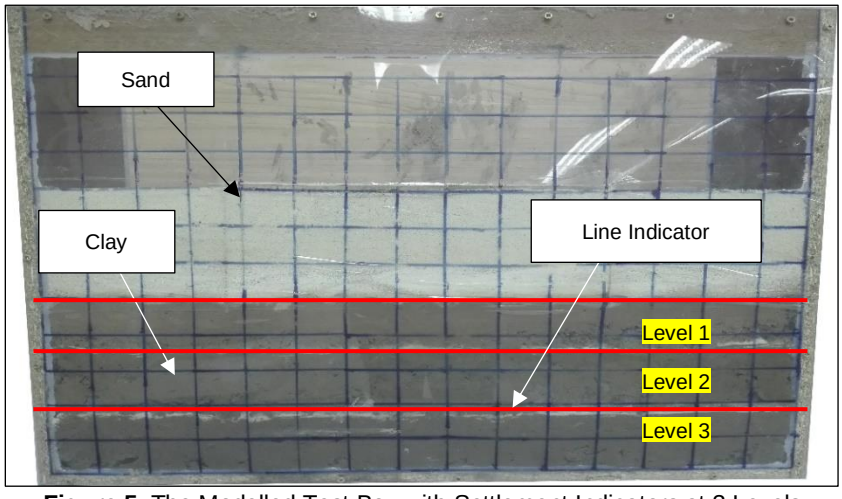


Figure 5. The Modelled Test Box with Settlement Indicators at 3 Levels

The first test is the control test as shown in Figure 6. A 15 cm height of sand overly the marine clay of 25 cm thickness. The second test for EPS model 1 will continue the load test with EPS Geofoam in the test box. The EPS Geofoam will be arranged until a height of 20 cm (H) following the scale of real site conditions as shown in Figure 7. Furthermore, the marked EPS Geofoam will be taken out before the slotted weight is placed at the top of the sandy soil. The settlement and pattern failure reading is then taken.

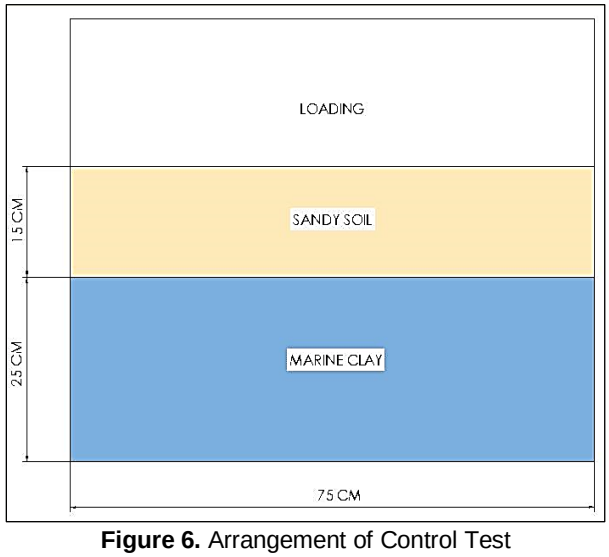


Figure 6. Arrangement of Control Test

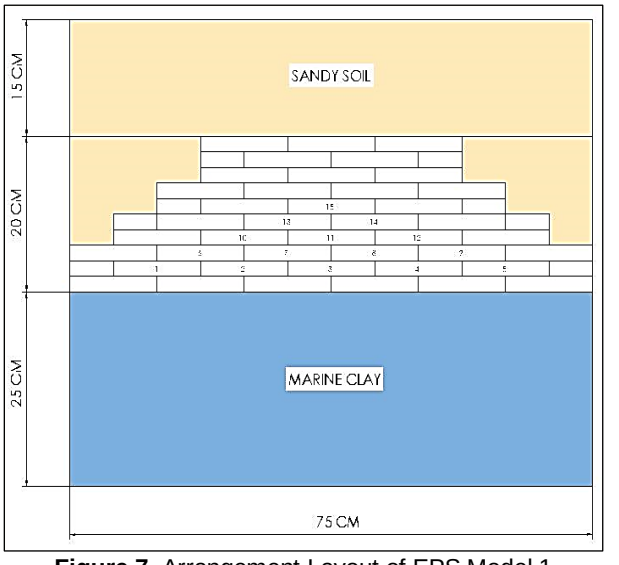


Figure 7. Arrangement Layout of EPS Model 1

The third test for EPS model 2 will continue the load test with an Arch type of EPS Geofoam in the test box. The arch form of EPS is placed in the centre of the test box. Furthermore, the diameter of the arch is 37.5 cm and the radius of the arch is 18.75 cm as shown in Figure 8. The semicircle EPS geofoam below the arch and the marked EPS Geofoam is taken out before placing the slotted weight at top of the sandy soil. After that, the settlement and pattern failure reading are taken.

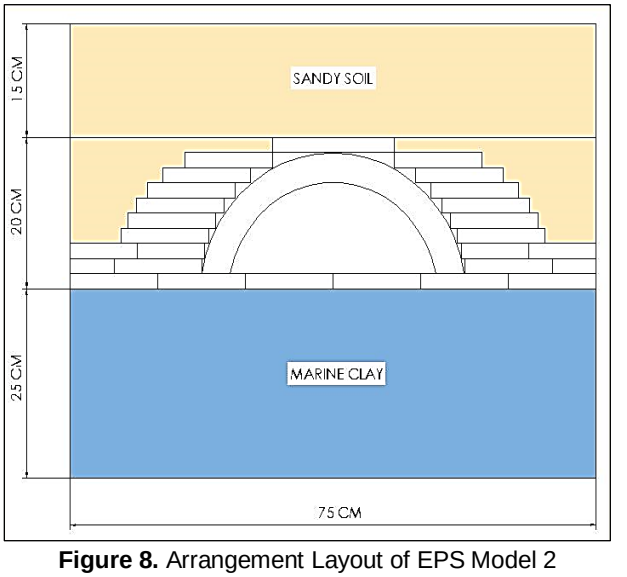


Figure 8. Arrangement Layout of EPS Model 2

RESULTS AND DISCUSSIONS

The result was discussed based on the settlement obtained from the loading test of marine clay model. The minimum settlement was selected for the best model for the real construction of the road embankment.

Results of Settlement of Height/Layer for Each Loading Test

The settlement analysis had been done by measuring the settlement from the actual level that have been marked on the test box. Figure 9 shows the graph of settlement reading with height versus level for 10 kg of loading test. From the graph the control test from the level 1 gives highest settlement which is 1.5 cm if compared with the control test level 2 and level 3. For the EPS Model 1 and EPS Model 2 there is 0 cm of settlement occurs at the 10 kg of loading test. It shows the marine clay at control test is very weak to settle compared to EPS Model 1 and EPS Model 2.

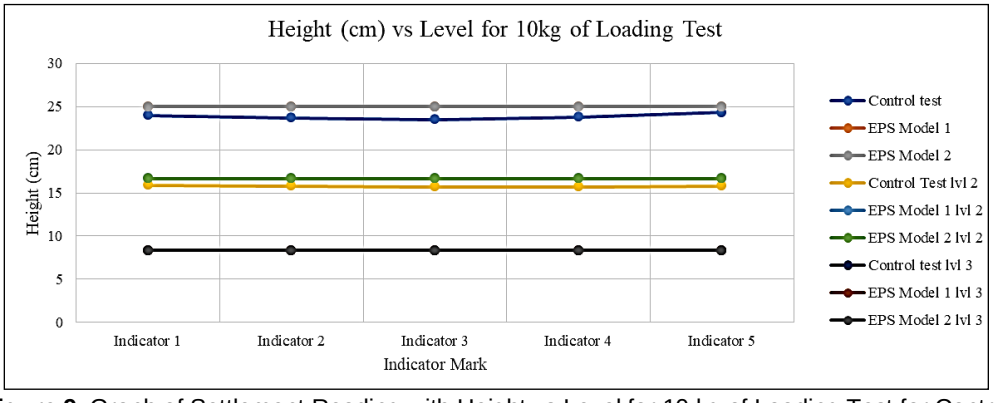


Figure 9. Graph of Settlement Reading with Height vs Level for 10 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 10 shows the graph of settlement reading with height versus level for 20 kg of loading test. From the graph the control test settlement from the level 1 gives highest settlement which is 1.6 cm if compared with the control test level 2 and level 3. For the EPS model 1 the settlement started at level 1 which is 0.5 cm at indicator 4 compared to level 2 and level 3 is still on the lower settlement. While for the EPS Model 3 there was 0 cm settlement occurs in 20 kg of loading.

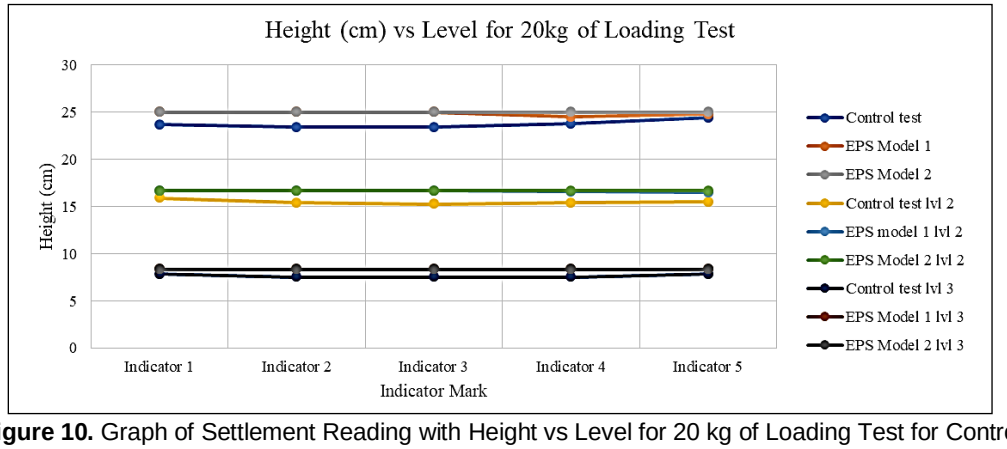


Figure 10. Graph of Settlement Reading with Height vs Level for 20 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 11 shows the graph of settlement reading with height versus level for 30 kg of loading test. From the graph the control test at level 1 shows the highest settlement among the

three level which is 1.8 cm at the indicator 3 and follow by level 2 and level 3. For the EPS Model 1 the highest settlement shows at level 1 indicator 4 which is 1 cm for 30 kg of loading. While for the EPS Model 2 there was still 0 cm of settlement for 30 kg of loading.

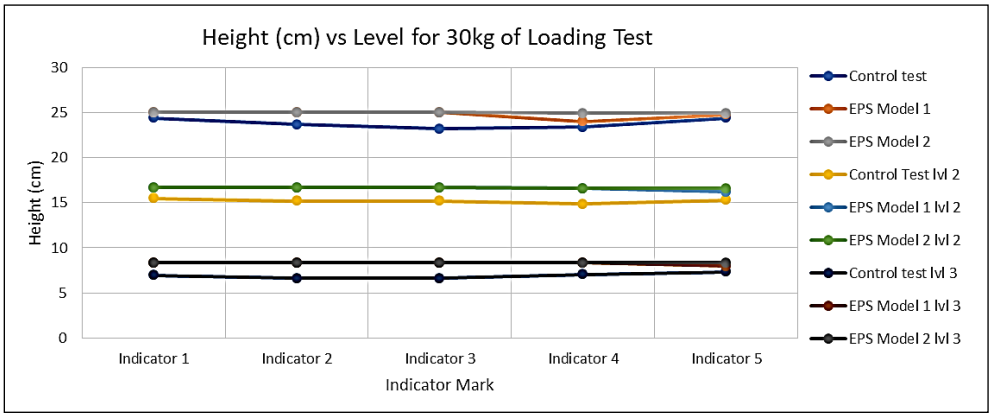


Figure 11. Graph of Settlement Reading with Height vs Level for 30 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 12 shows the graph of settlement reading with height versus level for 45 kg of loading test. From the graph the control test at level 1 gives the highest settlement which is 3.7 cm at indicator 4. This is the drastic settlement at 45 kg within 1 hour of loading test. For EPS Model 1 the highest settlement gives at level 1, indicator 4 again which is 1cm from the previous loading test. While EPS Model 2 shows the highest settlement at level 1, indicator 2 which is 0.2 cm for 45 kg of loading.

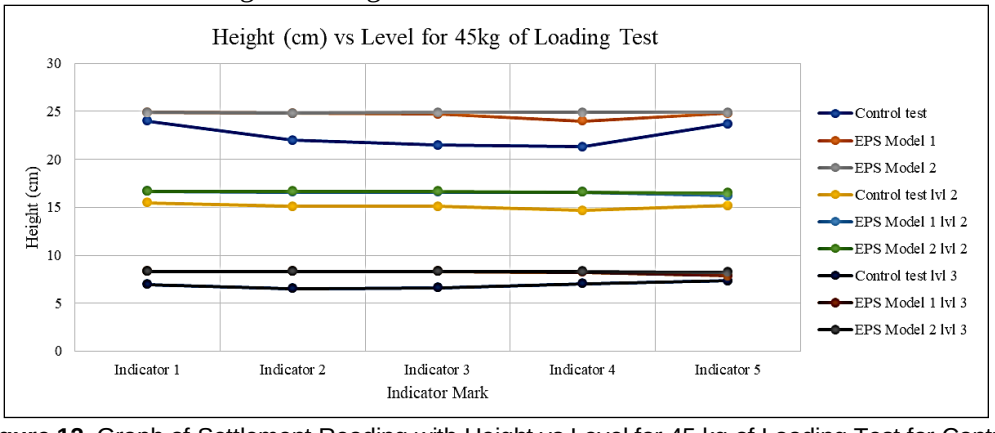


Figure 12. Graph of Settlement Reading with Height vs Level for 45 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 13 shows the graph of settlement reading with height versus level for 60 kg of loading test. From the graph the control test at level 1 shows the highest settlement among the three level which is 3.8 cm at indicator 4 for 60 kg of loading. For the EPS Model 1 the highest settlement shows at level 1 again at indicator 4 which is stay at 1 cm of settlement from original level. The higher settlement for EPS Model 2 at level 1 which is 0.3 cm at indicator 4.

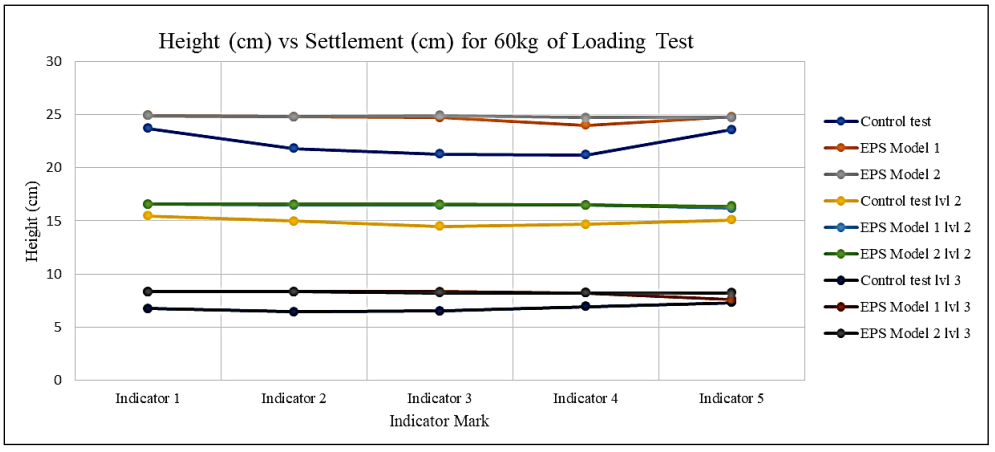


Figure 13. Graph of Settlement Reading with Height vs Level for 60 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 14 shows the graph of settlement reading with height versus level for 75 kg of loading test. From the graph the control test from the level 1 again gives highest settlement which is 3.8 cm at indicator 4 same as the previous loading test for level 1. For the EPS Model 1, the higher settlement shows at level 1, indicator 4 which is 1.3 cm more than previous settlement. While EPS Model 2 gives the highest settlement at level 1 and level 2 which 0.3 cm at indicator 4 and indicator 5 for 75 kg loading.

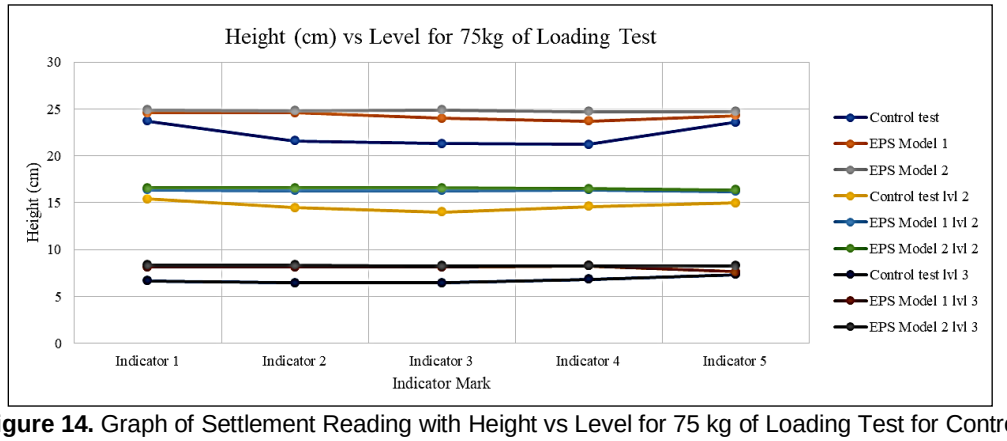


Figure 14. Graph of Settlement Reading with Height vs Level for 75 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 15 shows the graph of settlement reading with height versus level for 90 kg of loading test. From the graph the control test at level 1 gives the highest settlement which is 4.0 cm at indicator 3 and indicator 4. For the EPS Model 1 the highest settlement shows at level 1 which is 1.5 cm at indicator 2. EPS Model 2 gives the highest settlement at level 1 which is 0.3 cm at indicator 2 and indicator 4 for 90 kg of loading.

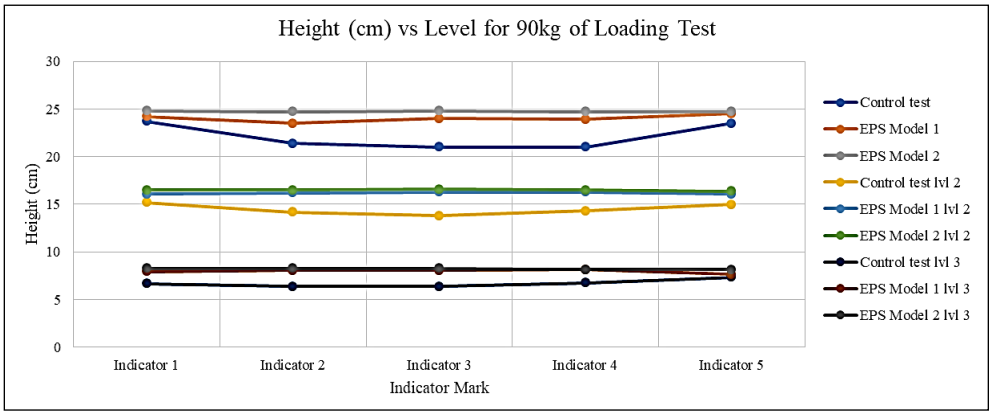


Figure 15. Graph of Settlement Reading with Height vs Level for 90 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Figure 16 shows the graph of settlement reading with height versus level for 100 kg of loading test. From the graph the control test at level 1 gives the highest settlement again which is 4.2 cm at indicator 3 and indicator 4. This is higher settlement that get from the level 1 for control test. For EPS Model 1 the highest settlement gives at level 1, which is 1.9 cm at indicator 2 increases about 0.4 cm from the previous loading test. While the highest settlement for EPS Model 2 shows at level 1, indicator 4 which is increase the settlement of marine clay about 0.2 cm from the previous loading test.

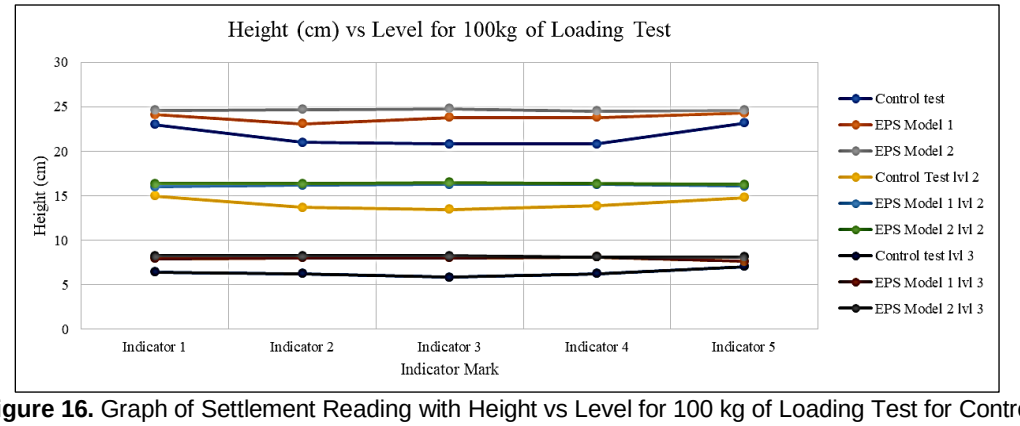


Figure 16. Graph of Settlement Reading with Height vs Level for 100 kg of Loading Test for Control Test, EPS Model 1 and EPS Model 2

Maximum Settlement at Each Loading Test

The maximum settlement readings for all loading tests with different loading and different physical model test are presented in this section. Figure 17 shows the graph of settlement reading for three physical model tests. From the graph, it shows the maximum settlement which is 4.2 cm for 100kg of loading taking place at control test. While the for the EPS model 1 and EPS model 2, the settlement of marine clay is much lower than the control test which is 1.9 cm and 0.5 cm for the maximum loading test.

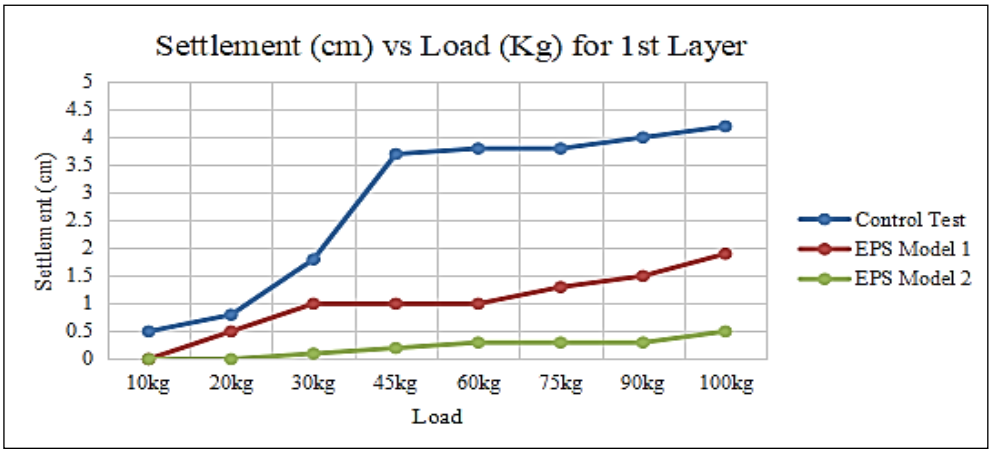


Figure 17. Graph of Settlement Reading for 1st Layer of All Physical Model Test

Figure 18 shows the graph of settlement reading for three physical model test. From the graph, it shows the maximum settlement that taking place at control test which is 3.2 cm for 100kg of loading. Furthermore, the graph shows the continuation of increasing settlement for the control test for 2nd layer of marine clay. It also show the lower settlement of marine clay for EPS model 1 and EPS model 2 which is 0.6 cm and 0.4 cm of 100kg loading.

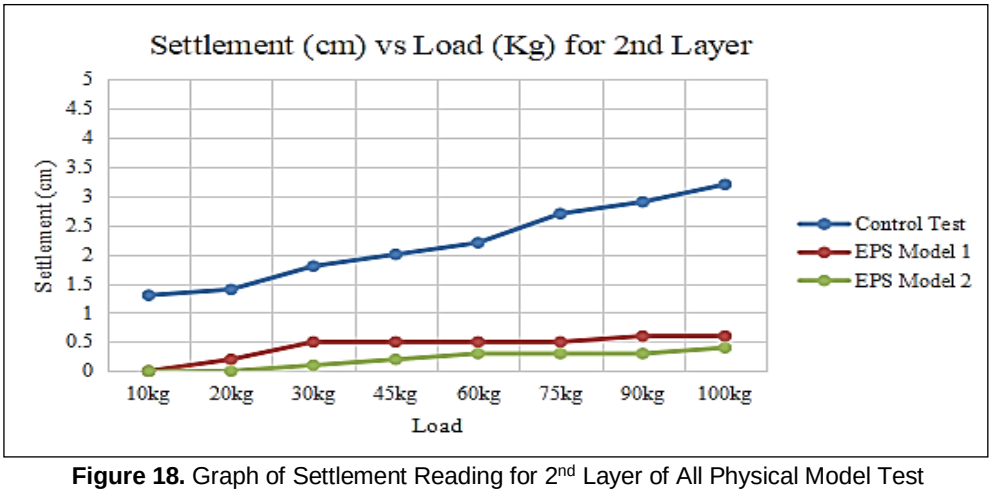


Figure 18. Graph of Settlement Reading for 2nd Layer of All Physical Model Test

Figure 19 shows the graph of settlement reading for three physical model test. The graph shows the light settlement of marine clay for the 3rd layer. The maximum settlement in control test taking place at 100kg loading which is 2.5 cm. Furthermore, the EPS model 1 and EPS model 2 maintain the 0 cm of settlement reading at 10kg to 20kg of loading test. The maximum settlement for both of EPS model 1 and EPS model 2 is lower than control test of 1st layer which is 0.7 cm and 0.2 cm for 100kg of loading.

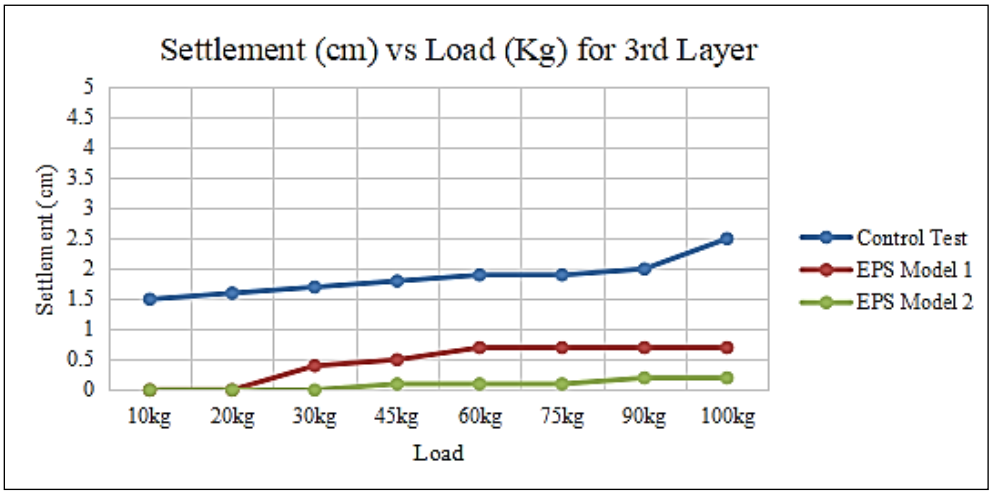


Figure 19. Graph of Settlement Reading for 3rd Layer of All Physical Model Test

DISCUSSION

The settlement of marine clay in bottom of the arch structure EPS Geofoam from the layout of EPS model 2 with 100 kg of slotted weight loading test gives the lowest settlement reading among the three models that tested with slotted weight loading test on top of sandy soil. This shows that the arch structure EPS formation gives the best results in reducing settlement. This is due to the extra light weight of the structure where less materials is used for the structure. The arch structure also gives the best structural integrity for the embankment.

CONCLUSION

As the result, it shows the EPS model 2 gives the lowest settlement rate for the 100 kg of slotted weight. The result was expected that the arch structure using lightweight structure can withstand with heavy weight as tested in the laboratory. Moreover, the arch structure has a capability to withstand with overburden weight of building and bridges, as settlement is reduced and at the same time it is able to give a high structural integrity. An arch can traverse between two purposes of vertical support than a straight beam. The utilization of EPS ordinarily makes an interpretation of into an advantages to development the construction schedules and lowers the overall cost of construction because it is easy to handle during construction works. As a conclusion, it can be conclude that the expanded polystyrene (EPS) geofoam using an arch structure design is a better material to be used as a new concept of lightweight fill material construction in embankments due to its low settlement behaviour on for the marine clay soil.

ACKNOWLEDGEMENT

The authors would like to acknowledge Universiti Tun Hussein Onn Malaysia (UTHM) and the Ministry of Education (KPM) for the financial support for this study under the Fundamental Research Grant Scheme (FRGS) with the grant number FRGS / 1 / 2017 / TK10 / UTHM / 02 /12.

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PHYSICAL PROPERTIES OF BLENDED CEMENT CONCRETE CONTAINING PALM OIL FUEL ASH AND EGGSHELL POWDER

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Abstract
This paper aims to present the results of an experiment incorporating palm oil fuel ash (POFA) and eggshell powder (ESP) to partially replace ordinary Portland cement (OPC) in concrete-making. Both POFA and ESP are waste material that are widely available in Malaysia and similar palm oil-producing countries. The discussion focuses on the density, the water absorption, and the ultra-pulse velocity (UPV) of six concrete mixes to interpret the quality and strength of the final concrete specimens. All mixes had been designed to reach 30 MPa concrete strength after 28 days. Besides the control group, which contained 100% OPC, the rest contained varying amount of POFA, ESP or both to replace up to a maximum of 20% of total cement weight. the results were promising, whereby both water absorption and UPV had improved while the density maintained within the normal range. The advancement of this technology is envisioned to drive down construction cost due to the re-use of waste material and contribute towards sustainability efforts through carbon footprint reduction. However, to achieve these, more research and development are required to better understand the physical, chemical and mechanical properties of these waste material as well as to optimize its deployment at a commercial scale.

Keywords: *Palm oil fuel ash, eggshell, density, water absorption, ultra-pulse velocity.*

INTRODUCTION

Concrete is a widely used construction material in Malaysia and most developing countries. However, concrete cement releases carbon dioxide (CO_2) and greenhouse gases into the atmosphere during its manufacturing. This can be addressed by substituting cement with waste material such as palm oil fuel ash (POFA), eggshell powder (ESP), fly ash (FA), rice husk ash (RHA) and silica fume (SF).

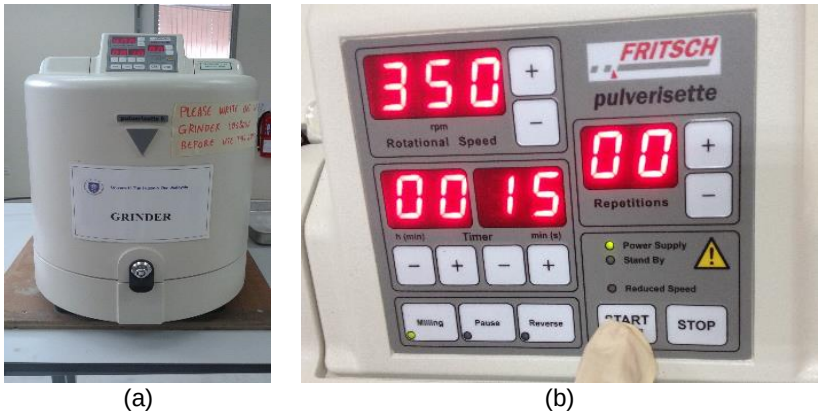
This paper intends to discuss the incorporation of POFA and eggshell powder (ESP) as cement substitution (Vasoya and Varia, 2015). POFA has slow pozzolanic reaction due to its large particle size and porous structure, but this can be improved through the grinding process. The most favorable attribute is its silica content, which is 45 to 60% of its total content and the most important for developing sustainable concrete (Sata, Jaturapitakkul and Kiattikomol, 2007; Chindaprasirt, Rukzon and Sirivivatnanon, 2008; Tangchirapat, Jaturapitakkul and Kiattikomol, 2009; Jaturapitakkul, Tangpagasit, Songmue and Kiattikomol, 2011; Kroehong, Theerawat, Chai and Chindaprasirt, 2011; Karim, Zain, Jamil and Lai, 2011; Lim et al., 2015). Eggshell, on the other hand, is a common food waste with known good strength characteristics when mixed with concrete. Eggshell powder contains a very pure form of calcium carbonate (CaCO3) (Ing and Choo, 2014; Jamellodin et al., 2017; Mohamad, Mahmood, Yik and Abdul Rasid, 2018) and its incorporation into concrete also decreases the setting time. The ESP is produced by grinding eggshell into fine powder and sieving it to the required size (Gowsika, Sarankokila and Sagunan, 2014; Ujin, Ali and Harith, 2016; Balamurugan and Santhosh, 2017; Jamellodin et al., 2018; Abd Khalid et al., 2019).

This paper focuses on the chemical composition and physical properties of concrete incorporated with POFA and ESP. Although ESP is rich in calcium and analogous to limestone, it is still a waste material so there is a need to understand its physical and mechanical properties. The parameters which will be discussed include the density, water absorption, ultrasonic pulse velocity and also the development of hydration temperature of the concrete.

MATERIALS AND EXPERIMENTAL PROGRAM

Materials Preparation

The Palm Oil Fuel Ash (POFA) was obtained from a factory in Johor, the southern state of Malaysia. The collected POFA was dried in an oven at 100°C ± 5°C for 24 hours and then sieved through a 300 µm sieve to remove the debris and extraneous materials such as unburned fibres and shells. After that, it was grinded as shown in Figure 1(a) to obtain the fineness similar to Ordinary Portland Cement (OPC). Figure 1(b) shows the grinder setup, programmed to grind for 5 minutes with 350 run per minute (rpm). Following the ASTM C618 (2015) requirement, the ground POFA was grinded down to have 97% passing through sieve number 325 (45µm). Figure 2(a) and (b) depict a comparison between industrial POFA and the ground POFA.



(a) (b)

Figure 1. (a) Grinding Machine and (b) Grinding Setup



(a) (b)

Figure 2. (a) POFA at Factory and (b) Ground POFA

The ESP were obtained by grinding the eggshell from bakeries, restaurants and stalls. The collected eggshells were cleaned with tap water and then dried to remove the remnants as in Figure 3(a). Then, the eggshells were oven-dried at 100°C ± 5°C for 24 hours and cooled for an hour. After that, the dried eggshells were grinded to obtain the ESP as shown in Figure 3(b). The grinder's setup and passing grade were similar to POFA's.

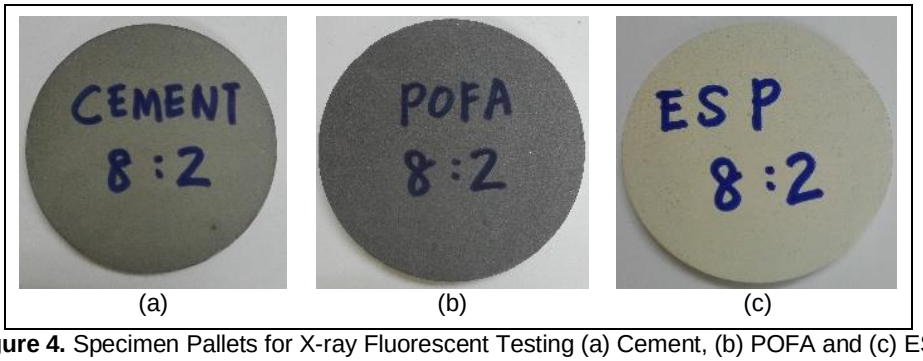


(a) (b)

Figure 3. (a) Cleaned and Dried Eggshells and (b) Ground ESP

X-Ray Fluorescence (XRF)

X-ray Fluorescence (XRF) analysis was performed to assess the chemical composition of the OPC, POFA and ESP. The Licomax C Micro Powder PM was used to bind the pozzolanic material together, and weighed about 20% of the sample's total weight to produce a very strong pellet that could be dropped from half a meter above ground level without breaking. The ratio of pozzolanic material to the Licomax Powder was kept constant at 8:2. Once the sample and the glue were mixed, the mixture was pressed in a die using a hydraulic sample press at a pressure of 15 x 10^4 N/m^2. Figure 4 depicts the final product.



(a) (b) (c)

Figure 4. Specimen Pallets for X-ray Fluorescent Testing (a) Cement, (b) POFA and (c) ESP

Mix Design Proportions

Six groups of concrete mix designs containing OPC, POFA and ESP were prepared. Each group contained two sets of sample and each set contained three (3) specimens. This means a total of 36 specimens were prepared. Each specimen measured 100mm x 100mm x 100mm. The specimens were tested after 7 and 28 days of curing. The weight of the specimen was

calculated using the Department of Environment (DoE) method. All specimens contained OPC (density = 450 kg/m^3) mixed with water at a ratio of 0.55. The fine and coarse aggregates were fixed at 1030 kg/m^3 and 809 kg/m^3, respectively.

Among the six groups of concrete mixes, the first was reserved as the control sample with 100% OPC. Group 2 and 3 had 20 % OPC replaced with POFA and ESP, respectively. The remaining three groups had varying proportion replaced with POFA and ESP, as shown in Table 1. All concrete specimens had been designed to reach 30 MPa concrete strength after 28 curing days.

Table 1. Mix Design Proportion of OPC, POFA and ESP

No.	Mix design	OPC (C) (%)	POFA (P) (%)	ESP (E) (%)
1	C100	100	-	-
2	C80P20	80	20	-
3	C80E20	80	-	20
4	C80P10E10	80	10	10
5	C80P15E5	80	15	5

Density of Concrete

Before the concrete specimens were submerged in water for curing, the mass of each specimen was weighing and recorded. Weighing was conducted again after seven and 28 days of curing according to BS EN 12390-7: 2009 requirement. Three readings were taken for each specimen and an average was obtained.

The density of the concrete is an important physical property that influences its mechanical performance. A denser concrete generally provides higher strength, fewer voids and thus, lower porosity. The smaller the voids are, the lesser the permeability to water and other soluble elements. The density of normal concrete is normally about 2,400 kg/m^3.

Water Absorption Test

The water absorption test was conducted based on BS1881-122 (2011) requirement. Firstly, the specimens were oven-dried for 72 hours, removed from the oven, and then cooled for 24 hours. After that, the specimens were weighed and then immersed in water with its longitudinal axis horizontal and at a depth of 25 mm below water surface, as shown in Figure 5. The cubes were left immersed for 30 minutes before being removed and towel-dried completely. Finally, the specimens were weighed again.



Figure 5. Cubes Immersed in Water

Ultra-Pulse Velocity (UPV) Test

The ultrasonic pulse velocity test (UPV) was carried out in accordance with ASTM C597 to determine the propagation velocity of the longitudinal stress waves pulsing through the concrete specimens. The UPV test was conducted using the PROCEQ Testing Instrument and followed the requirement of BS EN 12504-4, as shown in Figure 6. This is the arrangement which gives the most reliable results because the signal strength is the strongest.



Figure 6. The Apparatus Set Up of UPV in Accordance with ASTM C597 for Direct Transmission Method

RESULTS AND DISCUSSIONS

Chemical Composition

The chemical composition of POFA showed that 53.3% of the total content was made up of silica dioxide (SiO2), followed by 9.2% of calcium oxide (CaO). Silica is an important ingredient needed in concrete-making for strength development, but excessive amount of silica tends to prolong the setting time. As a comparison, Table 2 shows the chemical composition of OPC, POFA and ESP. A point to highlight is that POFA has the highest silica content while ESP has the highest calcium oxide content.

Table 2. Chemical Composition of OPC, POFA and ESP

Chemical Composition	Ordinary Portland Cement, OPC (%)	Palm Oil Fuel Ash, POFA (%)	Eggshell Powder, ESP (%)
Silica dioxide (SiO_2)	16.40	53.30	0.05
Magnesium Oxide (MgO)	1.22	4.10	1.12
Calcium Oxide (CaO)	69.60	9.20	98.00
Potassium Oxide (K_2O)	0.67	6.10	0.11
Iron Oxide (Fe_2O_3)	3.70	1.90	0.02
Zinc Oxide (ZnO)	-	-	0.01
Aluminum Oxide (Al_2O_3)	4.70	1.90	0.05
Phosphorus Pentoxide (P_2O_5)	-	2.40	0.1
Sulphur Trioxide (SO_3)	2.65	-	0.5

Density of Concrete

Table 3 shows the density of each specimen after seven and 28 days of curing. This is graphically presented in Figure 7 for better understanding. The density of all groups containing POFA, ESP or both had higher density than the control group after seven days of curing. After 28 days, only the group containing purely POFA (C80P20) and that with 15% POFA and 5% ESP (C80P15E5) had lower density than the control group. All groups also depicted increased density with time except for the same two groups as aforementioned. The decrease in density was attributed to higher amount of entrapped water in POFA that had been dispersed over time. Nevertheless, the average density of all specimens maintained within the range close to a normal concrete, which was between 2306 kg/m^3 and 2350 kg/m^3.

Table 3. The Average Weight and Density of Concrete Specimens

No.	Mix design	7 Days		28 Days	
		Average Weight (kg)	Average Density (kg/m^3)	Average Weight (kg)	Average Density (kg/m^3)
1	C100	2.31	2313.33	2.33	2326.67
2	C80P20	2.34	2340.00	2.32	2323.33
3	C80E20	2.33	2326.67	2.35	2350.00
4	C80P10E10	2.32	2323.33	2.35	2353.33
5	C80P15E5	2.35	2350.00	2.38	2380.00
6	C80E15P5	2.33	2326.67	2.31	2306.67

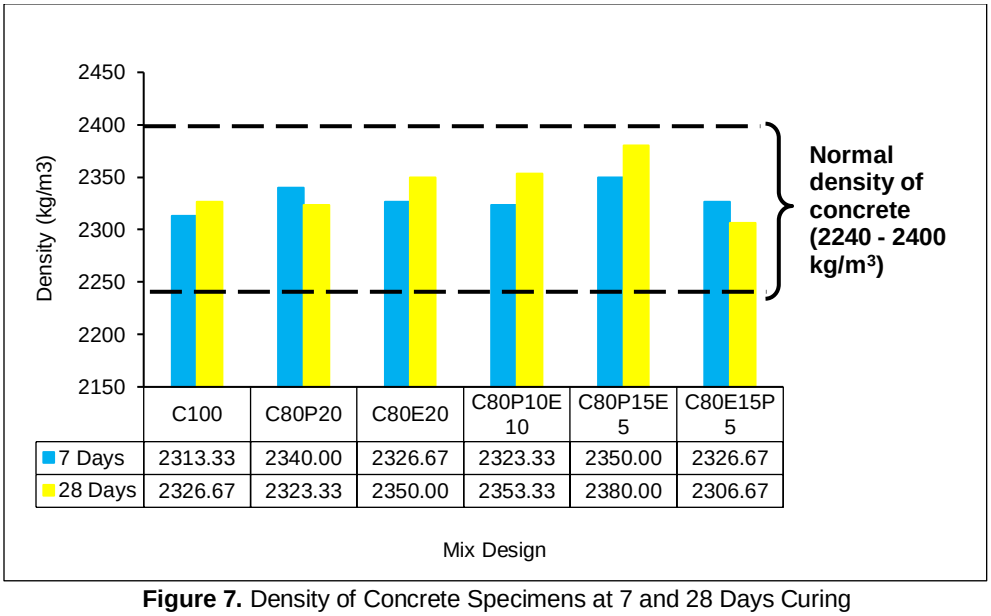


Figure 7. Density of Concrete Specimens at 7 and 28 Days Curing

Water Absorption

The water absorption of all specimens are shown in Table 4 and graphically presented in Figure 8. The test was conducted on hardened concrete after 28 days of water curing and oven-dried at 110ºC for 72 hours. The results showed that C80E15P5 (15% ESP and 5% POFA) had the highest water absorption in general, recorded at 3.03% after 30 minutes and 3.94% after 60 minutes. The C80P10E10 and C80P15E4 groups had water absorption

consistently at 2.62% and 2.02%, respectively. The water absorption for all groups are considered low and with most groups having lower absorption than the control group except for C80P15E5. The water absorption of concrete is an important indication of its strength, as higher water absorption depicts more porous concrete and thus, lower strength.

Table 4. Water Absorption of Concrete Specimens After 28 Days of Curing

No.	Mix Design	Water Absorption (%)	
		30 minutes	30 minutes
1	C100	2.69	2.69
2	C80P20	2.39	2.39
3	C80E20	2.06	2.06
4	C80P10E10	2.62	2.62
5	C80P15E5	2.02	2.02
6	C80E15P5	3.03	3.03

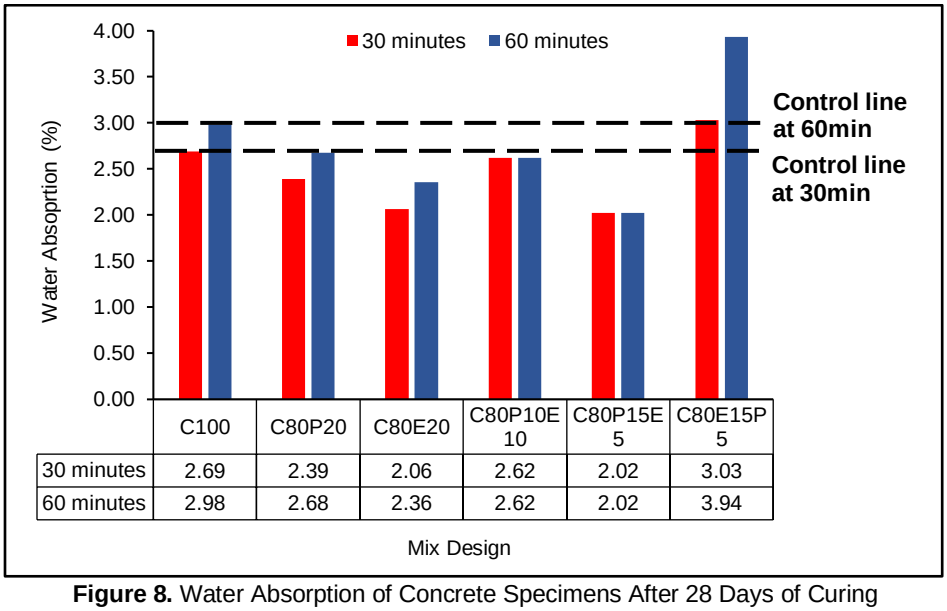


Figure 8. Water Absorption of Concrete Specimens After 28 Days of Curing

Ultrasonic Pulse Velocity (UPV)

The UPV was calculated as a function of the time taken for a pulse to travel through a concrete specimen. The value gives a good gauge of a concrete's quality, uniformity, strength, density and general physical condition. Table 5 shows the average UPV in km/s for all specimens after seven and 28 days of curing. Table 6 depicts the corresponding quality of the concrete based on UPV as specified in IS13311-Part 1. The higher the UPV, the better the quality because, if a concrete has cracks or internal imperfections, the pulse will take a longer time to travel through. The quality, in this sense, is closely related to the material; the mix proportion; and the method of placing, compacting and curing the concrete. Based on Table 5, the average UPV for all specimens are between 3.68 km/s and 4.11 km/s. This means that the concrete specimens are of good quality regardless of its OPC, POFA and ESP composition. The specimens with 20% ESP generally had the highest quality although its UPV slighted decreased from 4.09 km/s to 3.96 km/s. The C80P15E5 group had the lowest UPV during both test periods.

Table 5. Average UPV for All Mix Designs at 7 and 28 Days

No.	Mix Design	Average Ultra Pulse Velocity (km/s) 28 Days	
		7 Days	28 Days
1	C100	3.95	3.86
2	C80P20	4.11	3.82
3	C80E20	4.09	3.96
4	C80P10E10	3.99	3.76
5	C80P15E5	4.08	3.80
6	C80E15P5	3.74	3.68

Table 6. Concrete Quality Grading (IS13311-Part 1, 1992)

No.	Pulse Velocity Cross Probing, (km/s)	Concrete Quality Grading
1	>4.5	Excellent
2	3.5 – 4.5	Good
3	3.0 – 3.5	Medium
4	<3.0	Doubtful

(Source: IS13311-Part 1, 1992)

CONCLUSION

Incorporation of both POFA and ESP in concrete to partially replace OPC is a relatively new method to produce sustainable concrete. The results presented in this paper clearly showed that the method is promising based on an improvement in the density, water absorption percentage and UPV of the concrete specimens. The density remained within the normal range of a concrete, but the water absorption had remarkably reduced while the UPV had generally increased when compared to the control group.

Both POFA and ESP are some readily available waste material in Malaysia and similar palm oil-producing countries. As such, more research and development are encouraged to further optimize the technology at a commercial scale and contribute towards sustainability efforts to reduce carbon emissions.

ACKNOWLEDGEMENT

The author is grateful to all the technical staff of the laboratory and members of the Faculty of Civil Engineering and Built Environment, Universiti Tun Hussein Onn Malaysia (UTHM) for their support and cooperation.

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ENERGY ABSORPTION OF CONCRETE CONTAINING WASTE CRUMB RUBBER AND RICE HUSK ASH (RHA) UNDER IMPACT

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Abstract

Recently, some researchers investigate the idea to reduce the impact during an accident between vehicles with concrete barriers. In this research, the modified rubberized concrete as a flexible concrete barrier that was utilizing waste materials under impact loading was studied. The waste CR (Crumb Rubber) and RHA (Rice Husk Ash) were used as a replacement for fine aggregate and cement which 6% and 10% of waste CR with 10% RHA replaced in the normal concrete mix. This study experimentally examines the compressive strength of the concrete cube containing different percentages of CR with a combination of 10% of RHA. The replacement of CR as fine aggregate with 0%, 6%, and 10% yielded compressive strength results in about 52.3 MPa, 34.3 MPa, and 28.6 MPa respectively. From the cube test result, the optimum percentage was chosen, and block specimens were prepared. All block specimens were tested under an impact test with 4 m/s velocity. The results reveal the penetration depth for block specimens are 3.41 mm, 6.88 mm and 8.85 mm for 0%, 6% and 10% of CR respectively. The penetration depth shows the increasing rubber content in the concrete mix causing high penetration depth. In this study, 6% of CR was considered as a good result since the crater diameters closed to the nose hemispherical projectile diameter (less percentage). In terms of energy absorption, the result was directly proportional to penetration depth. Thus, rubberized concrete able to improve the function of concrete as a concrete barrier in reducing impact since it shows high energy absorption.

Keywords: *Crumb rubber; rice husk ash; energy absorption; impact load.*

INTRODUCTION

In concrete technology, additional of other materials in a concrete mixture was widely exposed. The addition of other substances in the concrete mix as an innovation material could increase the concrete performance and resolve its weaknesses. The characteristic of normal concrete is too rigid and can be dangerous for the road user when an accident happened. Nowadays, the road barrier extendedly used as a separation of a road to prevent any potential of a head-on collision between vehicles and prevent vehicles from any hazardous passage. However, the existing road barrier somehow can be the reason for bad injury and fatality on the road due to the hard material and the design, which may cause high impact during the collision. The accident on the road, which causes a collision with a metal barrier, was highly contributed to the fatality compare to the concrete barrier (Gabler, 2007).

Data collection from the police, government authorities, and national/ international research institutes show that between the ASEAN countries, Malaysia was in the top range of road fatality risk (per 100,000 population) with 50% of the fatality's accident contributed by motorcyclists (Abdul Manan & Varhelyi, 2012). Besides, a motorist that ride on the road with

speed around 60 km/h and collide with road barrier will have a high potential of fatal injury although the barrier is concrete, steel or wire rope (Grzebieta et al., 2009).

On the other hand, our world has also faced many environmental issues related to the crisis of disposing rubber and local waste product at a landfill. The solid waste material such as used rubber tires is difficult to compact and not easy to decompose at the disposal site. Further, waste rubber tires become a habitat for mosquitoes and rodents that finally could contribute to dangerous disease such as dengue, cholera, etc. While local waste product amount at the disposal site also keeps on increasing due to the increase of population. As a result, it reduces landfill space. National Ministry of Urban Wellbeing, Housing and Local Government said that the production of waste every day in Malaysia is in the average of 30,000 tons (Yahya, 2007).

Based on the above-highlighted issue, waste rubber and local waste material known as Rice Husk Ash (RHA) were proposed as a replacement material in the concrete mixture which replaces some percentage of fine aggregate and cement. Rubber as an elastic material has the potential to prohibit damage during a collision by absorbing the energy generated from the impact force (Topcu, 1995). However, the presence of rubber in concrete may reduce the concrete strength as compared to the normal concrete. Due to this disadvantage, RHA can be a medium to increase the concrete strength by replacing cement with a suitable percentage into the concrete mixture (Yahya, 2018).

Besides, this idea can help in reducing the bulk of waste in landfills especially for daily waste and non-degradable waste since rubber, and local waste product are two (2) types of waste that are commonly dumped into landfill areas. Sustainable usage of these two (2) materials in the concrete mixture were investigated under impact conditions. Many researchers such as Mushunje et al. (2018), Elchalakani (2015) and Meikandaan (2016) conducted studies on these materials as well. However, no details study has been carried out on research for energy absorption due to penetration depth and crater diameter. Thus, this study will experimentally examine the efficiency of proposed rubberized concrete under the condition of the impact load.

METHOD OF PREPARATION AND TESTING

Generally, cement was replaced about 10% with RHA while fine aggregate was replaced about 6% and 10% with waste CR. These specimens experienced a curing process and were tested after 28 days which the concrete would be near-optimal strength and unlikely to shift any further after 28 days of curing it. This is important to obtain the compressive and impact behaviour results of rubberized concrete. The RHA preparation includes a) the process of oven-dry RHA with the temperature of 105°C in 24 hours period, b) the process of crusher/ grind RHA in Los Angeles Abrasion Machine, c) sieved RHA passing 45-micron sieve, d) weighed RHA to achieve 10% from cement replacement. The CR was taken from the rubber factory which the size in the range of 0.5 mm to 3.0 mm diameter, where this size was a suitable size to mix into the concrete mix compare with other sizes (Yilmaz & Degirmenci, 2009). Figure 1 (a) and Figure 1 (b) show the partial replacement materials used for the experimental works.



Figure 1 (a). CR Rubber



Figure 1 (b). Sieved RHA

Concrete Mix Design And Specimen Size

This research was based on the Design of Normal Concrete Mix by DOE method with a target of strength after 28 days about 40 N/mm2. The cube size is 0.1 m × 0.1 m × 0.1 m, cylinder size 0.2 m (height) × 0.1 m (diameter) and the block size is 0.3 m × 0.3 m × 0.2 m were prepared accordingly. In this study, waste CR and RHA acted as a replacement of fine aggregate and cement respectively which affect the normal proportion of main ingredients in the concrete mix design. The procedures to prepare the concrete sample includes a) preparation of mould/ formwork, b) dry ingredients were added into the mechanical concrete mixer and left for a few minutes, c) water added into the mixer and poured into mould/ formwork, d) vibrator machine was used to remove air trapped. Table 1 shows the quantity of RHA and CR in each sample.

Table 1. Quantities of RHA and CR

Weight of RHA/Rubber (kg)			
Percentage of RHA / CR / **Sample**	**10% (RHA)**	**6% (CR)**	**10% (CR)**
Cube	0.15	0.17	0.28
Cylinder	0.28	0.31	0.53
Block	2.84	3.26	5.43

Impact Test And Energy Absorption Test Procedure

All rubberized concrete blocks were tested under impact test using hemispherical nose projectile (40 mm diameter) as an impactor. The load was 80 kg and its drop height about 0.8 m. The test also conducted with a free-falling load (Latif et al., 2012; Shahrul et al., 2018) with a velocity of 4 m/s as shown in Figure 2 (a) and Figure 2 (b). These weight, height and velocity were used due to the ability of it to produced impact on concrete so that it is easy to compare the penetration depth produced on each specimen.

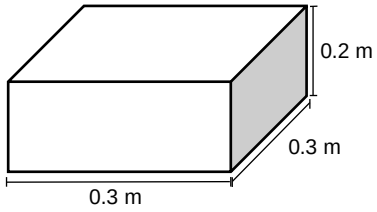


Figure 2 (a). Concrete Block Dimension

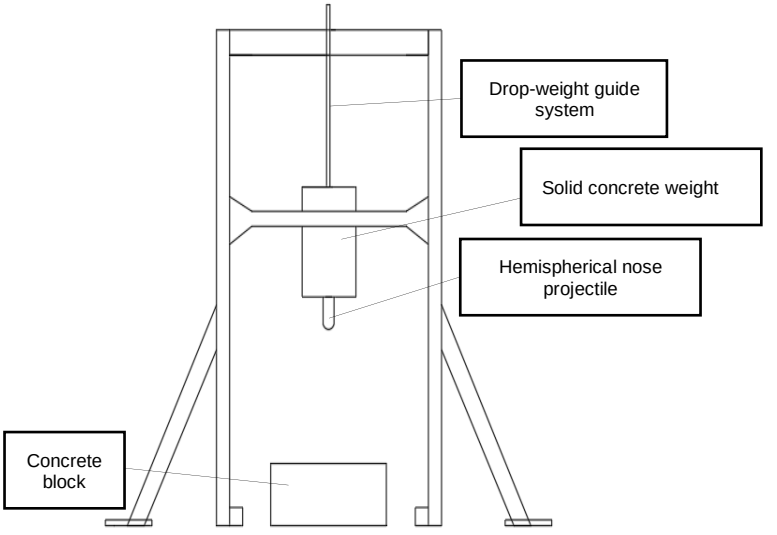


Figure 2 (b). Impact Test

This test was significant to investigate the behaviour of rubberized concrete that illustrated a real case during accident occurrence. From this test, penetration depth and cratering diameter can be observed clearly. This can help in analysing the amount of energy transfer towards the concrete material. The depth of impact can be calculated by using a thread and Vernier calliper (Hadipramana et al., 2017). Energy absorption can be calculated by using the existing empirical formula by Hadipramana et al. (2016), which shown in Equation (1).

$$E_a = m.g.X \quad (1)$$

where:

Ea - energy absorption
m - mass of impactor (kg)
g - acceleration due to gravity at 9.81 m/s2
X - penetration depth

RESULTS AND DISCUSSION

The compressive strength test for cube specimens and tensile strength test were recorded and discussed comprehensively, in which the discussion focuses on the strength of concrete. From the optimum result of the compressive strength test, block specimens were prepared based on 30 MPa as an optimum value, whereby, exceeds the Public Work Department standard/specification. The standard concrete barrier is grade 30 and above (Standard Specification for Road Works, 1988). Besides, the result for impact test that concern on penetration depth, cratering diameter as well as failure mode were also studied accordingly. Moreover, from the result, the energy absorption has been calculated, compared and analysed comprehensively.

Compressive Strength Test

Results for the compressive test of the cubes after 28 days are shown in Figure 3. The test was conducted using Universal Testing Machine (UTM) followed BS EN 12390-3:2009 standard. It can be observed that the presence of 6% CR in the concrete mix has decreased the result of compressive strength of about 34%. Besides, further increasing the percentage of CR, about 10% replacement in the fine aggregate has gradually decreased the strength of concrete approximately 45%. From the result, it can be concluded that the existence of rubber in the concrete mix cause diminished of the compressive strength at 28 days. Therefore, 6% replacement of fine aggregate with CR considered acceptable since the compressive strength was 34.3 MPa which exceeds than 30 MPa as a standard requirement. For 10% replacement of CR, the result was slightly lower than 30 MPa (recorded approximately only 28.6 MPa) which was not considered suitable to be applied as a concrete barrier. However, it is expected that the result of a 10% replacement of CR will be further increased up to 30 MPa after 28 days (Ettu et al., 2013).

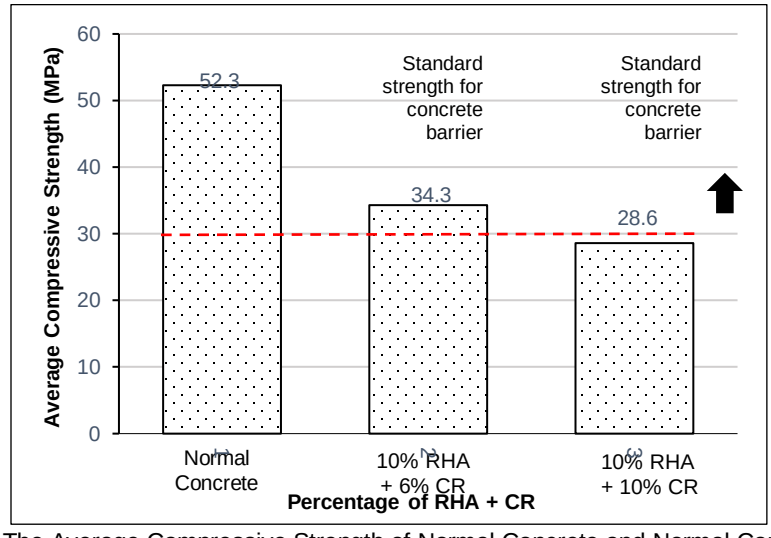


Figure 3. The Average Compressive Strength of Normal Concrete and Normal Concrete with Replacement of RHA and CR After 28 Days

It can be said that the replacement of CR in the concrete mix does not contribute to increasing the result of compressive strength. Most of the researchers also agreed that CR only causes decreasing in compressive strength results (Su et al., 2015); (Isa et al., 2014); (Mavroulidou & Figueiredo, 2010). This is due to the physical characteristic of the rubber itself which low water absorption associated with low in specific gravity. This characteristic caused less bonding of CR and other concrete ingredients. Besides, weak bonding occurs between cement paste, RHA, aggregate, and rubber. It creates void and trapped air between them. RHA consist of silica contains that influence the strength in acted as pozzolanic (Alanemea & Adewalea, 2013). The different locations will produce a different percentage of silica Chandrasekhar et al. (2003) depends on the location of crop, temperature, and duration of burning. The increasing of compressive strength strictly relies on the percentage of silica in RHA. In this study, it can be said that the quantity of silica in RHA possibly influenced the compressive strength. However, the compressive strength of rubberized concrete can increase slightly. It can also be summarized that the leading cause of a reduction in compressive

strength is due to the different kind of properties of rubber and fine aggregate. Hence, this is the reason for the reduction value of compressive strength after the replacement of CR and RHA for fine aggregate and cement, respectively.

Split Tensile Test

Since tensile strength is weak in the tensile region, hence it is interesting to compare the result of strength between the normal concrete and rubberized concrete (concrete containing CR). The split tensile test results of the cylinder after 28 days are shown in Figure 4. The test followed BS EN 12390-6:2009 standard. Based on the result on the 28th day, the tensile strength of concrete was decreased with increment of rubber replacement. Concrete containing 6% and 10% of CR were decreased the strength significantly. For 6% of CR, the percentage differences were about 23% if compared to normal concrete. Further observation reveals that 10% of CR decreased by approximately 34% of the normal concrete. Hence, it can be concluded that the application of CR in a concrete mix reduces the tensile strength of concrete significantly.

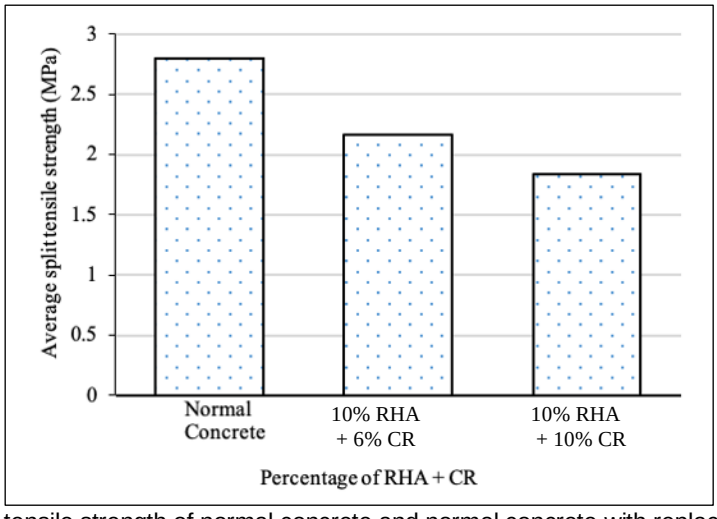


Figure 4. Average tensile strength of normal concrete and normal concrete with replacement of RHA and CR after 28 days

From the test, the result of split tensile strength shows similar trends with compressive strength results. One of the reasons for the dropped result was due to the poor bonding between rubber and cement paste (Elchalakani, 2015). Hence, weak bonding interaction of rubber and concrete is the leading potential cause. Rubber is low in specific gravity (Yilmaz & Degirmenci, 2009). Therefore, it can be the reason for the lightweight rubber has a high potential to lift during the vibration process. This is called as non-homogeneous concrete which rubber moves on the top layer of the concrete mix that causes a high intensity of rubber in the upper area. In other words, the rubber spread unevenly across the concrete layers. Overall, it can be observed that the rubber does not benefit in increasing the strength in the tensile zone.

Impact Test

In this test, the concrete mix for the block specimen was designed to achieve 30 MPa and above, with 6% of CR and 10% of RHA as fine aggregate and cement replacement, respectively. Further, 10% of CR with 10% of RHA sample was also prepared based on the compressive strength results. All plain concrete block specimens were tested under 4 m/s impact velocity. Table 2 and Figure 5 show the result of the penetration depth of each block specimen which was measured accurately using a Vernier calliper. From this test, the effect of penetration depth and cratering phenomenon were discussed in details.

It can be seen that 6% of CR in concrete gives 67% difference in term of penetration depth as compared to normal concrete, while 10% of CR in concrete produce 89% deeper penetration depth as compared to control specimen. The trend of the graph clearly shows that the rubber give benefits in term of higher penetration rate. Since rubber has a popular character which is useful in terms of energy absorption, hence it influences the concrete to act flexibly and reduced the concrete rigidity. During impact event on rubberized concrete, plastic energy is increased, and the elastic energy decreased (Topcu, 1995). In this state, the maximum depth will be achieved due to the impact load. Generally, the result of penetration depth was influenced by compressive strength results. Overall, higher compressive strength, the shallow penetration depth can be produced. Logically, high compressive strength (rigid concrete) able to withstand load effectively compare with low compressive strength and vice versa. Hence, this is the contribution to the penetration depth pattern, as shown in Figure 5. In this case, CR and RHA in concrete play a vital role in slight reducing the concrete strength. However, its presence can produce flexible concrete where this characteristic is suitable for concrete road barriers.

Table 2. Results of penetration depth due to impact test

Sample	Avg. Compressive Strength (MPa)	Block Samples	Penetration Depth (mm)	Avg. Penetration (mm)
Normal concrete	52.3	1	2.79	3.41
		2	4.08	
		3	3.36	
10% RHA + 6% CR	34.3	1	6.86	6.88
		2	7.33	
		3	6.45	
10% RHA + 10% CR	28.6	1	6.16	8.85
		2	7.93	
		3	12.46	

* Avg. = Average

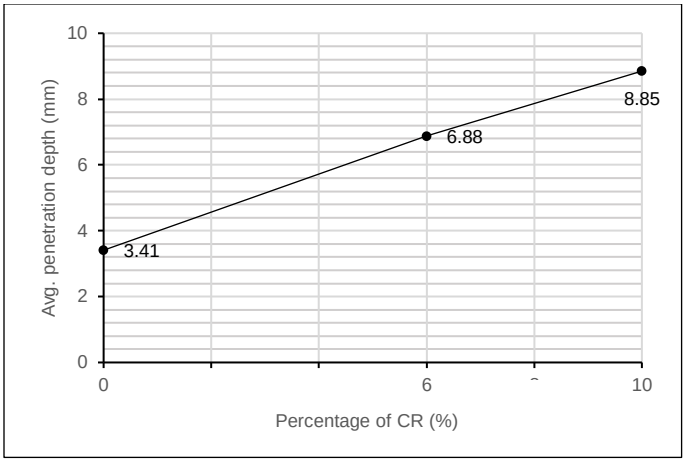


Figure 5. Average Penetration Depth on Different Percentage of Crumb Rubber

Rubberized Concrete Behaviour (Cratering) Under Impact

The cratering pattern for three different samples of block specimens are shown in Figure 6 (a) - (c). This failure mode is essential to be taken into account for further investigation on the behaviour of rubberized concrete under the impact.



Figure 6 (a). Cratering of Control Specimen (0% CR + 0% RHA)



Figure 6 (b). Cratering of 10% RHA + 6% CR Specimen

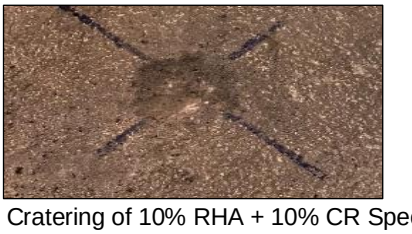


Figure 6 (c). Cratering of 10% RHA + 10% CR Specimen

Based on Figure 6 (a) - (c), it can be seen that there is no scabbing or perforation in the tested concrete block. Overall, the block specimens show crater failure beneath the impact zone. Batarlar (2003) define the cratering failure mode as local damage. Further, the vibration

due to the collision is the leading cause of the deformation as agreed by Sallam et al., (2008). Since the produced energy unable to be eliminated, it will transfer the energy to the nearest and contact area. The result of the crater on plain concrete occurs due to strain capacity that has been extended to rubberized concrete.

The diameter of cratering failure of the plain concrete block after exposed to 4 m/s impact loads are tabulated in Table 3 (a) - (c). To calculate the equivalent diameter as recorded in Table 3 (a) – (c), the crater diameters of all concrete blocks were initially measured at four (4) different points (refer Figure 7). This method is utilized to observe the average diameter accurately (Iqbal & Rajput, 2017). In this study, the diameter of the hemispherical nose projectile is 40 mm. The crater diameters were then compared comprehensively with nose projectile diameters. The best cratering behaviour was achieved where its diameter is nearly 40 mm (nose projectile diameter). Less different percentage (crater with nose projectile diameter) can be defined as an ideal crater diameter. For normal concrete block, the differences between crater and nose projectile diameter achieve 52%. Meanwhile, the specimen of 10% RHA with 6% of CR shows the less different percentage, approximately 1.3% only. Finally, for 10% of RHA with 10% of CR produces about 19.3% different percentage.

Table 3 (a). The Diameter of Cratering (Control Specimen)

Specimen	Sample	Diameter (mm)	Equivalent Diameter (mm)	Average Crater Diameter (mm)
Normal Concrete	1	$D_1 = 100$ $D_2 = 38$ $D_3 = 55$ $D_4 = 61$	63.5	60.8
	2	$D_1 = 51$ $D_2 = 82$ $D_3 = 76$ $D_4 = 53$	65.5	
	3	$D_1 = 47$ $D_2 = 61$ $D_3 = 57$ $D_4 = 49$	53.5	

Table 3 (b). The Diameter of Cratering (Replacement of 6% CR)

Specimen	Sample	Diameter (mm)	Equivalent Diameter (mm)	Average Crater Diameter (mm)
10% RHA + 6% CR	1	$D_1 = 40$ $D_2 = 35$ $D_3 = 37$ $D_4 = 45$	39	40.5
	2	$D_1 = 41$ $D_2 = 37$ $D_3 = 34$ $D_4 = 4.5$	39	
	3	$D_1 = 45$ $D_2 = 42$ $D_3 = 47$ $D_4 = 40$	43.5	

Table 3 (c). The Diameter of Cratering (Replacement of 10% CR)

Specimen	Sample	Diameter (mm)	Equivalent Diameter (mm)	Average Crater Diameter (mm)
10% RHA + 10% CR	1	$D_1 = 45$ $D_2 = 45$ $D_3 = 47$ $D_4 = 47$	46	47.7
	2	$D_1 = 52$ $D_2 = 48$ $D_3 = 54$ $D_4 = 47$	50.3	
	3	$D_1 = 47$ $D_2 = 45$ $D_3 = 48$ $D_4 = 47$	46.8	

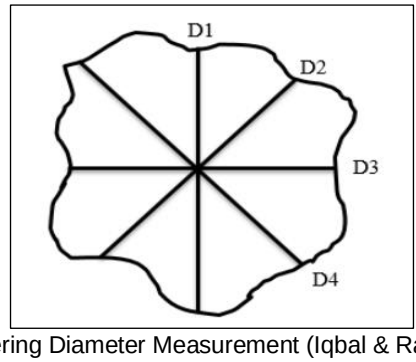


Figure 7. Cratering Diameter Measurement (Iqbal & Rajput, 2017)

Overall, it can be observed that plain concrete with 6% of CR and 10% of RHA gives the less different percentage. The rigid characteristic makes concrete incapable of deflecting or of being forced out of shape since the concrete is inflexible. Due to the rigidity characteristic, this produces concrete low in terms of plastic behaviour. Therefore, to sustain the load, the energy of plain concrete propagates from the impact area to the side area of the concrete block. This phenomenon makes the cratering diameter wider. From Table 3 (c), it shows that the 10% RHA with 10% CR crater diameters was wider than nose projectile diameter. Although 10% of CR in concrete is the best result in terms of penetration depth, however, wide crater diameter was observed. This situation reflects the capability of concrete to perform well under sudden load.

From the visual observation, all the concrete blocks experience local damage as recorded in Table 4. The failure behaviour for both plain concrete and rubberized concrete generated penetration and cratering under impact load test. For normal concrete block, spalling failure has been observed from the impact test, as shown in Figure 8. Some of the concrete fragments on the impact zone bounce out from the target area. This same failure condition also presented by Latif et al., (2012) and Batarlar, (2003). This situation caused the crater diameters wider than the nose projectile diameter. It is important to highlight that this failure only occurred on normal concrete block (rigid concrete).

Table 4. The Impact Failure Mode

Sample	Penetration Depth (mm)	Local Effect	Other Effect
Normal Concrete	3.41	Cratering	Spalling
10% RHA + 6% crumb rubber	6.88	Cratering	-
10% RHA + 10% crumb rubber	8.85	Cratering	-



Figure 8. Impact Effect on Control Specimen (Spalling)

Energy Absorption

The energy absorptions were calculated by utilizing equation (1) to complete the final objective of the study. The parameter, such as a mass of impactor and penetration depth, were employed by multiplying with gravity to obtain the energy absorption amount. The value of energy absorption can be referred to as Table 5. The energy absorptions amount are also plotted in Figure 9. The result from Table 5 and Figure 9 show that the penetration depth result influenced the energy absorption result. Both of the values were directly proportional to each other. Factually, increasing of CR in concrete was drive the concrete to behave flexibly because of rubber to cement composition (Kamal et al., 2009). Besides, the flexibility was closely related to energy absorption effectiveness which also related to the modulus of elasticity. This elastic behaviour generates high energy absorption compared with plain concrete during the impact event.

Table 5. The Energy Absorption on Plain and Rubberized Concrete

Sample	Mass of Impactor (kg)	Penetration Depth (mm)	Energy Absorption (Nm)
Control	80	3.41	2.68
10% RHA + 6% CR	80	6.88	5.40
10% RHA + 10% CR	80	8.85	6.95

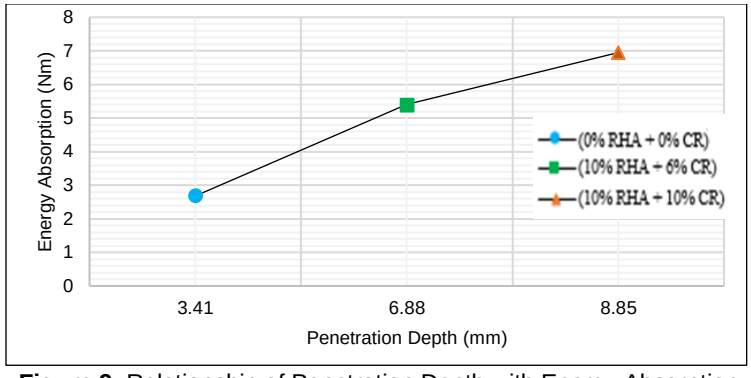


Figure 9. Relationship of Penetration Depth with Energy Absorption

From the line graph that plotted in Figure 10, the trend of graph showed that the compressive strength was inversely proportional to the energy absorption. This result proved that the compressive strength alters the ability of concrete to absorb energy. The involvement of pozzolanic like RHA helps a lot in controlling the reduction of strength due to the replacement of fine aggregate with CR. Therefore, the rubberized concrete was not loss its compressive strength too much but at the same time able to deformed which good in energy absorption during sudden impact. This situation is shown in 10% replacement of cement with RHA, and 6% replacement of fine aggregate with CR, which 34.3 MPa compressive strength was causing the energy absorption became 5.40 Nm.

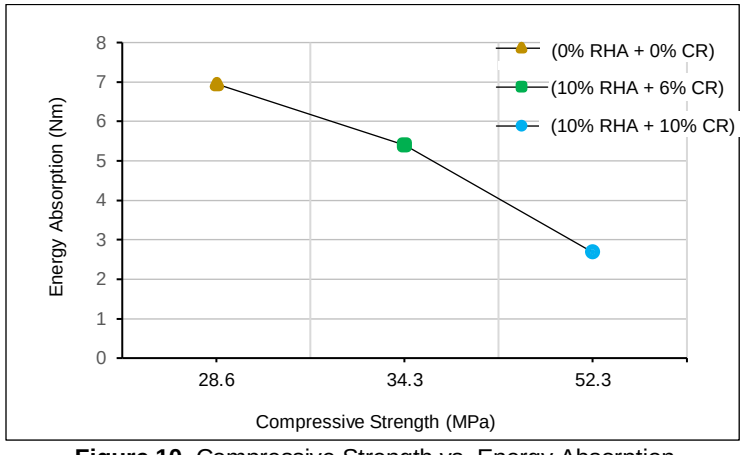


Figure 10. Compressive Strength vs. Energy Absorption

Moreover, it clearly shows that high compressive strength does not indicate excellent energy absorption. The compressive strength for normal concrete specimens about 52.3 MPa causing low energy absorption, which is only 2.68 Nm. Furthermore, for control specimen (normal concrete), high compressive strength concrete indicates brittle material which does not deform a lot and experience sudden breaking during the deformation. Low energy absorption does not help the concrete to manage and withstand the mechanism of failure in concrete. Hence, in a real case during accident occurrence on the road between vehicle and rigid concrete barrier, the vehicle will experience the worst damage since the concrete struggle to hold back itself from the sudden load. Different from rubberized concrete, the ductile and elastic characteristics help the concrete to experience more deformation before it breaks and reduced vehicle damage during an unexpected collision.

Rubberized concrete makes the concrete surface softer that acted as a cushion during impact. Rubber is a light material that makes it easy to lift during the vibration process of concrete mixing. This condition makes the rubber that mixed devoted to the surface of the concrete since it is less dense. This condition causes the strength decrease due to the uneven distribution of rubber but at the same time gave benefits towards the concrete flexibility during impact which the concrete surface was covered with rubbers (Euniza et al., 2014). In terms of concrete plastic and elastic deformation, rubberized concrete which shows high in energy absorption result was experience great plastic properties than elastic properties. This state makes the rubberized concrete fully utilising its plastic state during that period that makes it unable to recover it back (Topcu, 1997).

CONCLUSION

Based on the analysis and discussion, several conclusions can be made:

- The result of compressive strength test and split tensile strength test show a downward trend of the result.
- Increasing of CR content makes the concrete low in term of strength since the rubber has many different physical characteristics compare to fine aggregate.
- RHA was no contributed much in increasing compressive strength due to the percentage of silica.
- Replacement of 6% CR with 10% RHA was the optimum percentage since the compressive strength was more than 30 MPa (JKR standard).
- Penetration depth result shows that the increasing of CR contributed towards the increase of penetration depth.
- 10% CR with 10% RHA show the deeper values of penetration depth.
- Crater diameters for 6% CR with 10% RHA almost near the nose projectile diameter. Meanwhile, normal concrete specimen and 10% CR with 10% RHA were quite different with projectile diameter.
- The impact zone showed local damages of penetration for all samples, and spalling existed on plain concrete.
- The result was closely related to concrete rigidity for plain concrete and increasing of plastic characteristics in rubberized concrete.
- Energy absorption was linearly proportional to the penetration depth.
- Increasing CR contains was gave good results for energy absorption.
- Rubberized concrete which shows high in energy absorption result, was experienced excellent plastic properties than elastic properties.
- Increasing of plastic energy capacities gives high deformation at the time before a fracture occurs, which can reduce collision effect significantly.
- Rubberized concrete is more suitable to be applied as road concrete barrier than conventional concrete barrier due to high ability as impact resistance.
- Rubberized concrete with 30 MPa above with 10% RHA and 6% CR has high potential to be employed as a concrete barrier on the road in reducing damages of vehicles and fatality rate during accident occurrences since it shows satisfaction results in term of mechanical properties and energy absorption.

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ENGINEERING PROPERTIES OF HIGH STRENGTH FIBRE REINFORCED CONCRETE USING STEEL AND POLYPROPYLENE FIBRES

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Abstract
Plain or unreinforced concrete is characterized by its low tensile strength, low strain capacities and brittle in nature. The tensile strength of plain concrete is considered lost once cracking occurred. Discrete short fibre reinforcement is being considered to be used for structural applications since it can reduce cracking phenomena, improve ductility and failure mode. Therefore, the aim of this research is to investigate the engineering properties of reinforced concrete containing a combination of steel and polypropylene fibres. In this research, the physical and engineering properties of fibre reinforced concrete (FRC) are investigated by using hooked end steel fibre (SF) and virgin fibrillated polypropylene fibre (PPF). Laboratory testing program is conducted to identify the physical properties of the fibres. Then, FRC samples were tested to determine the engineering properties including flexural strength, tensile splitting strength, and compressive strength. The engineering properties and structural performance are determined based on the optimum percentage using high strength concrete grade C60 to simulate concrete strength of sample manufactured at the factory. The results indicated that the best combination of fibres was for concrete containing SF 75% + PPF 25%. The combination of SF and PPF fibres in concrete was able to enhance the engineering properties and controlling the growth of cracks in concrete. The results also indicated that concrete with both SF and PPF produced higher tensile and flexural strengths as compared with the control by 77% and 170%, respectively.

Keywords: *Concrete crack; fibre reinforced concrete; polypropylene fibre; steel fibre.*

INTRODUCTION

Plain or unreinforced concrete can be characterised as having inferior tensile strength, inferior strain capacities plus very brittle (inferior ductility). Plain concrete normally has a random distribution of fine and coarse aggregate particles throughout the cement matrix (Banthia & Nandakumar, 2003*;* Bazant, 2001; Chanh, 2005; Chen, 1995). It normally goes through a quasi-brittle failure. As a result, these characteristics limit the application of plain concrete in construction industry. These limitations can be solved by including small amount of randomly distributed short fibres. Short fibres, to some extent, can be used to remedy the weakness of plain concrete such as its low tensile strength, brittle behaviour and low durability.

The reinforcement used in concrete typically consists of continuous main reinforcing bars, transverse reinforcement bars and shear links. These should be designed properly according to the related design code to prevent uncontrolled crack development (Bentur & Mindess, 1990). Since short fibres are randomly distributed in the concrete compared to normal reinforcing bars, fibres were expected to perform better in controlling cracks in the

concrete matrix (Bentur & Mindess, 2007). Thus, while conventional reinforcement bars are used in enhancing the concrete load bearing capacity, the fibres are more effective for crack control.

Fibre Reinforced Concrete (FRC) can be defined as cement-based composite material that is reinforced with discrete and randomly distributed short fibres. Fibres can be made of various dimensions, shapes and functions that are produced from steel, synthetic, glass, and natural materials. Fibre reinforcement is now being used for structural applications due to its ability to reduce cracking development, improve ductility and failure mode, and to some extent improve the durability of reinforced concrete. The composite material of this behavioural is far superior to that of the plain concrete. The fibres were extended across the developing cracks during the propagation stage to provide few post-cracking ductility. It will enable the FRC to carry remarkable stresses over a relatively large strain capacity in the post-cracking stage, if the fibres are adequately strong and well bonded in the concrete.

A more recent development in fibre reinforced cement-based technology of the hybrid fibre system is according to the performance optimisation in several material. The hybridization of steel and polypropylene fibres is expected to show enhancement in the concrete properties due to better mechanical bond strength between the fibres and the cement matrix that can delay the formation of micro-cracks and macro cracks.

In this research, polypropylene and steel fibres are selected to be used based on the understanding that the synergistic combination of non-metallic and metallic fibres in the mixture of concrete will perform better to bridge crack and reduce shrinkage strain. At present, there are quite limited number of studies have been conducted by using two different type of fibres in concrete mixture as secondary reinforcement to mitigate macro-cracks. This study and paper focus on assessing the mechanical properties of high strength fibre reinforced concrete containing steel and polypropylene fibres.

RELATED PREVIOUS STUDY

It is understood that plain concrete is weak because of its brittle characteristics which possessed low tensile strength, restricted ductility, low strain capacity and small immunity to cracking (Khan *et al.*, 2016; Mammeri *et al.*, 2005; American Concrete Institute, 2002). Weak tensile strength combined with brittle behaviour can result in sudden tensile failure under loading. This is obviously not desirable for any construction material. Thus, concrete requires some form of tensile reinforcement to cater up for its brittleness and improve its tensile strength and strain capacity for structural applications. Traditionally, steel bar is used for main tensile reinforcement in concrete and had to be placed where high tensile stresses occurred and as per design requirement in concrete structures (Khan & Rizvi, 2013).

Steel Fibre

Various types of steel fibres (SF) had been produced and available in the market. Many researchers stated that the types of SF depend greatly on the geometry of the individual unit of fibre (Sarbini, 2014; Katzer & Domski, 2012). The steel fibre may be manufactured with prismatic, irregular cross section, collated group of fibres and other forms of geometry. Prismatic geometry is for SF with rounded or polygons cross-section and has smooth surfaces.

It may also be manufactured with deformed shape throughout its length or only at its ends. Irregular geometry means that the cross-section varies along the SF length, while the bundled group of fibres are manufactured in such a way that it could be separated during the mixing process. The common types of SF available in the market are shown in Figure 1.

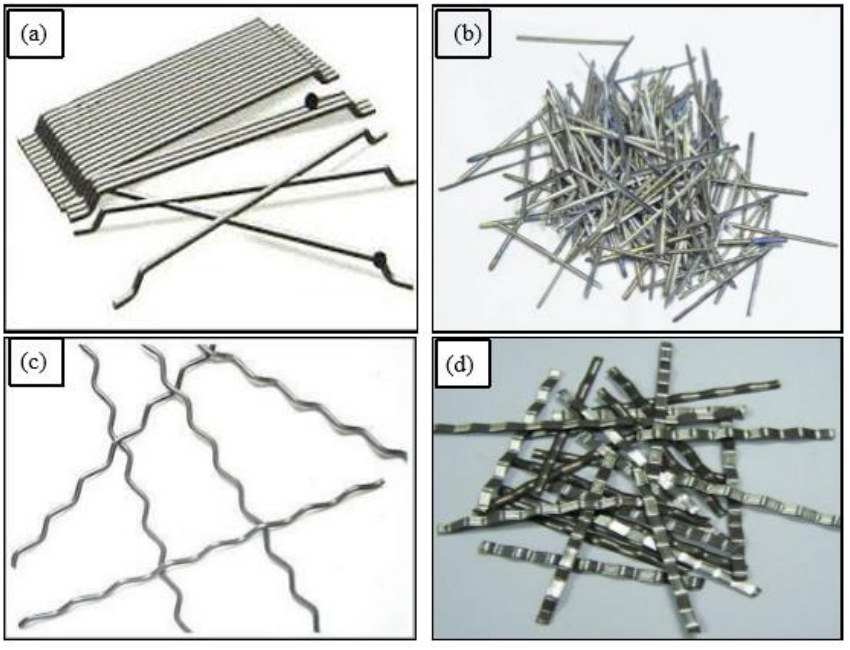


Figure 1. Types of steel fibre available in the market (a) Hooked end, (b) Straight, (c) Crimped, (d) Corrugated (Sarbini, 2014)

Polypropylene Fibre

Synthetic fibres are used initially as the construction material for reinforcement of cementitious materials in the 20th century. According to a report, synthetic fibre includes aramid, acrylic, nylon, carbon, polyester, polyethylene and polypropylene (American Concrete Institute, 2002). In recent years, polypropylene fibre is used in a large scale in enhancing concrete properties as the successful commercial application to reinforce concrete (Patel et al., 2012).

Typically, polypropylene fibre is manufactured with three different geometries such as monofilament, film, and extruded tape in which the last two forms are used extensively for concrete reinforcement (Bentur & Mindess, 2007). Some of the advantages of polypropylene fibre include its high melting point (165 ºC), alkali resistance and low cost of raw material. It was also claimed that polypropylene fibre has unique properties which are chemically inert and non-corrosive (Patel et al., 2012). In addition, it is also a hydrophobic material which is zero water absorption and can easily be evenly distributed in the concrete mixture (American Concrete Institute, 2002).

On the other hand, polypropylene fibre (PPF) has some weakness points such as its sensitivity to oxygen and sunlight, poor fire resistance, low Modulus of Elasticity, and poor bonding with the concrete matrix. Although various mixing methods had been studied, the

most successful mixing method is to disperse the fibres directly into the mixer (tumbler or pan mixers). In order to achieve effective performance, the suggested amount of polypropylene fibres is 0.9 kg/m3, about 0.1% by volume (Madhavi et al., 2014; Saketh et al., 2017). The common types of polypropylene fibres available in the market are shown in Figure 2.

Synthetic fibre is produced from materials that can endure for long-term in alkaline environment of concrete. It can be produced from synthesized polymers. The raw materials such as petroleum-based chemical or petrochemical are the compounds that are utilized to produce these fibres (Sivakumar, 2011). Polypropylene fibre is manufactured in various sizes and shapes, and with different properties.

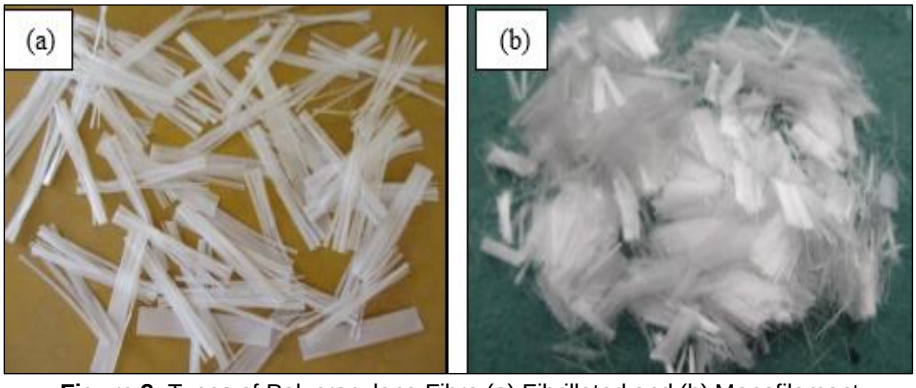


Figure 2. Types of Polypropylene Fibre (a) Fibrillated and (b) Monofilament

Orientation and Distribution of Fibre

In most of the research work conducted, it is agreed that the randomly distributed fibres improved the tensile characteristic of concrete. According to a researcher, the orientation of fibre plays a significant role in the mechanical properties of fibre reinforced composites (Lofgren, 2005). The technology used during mixing provides for direct and random orientation of fibres in the concrete. Direct orientation, as shown in Figure 3(a) – Figure 3(h), is realized mainly by using continuous filaments, plates, fabrics and non-fabric nets, or by special production techniques like pre-placing the fibres.

Body-random orientation is characterized by unlimited (free) distribution of short fibres throughout the body of the concrete (in three-dimensional space). The angles of inclination of the fibres relative to the surface of the component ranging from 0° to 90°, as long as the dimensions of the component, in all directions, exceed the length of fibres considerably. The smaller the cross-section, the more restricted the possibilities of the free orientation of the fibres. However, it should also be noted that for FRC there are a number of other factors influencing the fibre orientation and distribution apart from purely geometrical considerations such as the method of placement, the equipment used (e.g. pumping), and the properties of the fresh concrete.

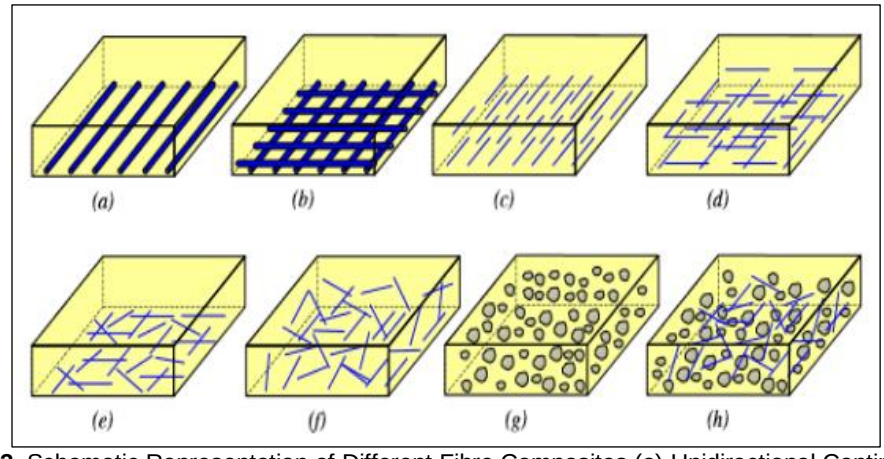


Figure 3. Schematic Representation of Different Fibre Composites (a) Unidirectional Continues; (b) Bi-Directional Continuous; (c) Discontinuous with Biased 1-D Fibre Orientation; d) Discontinuous with Biased 2-D Fibre Orientation; (e) Discontinuous with Plane-Random Orientation; (f) Discontinuous with Random Fibre Orientation; (g) Particulate Composite (Particle Suspension); and (h) Fibre-Reinforced; and Particulate Composite (e.g. Fibre-Reinforced Concrete) (Lofgren, 2005).

MATERIAL AND METHOD

Steel (SF) and Polypropylene (PPF) fibres used in this study are supplied by Oriental Housetop Sdn Bhd. The density test result is utilized to compute the amount of fibre needed in the mixture of concrete for each batch.

Material and Mix Proportion

The materials used to produce concrete are Ordinary Portland Cement, crushed coarse aggregate, fine aggregate, and water as shown in Figure 4. The characteristic strength of concrete is designed for 60 N/mm^2 at 28 days with the slump in the range of 60 – 180 mm. Superplasticizer is also added to improve the workability of the concrete.



Figure 4. Materials for concrete (a) coarse aggregate; (b) fine aggregate; (c) cement; and (d) water

Mix Design

In this study, concrete specimens were cast and used to determine the strength and performance of concrete with SF and PPF. The concrete mix is generally specified by its required performance in terms of strength grade. Strength testing usually formed an essential part of the assessment of conformity to the specification. Based on the Department of Environment (DoE) mix design, Ordinary Portland Cement (OPC) with water-to cement ratio of 0.38 and with 60 -180 mm slump was used in the design of concrete mix for a targeted strength of 60 MPa. Fine and coarse aggregates used are graded river sand and 10 mm crushed granite, repectively as coarse. The sieve analysis test of the aggregates is carried out in accordance with the relevant standard (British Standards Institution, 2003).

The type of superplasticizer used is GLENIUM ACE 389 and in reference to the guideline for type F admixture (British Standards Institution, 2001; American Society for Testing and Materials, 2017). It is formulated to produce rheoplastic concrete that can flow easily while maintaining high plasticity. Table 1 and Table 2 show the properties of SF and PPF obtained based on experimental work and verified with information obtained from supplier. The four different type of concrete mixes used in this study are 25% PPF, 75% SF, combination of 75% SF + 25% PPF and control mixes.

Table 1. Properties of SF

Parameter	Capability
Type:	HE 0.75/60
Shape	Hooked-End Deformed
Length (mm)	60
Diameter (mm)	0.75
Aspect Ratio, *l*/*d*	80
Tensile Strength (N/mm^2)	1100
Unit Weight (kg/m^3)	7850
Coating	None
Elastic Modulus, *E* (MPa)	205 000

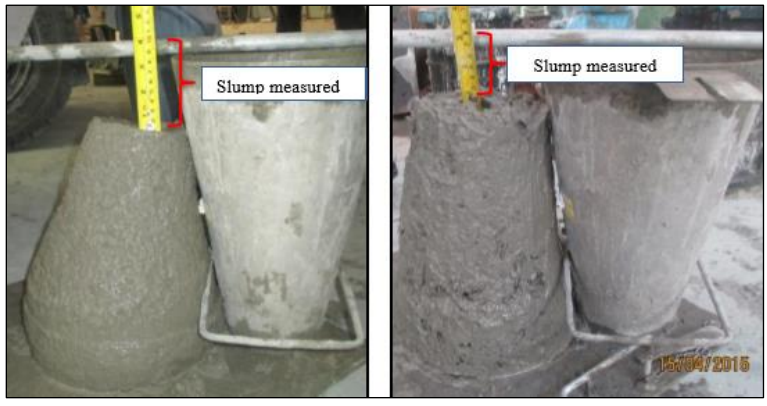
Table 2. Properties of PP

Parameter	Capability
Type	Virgin PPF
Length (mm)	19
Thickness (mm)	0.2
Unit Weight (kg/m^3)	446
Tensile Strength (N/mm^2)	400
Thermal Conductivity	Low
Elastic Modulus, *E* (MPa)	3500
Diameter (mm)	0.05

RESULTS

Workability

The evaluation of the fresh properties of FRC in terms of workability can be determined using the slump test. In assessing the workability of fresh concrete, slump test was carried out in accordance to a relevant standard (British Standards Institution, 2009). The slump measurement is done using a truncated cone of 300 mm height as shown in Figure 5(a) and 5(b). The design slump for all concrete batches is between 60 and 180 mm. After the concrete slump is recorded, the fresh concrete is transferred into the mould for sample casting in order to determine the FRC properties at the hardened state. Generally, there are three types of mode of failure for the slumps, namely true slump, shear slump and collapse as elaborated in a relevant standard (Ezeldin & Balaguru, 1993). However, only true slump is accepted based on the rheological properties of fresh concrete.



(a) (b)

Figure 5. Test Procedure for Workability Test: (a) Slump Fall Measured for Plain Concrete (b) Slump Fall Measured for Fibre Reinforced Concrete

A good workable concrete is one of the important criteria to ensure the fresh concrete can be placed and well compacted to achieve its maximum compressive strength. By adding superplasticizer at 2% from the weight of cement, the workability of concrete can be increased although lower water-to-cement ratio is used in the mixture. In general, the descending pattern of slump value is observed from the mix proportion of the control to 25% PPF, 75% SF and combined mixture 75% SF + 25% PPF. For the control, the concrete slump was 93 mm and decreasing for mixes with 25% PPF, 75% SF and combined mixture 75%SF + 25%PPF as shown in Figure 6. For the combined mixture 75%SF + 25%PPF, the lowest concrete slump was recorded at 62 mm. However, it is still in the range of the design slump. For this declining pattern, it was probably because the random orientation of the fibres preventing the flow of the fresh concrete during the test.

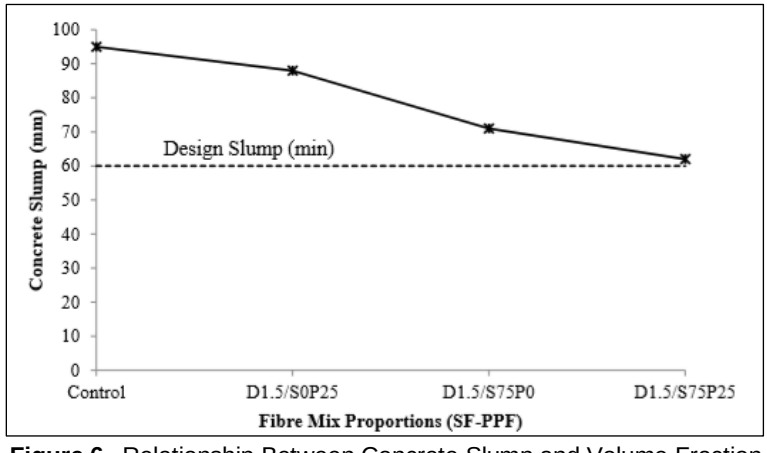


Figure 6. Relationship Between Concrete Slump and Volume Fraction

Compressive Strength Results

The compression test is performed at 7 and 28 days Normally for the 7-day test, the compressive strength recorded will be used to determine whether it had achieved or not the 60% to 75% of the design strength of 60 N/mm^2. Furthermore, the 7-day compression test was conducted to determine the early strength development due to the addition of fibres in

plain concrete. The relationship between the 7 and 28 days compressive strength for the control, 75% SF, combined 75% SF + 25% PPF and 25% PPF specimens are shown in Fig.7. At 7 days, the finding shows that the compressive strength increases for the combined 75% SF + 25% PPF and 75% SF alone compared with the control. There is less effect on the compressive strength when PPF is added to plain concrete. On the other hand, at 28 days the 75% SF recorded the highest compressive strength followed by the combined 75% SF + 25% PPF and the lowest was for 25% PPF mix.

The relationship shows an increasing pattern when SF was added to the concrete mixture. It was also reported that the compressive strength augment with the augmenting volume of SF (Ezeldin & Balaguru, 1993). These effects are most likely due to the transverse confinement effect of the SF, which restrained the lateral expansion of FRC specimens under load. On the other hand, other researchers reported that SF did not contribute to an increase in the compressive strength since more voids could be produced because of its low workability (Hsu & Hsu, 2006). These effects can also be minimized from the concrete slump value where superplasticizer was added in the mixture to improve the workability.

The relationship between compressive strength and fibre mix proportion shown in Figure 7 indicated that there was an increased in the compressive strength at 28 days with the addition of SF to the concrete mixture. This can be seen from the 10.6% increase for 75% SF followed by 10.0% and 0.98% marginal increased for the combined 75% SF + 25% PPF, and 25% PPF, respectively as compared with the control. The findings suggested that to some extent the SF fibre mixture has an effect on the improvement of the compressive strength. The increase resulted most likely from the improvement on the resistance to crack formation develop under axial load produced from SF and followed by PPF. Thus, it can be said that the addition of SF in plain concrete affected the compressive strength more than the PPF.

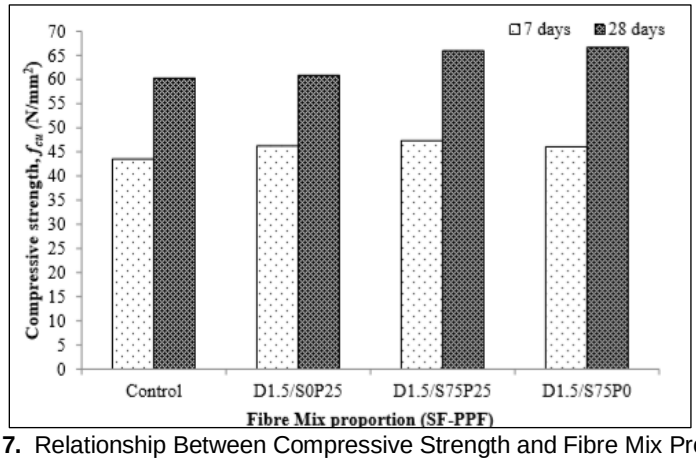


Figure 7. Relationship Between Compressive Strength and Fibre Mix Proportion

The most important contribution of SF and PPF in concrete can be seen from the failure mode. Distinctive failure patterns are observed between each batch. The comparison between 75% SF, combined 75% SF + 25% PPF, and 25% PPF show that the failure mode changed from brittle to ductile with lesser cracks develop at failure. Figure 8 shows the failure mode of the cube specimens at ultimate load for different mixes.

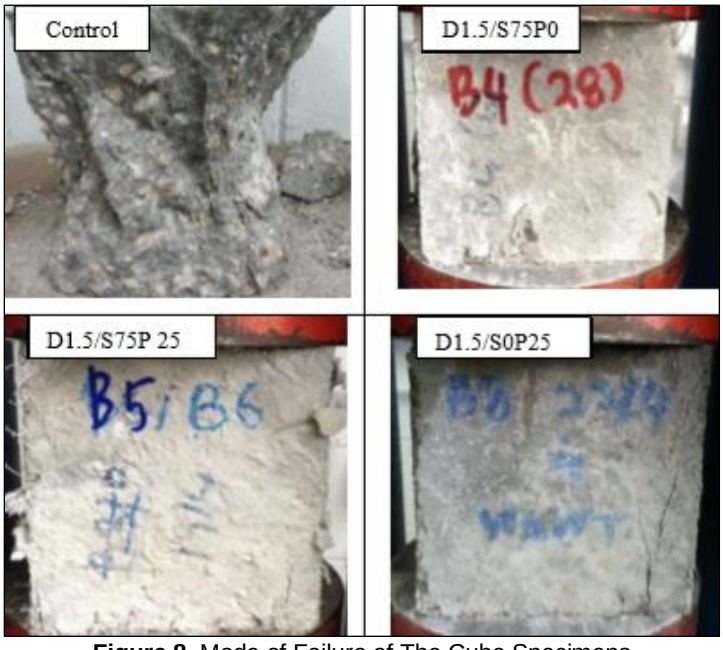


Figure 8. Mode of Failure of The Cube Specimens

Tensile Splitting Strength Results

Tensile splitting strength is performed using cylindrical specimen with the dimension of 150 mm in diameter and 300 mm height. In this section, the tensile splitting strength test is performed at the age of 28 days. The recorded tensile strength for concrete with 75% SF increased by 72.1% as compared with the control. Meanwhile, for the combined 75% SF + 25% PPF, the tensile strength was found to increase by 77.2% and the lowest was for the 25% PPF with only 3.41%. This shows that PPF had less effect on the tensile strength as compared with SF due to the difference in length, properties and stiffness. It is believed that the specimens for the combined 75% SF + 25% PPF and 75% SF yielded higher tensile splitting strength due to the inclusion of SF inside the specimen. This finding is as expected since previous studies showed that PPF only contributed in reducing micro-cracking at fresh state of concrete (Kashiyani et al., 2013; Sahoo, Maran & Kumar, 2015).

The relationship between the tensile strength and fibre mix proportions is shown in Figure 9. It was found that there is a substantial increase of about 77.2% on the tensile strength by the fibre hybridization process of both SF and PPF. The failure mode of the cylindrical specimens was also improved as shown in Figure 10. The cracking pattern by the fibre hybridization process was less than the one with the single fibre only (either SF or PPF), while the control (plain concrete) splits into two parts as shown in Figure 10. This shows that specimens with the fibre hybridization process have higher and better ductility than the other specimens.

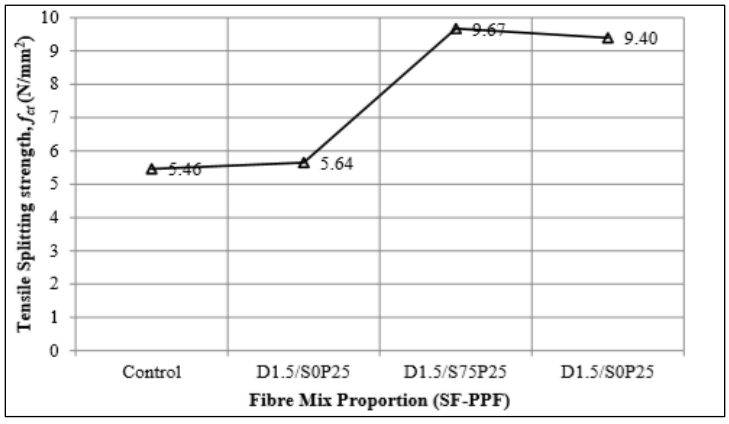


Figure 9. Relationship Between Tensile Splitting Strength and Fibre Mix Proportions

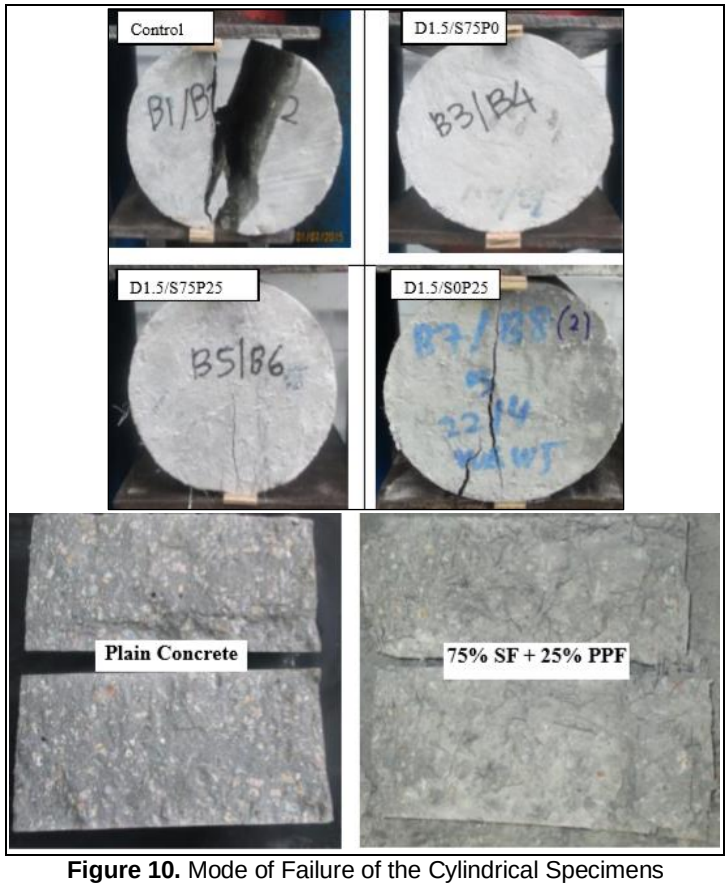


Figure 10. Mode of Failure of the Cylindrical Specimens

Flexural Strength Test Results

The flexural strength results represented by the ultimate load of the 100 × 100 × 500 mm prism specimens at twenty-eight days. The relationship between flexural strengths and fibre mix proportions for all batches are shown in Figure 11. Comparing the fibrous specimens and the control, it can be observed that the flexural strength is greatly affected by the inclusion of fibres in concrete mix.

Figure 11 shows that the 75% SF + 25% PPF specimen recorded an increase in flexural strength of 170% than the control, while for the 75% SF show an increased by 120%. The increase in flexural strength is likely attributed by the enhancement of concrete tensile properties resulted from confinement effects and good bonding between the fibres and the concrete matrix. However, for the mixture of 25% PPF, a slight decreased of about 3.1% in strength was recorded than the control. This is mainly due to the shorter size of PPF compared with SF which resulted in lower bonding mechanism between the PPF and the concrete matrix (Othman, 2014). The experimental results show that the combined 75% SF + 25% PPF mixture produced better flexural resistance compared with the other mixtures. This pattern is relatively similar to the finding from the tensile strength results and also from other previous studies (Hsie et. al, 2008).

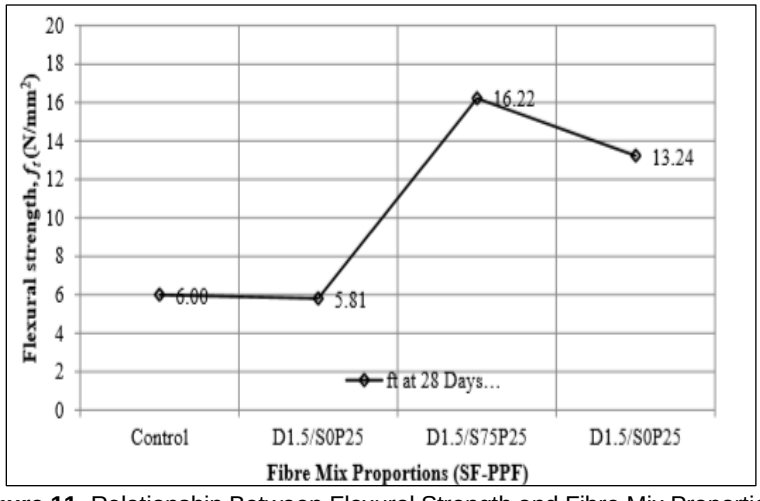


Figure 11. Relationship Between Flexural Strength and Fibre Mix Proportions

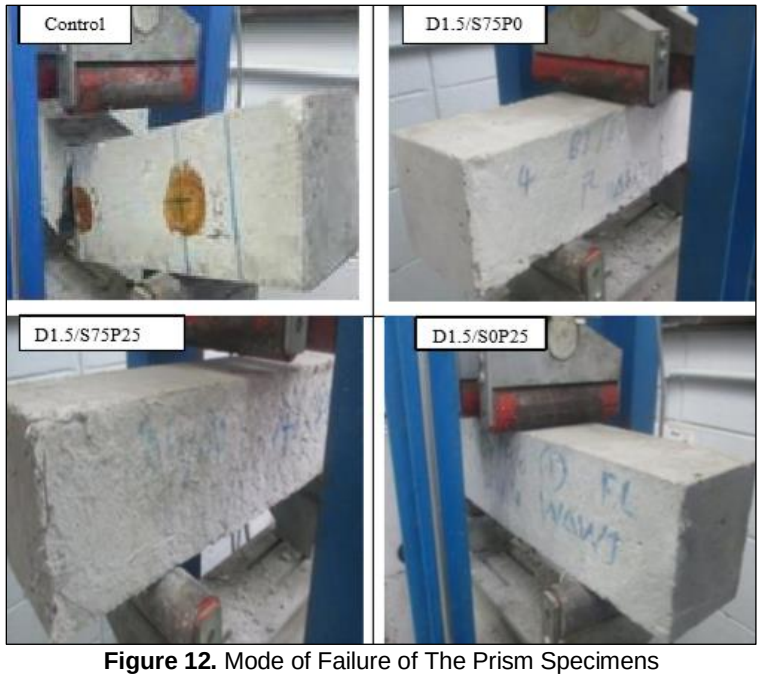


Figure 12. Mode of Failure of The Prism Specimens

The failure mode of the prism specimens is shown in Figure 12. All specimens with SF, PPF and combined SF-PPF did not break into two parts at failure as compared with the control. From visual inspection, the combined 75% SF + 25% PPF mixture had less cracking at failure than either the 25% PPF or the 75% SF alone. The test results also indicated that the combined 75% SF + 25% PPF enhanced the flexural strength in terms of its ultimate loading capacity and failure mode.

CONCLUSION

Through the experimental tests that have been carried out it was found that the best combination of fibres was for concrete containing SF 75% + PPF 25%. The combination of SF and PPF fibres in concrete was found to be able to enhance the engineering properties and controlling the growth of cracks in concrete. Engineering properties of the hybrid FRC show in an increasing pattern in the order starting from the control concrete, FRC mix with 25% PPF, 75% SF and 75% SF + 25% PPF. The inclusion of SF contributed positively effect on the tensile and flexural strengths of concrete. In addition, the failure mode for the FRC concrete also show more ductile behaviour for the 75% SF and the combined 75% SF + 25% PPF compared with control. Meanwhile for the 25% PPF it shows a bit brittle and almost similar failure mode with the control due to shorter fibre length. The findings also indicated the concept of hybrid fibre, by combining SF and PPF, the hardened properties of concrete can be enhanced not only on cracking behaviour but also the tensile properties.

ACKNOWLEDGMENTS

This research was supported and funded by the Contract Grant UTHM No. H525. Invaluable appreciation goes to all staffs, Assistant Engineers, and colleague in the Structures and Materials Laboratory, Faculty of Civil Engineering, Faculty of Chemical, Universiti Teknologi Malaysia (UTM), Faculty of Civil Engineering Technology and Universiti Tun Hussein Onn Malaysia. Special acknowledgment also goes to the Ministry of Higher Education Malaysia (MoHE) and Universiti Tun Hussein Onn Malaysia (UTHM) for the scholarship sponsored throughout the study.

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PERFORMANCE OF CONCRETE CONTAINING COARSE RECYCLED CONCRETE AGGREGATE AND SYNTHETIC WIRE WASTE AS FIBER MATERIALS

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Abstract
Concrete demand in Malaysia construction field is growing directly proportional with the local economic growth and due to the improvement of technology in the particular field, this demand is expected to increase continuously. Due to that reason, this study was held to determine the performance of partial coarse recycle concrete aggregate containing insulator synthetic wire waste as fiber materials can reduce the amount of waste dumped into landfills. The uncontrollable amount of wire waste being dumped in the landfilling area may lead to the environmental related issue. There is still no researcher conduct a research regarding the performance of concrete incorporating combination of coarse recycle concrete aggregate and insulator synthetic wire waste in the concrete mixing design. Therefore, to curb the environmental related problem, a study was made by using coarse recycle concrete aggregate and wire waste as materials in the concrete mixture. The amount of insulation synthetic wire waste as fiber materials 0.5%, 1.0% and 1.5%. Meanwhile, coarse recycle concrete aggregate was incorporating with concrete as partial coarse aggregate replacement at 10%, 15% and 20% with water-cement ratio was 0.55. The workability, density, compressive strength, and tensile strength tests have been conducted for 7 and 28 days. The strength increasing at 15% CRCA and 1.5% wire waste for 9.8% compared to normal specimen. However, the concrete strength starts to decrease when 20% CRCA and 1.5% wire waste were added compared to normal specimen. Therefore, it can be concluded that optimum percentage for CRCA is 15% replacement materials and addition of 1.5% wire waste as fiber. At the end of study was produced relationship between compressive and tensile strength which is fct, wire waste = $0.002(fc)^{2.098}$ with R^2 was 0.786.

Keywords: *Synthetic wire waste, fiber materials, strength, durability.*

INTRODUCTION

Fibers is a suitable material to be used in the structural and non-structural element in construction field either as partially of totally replacing materials. Several countries had acknowledged the use of fibers in these elements as the quality of it have been significantly improved over the past decade and the design guidelines for using it had been prepared due to its ability to increase toughness of concrete (Azmi et al., 2018), (Irwan et al., 2014) and (Shahidan et al., 2018). Despite of its benefit to the concrete performance, the disadvantages of using fiber is it can reduce the workability, stability and mobility of a concrete (Irwan, Annas, Aeslina, Asyraf & Faisal, 2013) and (Ali, Shekarachi, Mahoutian & Soroushian, 2011) especially when they are mixed together with the recycled concrete aggregate (RCA). The reason for this is both fiber and RCA have a relatively high specific surface area which result in increasing water demand during concrete mixing, placing and compacting. Other than that, the difficulty that faced during the usage of fiber in concrete mix is where fiber could be entangle and its distribution or dispersion in concrete matrix may become ununiform.

The incorporation of RCA in concrete mix to be applied in the construction is relatively a new technique. The history of considering the use of RCA was started at the end of world

war II. There was a lot of building debris from the demolition of buildings and roads result from the war. Due to tons of waste, Europe had an idea to recycle it to rebuild the country instead if get rid of it (Buck, 1977). In the 1970s, the United States began to reintroduce the used of RCA in non-structural uses, such as filled material, foundations, and base course material. Since this time, some researched had been conducted regarding viable RCA was as an option to replace unused natural aggregate in structural concrete. Physical properties of differentiate NA and RCA have been conducted as shown in Table 1.

Table 1. Physical Properties of NA and RCA (Ali et al., 2011)

Physical Properties	Natural Aggregate	RCA
Texture & Shape	Smooth to angular rough surface	Angular rough surface
Specific gravity	2.4 – 2.9	2.1 – 2.5
Bulk Density (kg/m^3)	1450 – 1750	1200 – 1425
Pore Vol. (%)	0.5 – 2	5.0 – 16.5
Water Absorption (%)	0.5 – 4	3 – 12

Consequently, RCA was less homogeneous, less dense and more porous than the natural aggregate (Azmi, Khalid, Irwan, Anting & Mazenan, 2017). In general, a specific gravity of RCA was smaller than natural aggregate (NA) because of the cement paste in RCA particles. As stated by (Ali, Yusup, Khalid, Shahidan & Abdullah, 2018) concluded that 16-32 mm aggregate particles of RCA had about 30% of old cement mortar. Fine recycles particle below 4mm contain approximately 20% of old cement paste, while the filler fraction 0-0. 3mm may contain as much as 65% of old cement paste. However, the mix design procedure for RCA has not had a different much from that of normal concrete. Meanwhile, synthetic fiber was made from chemicals and could have superior properties such as flexible and microscopically homogenous body with high ratio of length over diameter. This properties would lead to provide a small cross section fiber. Synthetic fiber is manufactured from naturally occurring macromolecules and synthetic polymers. According to (Cuenca, Echegaray Oviedo & Serna, 2015), fiber is produced from different material in various sizes and shapes. This can be supported by (Amezugbe, 2013) that stated the section of fiber may be triangular, crescent-shaped, polygonal, round or even square depending on the manufacturing process and purpose of the fiber will be used. There are two major categories of fibers that are micro and macro as shown in Table 2 (Irwan et al., 2013).

Table 2. The Types of Fiber Used in FRC (Cuenca et al., 2015)

Fiber Types	Materials Poperties				
	Diameter (mm)	Specific Gravity	Modulus of Elasticity (GPa)	Tensile Strength (MPa)	Strain at Failure (%)
High Tensile steel	0.2	7.8	200	5 – 25	3.5
Polypropylene	0.50 – 1.4	0.9	3.4	15 – 20	8
Polyethylene	0.25 – 0.41	0.96	13 – 25	6 – 60	3 – 80

Other than that, the usage of fiber in the concrete mix also may influence its workability, strength and durability due to the presence of air content by the fiber. However, this hypothesis is not strong enough and still be a topic of discussion among the research community (Ali et al., 2011). For example, (Bayasi & Soroushian, 1992) and (Ortiz-Naval, Navarro, Leiva, Serna & Cuenca, 2018) in their study found that the concrete air content increases as the percentage of fiber increases and also depend on the type and shape of the fiber. But on the other hand, (Balaguru & Ramakrishnan, 1988) stated oppositely and

strengthen the finding by saying that the rate of air in the concrete loss is higher when incorporating with FRC.

An effective way to improve concrete performance is by adding synthetic fiber to improve the fracture characteristics and behavior of structures through the capability of the fibers to bridge cracks. Another application of this fiber material is to reduce the shrinkage and shrinkage cracking of concrete associated with hardening and curing. If the rate of evaporation at the concrete surface is greater than the rate of water migration to the sur-face, plastic shrinkage will occur. This gradient in shrinkage forces cracks to appear on the surface. Plastic shrinkage occurs during hardening because the strength and stiffness of wet concrete are very low, only a small amount of fibers is needed to effectively reduce the plastic shrinkage. Another benefits of the fiber materials are improved fatigue strength, wear resistance and durability of the concrete. Section thickness can be reduced and cracking can be effectively controlled by using fiber materials instead of conventional concrete, resulting in lighter structures with a longer life expectancy.

The aim of this manuscript is to determine and evaluate performance of concrete containing partial coarse recycled concrete aggregate and insulator synthetic wire waste as fiber materials in terms of fresh and mechanical properties of fiber reinforced concrete.

MATERIALS

In this study, the CRCA was used in concrete production as partial replacement of natural coarse aggregate. The CRCA can be obtained by crushing the original concrete cube waste get from Structural and Materials Laboratory in University Tun Hussein Onn Malaysia and the original concrete cube waste was crushed by using crusher machine at the same place. After that, the material has passed to the sieve analysis test to find the proportion size of CRCA. Next, the coarse recycled concrete aggregate had to be grind until the size of 14 – 5 mm sieve and used 15%, 30% and 45% of CRCA for this study. Ordinary Portland cement was used in the mixtures.

Insulator synthetic wire waste was added material in the mixture. This insulator synthetic wire waste was used as a fiber material. The insulator synthetic wire waste of diameter 1mm – 2.1mm without copper inside was tested as fiber material in concrete. The content of insulator synthetic wire waste fibers is 0%, 0.5%, 1.0%, and 1.5%. The ratio over length of wire waste was fixed at 34.25 (L/df). The water-cement ratio of mix design concrete is 0.55. This study was used 10%, 15%, and 20%of CRCA.

RESULTS AND DISCUSSION

The tests that involve in this study are slump, density, compressive strength and tensile which been tested at UTHM Structure and Material laboratory. These tests were conducted to determine the workability and performance of CRCA.

Slump and Density Tests

Figure 1 shows the comparison slump value between normal concrete and CRCA. It shows that increasing in fiber materials can decrease the value of slump.

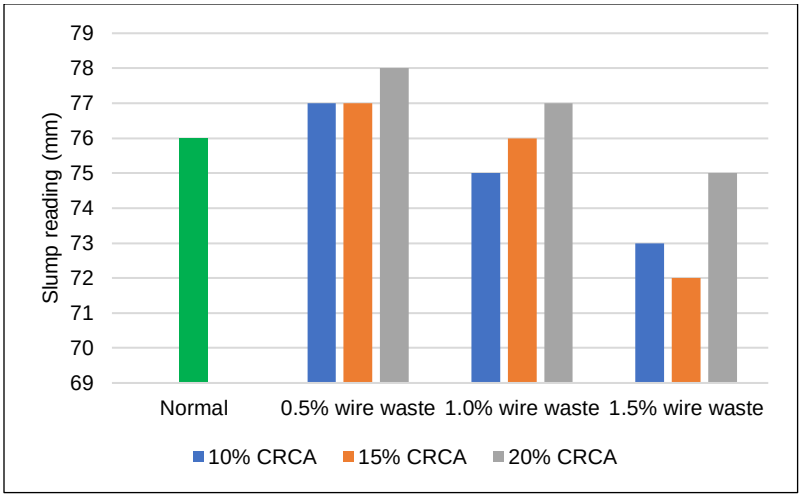


Figure 1. Slump Test Value

There is decreases in slump value on concrete incorporating with wire waste when compared to the control slump. Slump value recorded for the control concrete was 76mm while the slump value decreases on 0.5%, 1.0% and 1.5% wire waste which is 72mm, 69mm and 68mm respectively from the value of control slump. From the Fig. 1, it shows that increasing in fiber materials can decrease the value of slump. This trend has been observed by Yin et al. (2015) study that found fibers in concrete can form a network structure in the concrete matrix. This would lead to uniform spread fiber mixture concrete and cause segregation and insiginificant flow of fresh concrete.

Figure 2 shows that the result of average density for all specimen for 7 and 28 days of curing. The result concrete containing 10% of CRCA and 0.5% of wire waste at ages 7 days have the highest density value -1.36%. Cube concrete that contain 10% of CRCA and 1.0% of wire waste have density value -0.32% were decrease compared to normal specimen at age 7 and 28 days respectively. It was slightly decrease for the density when 0.5% of wire waste was used.

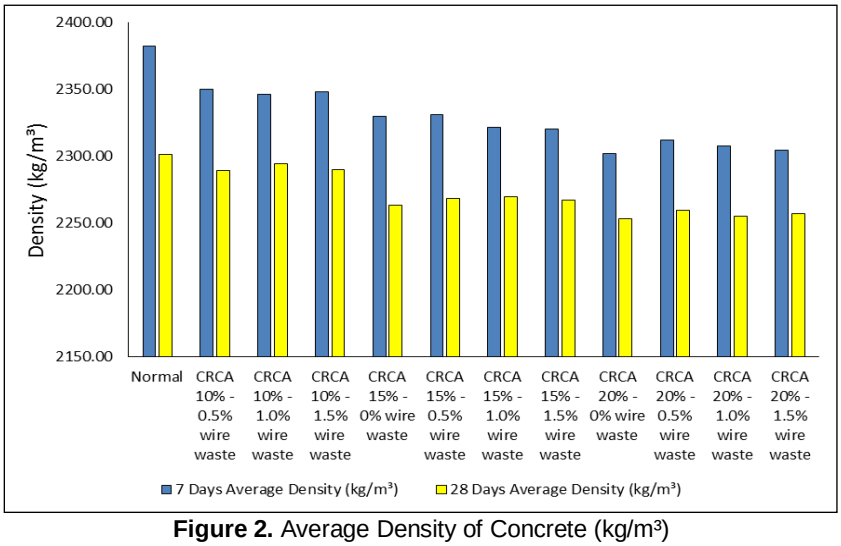


Figure 2. Average Density of Concrete (kg/m³)

It shows that the average density of the normal concrete is highest value compared to concrete containing CRCA and insulator synthetic wire waste. The comparisons between average density of concrete containing 20% CRCA and 1.5% wire waste at age 7 days is 2304.4 kg/m³ is lowest than to the control concrete, 2382.2 kg/m³, with -3.27% compared to the normal sample. Besides, the average density of concrete at age 28 days for the concrete 20% CRCA and 0% insulator synthetic wire waste, 2253.2 kg/m³ compared to normal sample 2301.5 kg/m³, while the percentage is -2.10%. This is because of the content cement mortar attached to RCA and the density of reclaimed mortar is lower than that of NCA (Zainora et al., 2018) .

Compressive Strength Test

Concrete incorporating with CRCA and wire waste was found a significant effect to the concrete compressive strength at ages of 7 and 28 days. Concrete containing 15% of CRCA and 1.5% insulator wire shows highest strength compared to others sample. It can clearly seen that sample at age of 7 days showed 24.30 MPa compared to normal specimen which is, 22.10 Mpa as shown in Fig. 3. Strength at age of 28 days obtained 35.80 MPa compared to normal specimen which is 32.60 MPa was also shown in the same Figure 3.

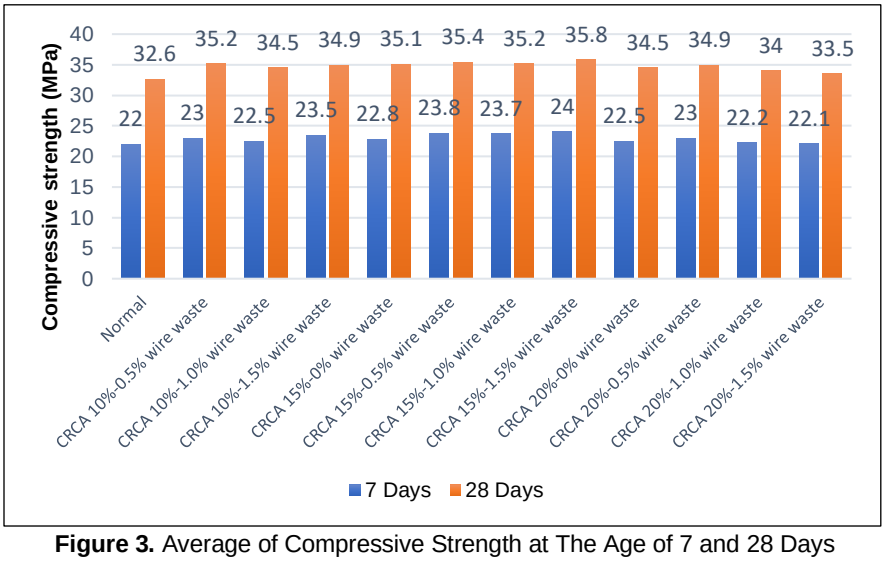


Figure 3. Average of Compressive Strength at The Age of 7 and 28 Days

Figure 3 shows the the compressive strength results at age of 28 days are similar pattern with compressive strength at age of 7 days. Concrete cubes containing 15% of CRCA and 1.5% was increased for 9.82% compared to normal specimen. Concrete cube containing 20% of CRCA and 1.5% wire waste decreased 2.76% respectively.

Thus, compressive strength of concrete conatining CRCA showed a significant increase in strength when 1.5% of insulator wire has been added in the mixture. By this result, it can conclude that the CRCA concrete with 1.5% of wire waste provide a better performance in compressive strength compared to CRCA concrete with 1.0% wire waste and also the control concrete. the wire waste act to filled up the void in concrete and with the optimum amount of wire waste, it can give advantage in term of concrete strength (Mazenan et al., 2018) and (Raphael & Jerome, 1984). By this finding, the optimum usage of wire waste is 1.5%.

Tensile Strength Test

Splitting tensile strength test was conducted to determine the tensile strength of concrete that containing CRCA and insulator wire waste as fiber materials. Figure 4 shows slightly increment of tensile strength of concrete with incrasing of CRCA and insulator wire. The result indicates that concrete contains 15% of CRCA and 1.5% of insulator wire has highest tensile strength compared to others sample. Sample containing 15% of CRCA and 1.5% of insulator wire increase 26.26% compared to normal sample. This result indicates that insulator wire in concrete forming a bridge locking tensile between matrix concrete. This locking tensile has increase ability of concrete to sustain the tensile loads compared to normal concrete without bridge locking tensile (Mazenan et al., 2018).

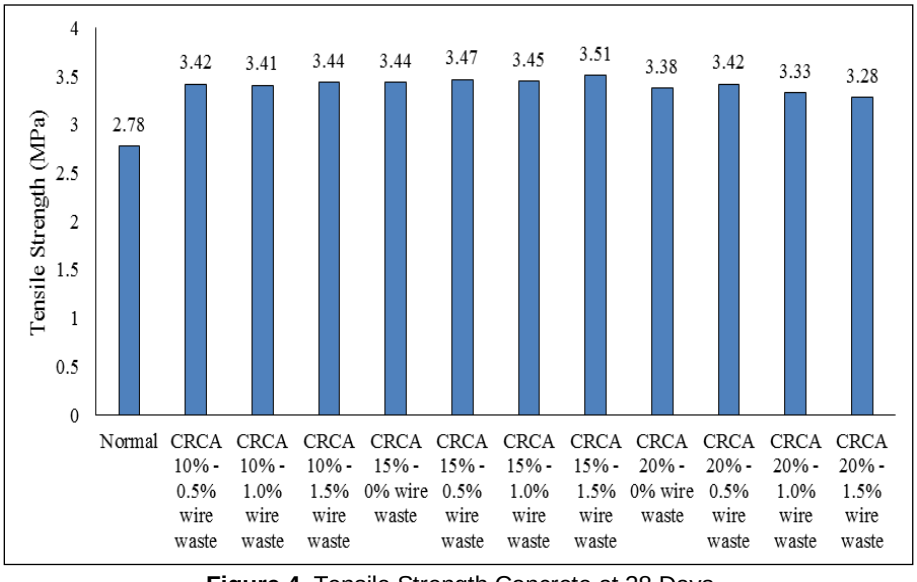


Figure 4. Tensile Strength Concrete at 28 Days

Correlation Strength between Compressive and Tensile Strength

Previous studies indicate that the compressive-tensile strength relationship as in Equation 1 (Raphael & Jerome, 1984) and (Arioglu, Girgin & Arioglu, 2006).

$$f_{ct}=A(f_c)^B \quad (1)$$

where;

A & B is Coefficients Regression

Based on Equation 1, the relationship for compressive-tensile strength for wire waste as fibre concrete derives as following Equation 2. The equation is obtained from statistical analysis. Figure 5 shows the comparison of compressive-tensile relationship between experiments and using predicted Equation 2.

$$f_{ct}, \text{ wire waste} = 0.002(f_c)^{2.098} \quad (2)$$

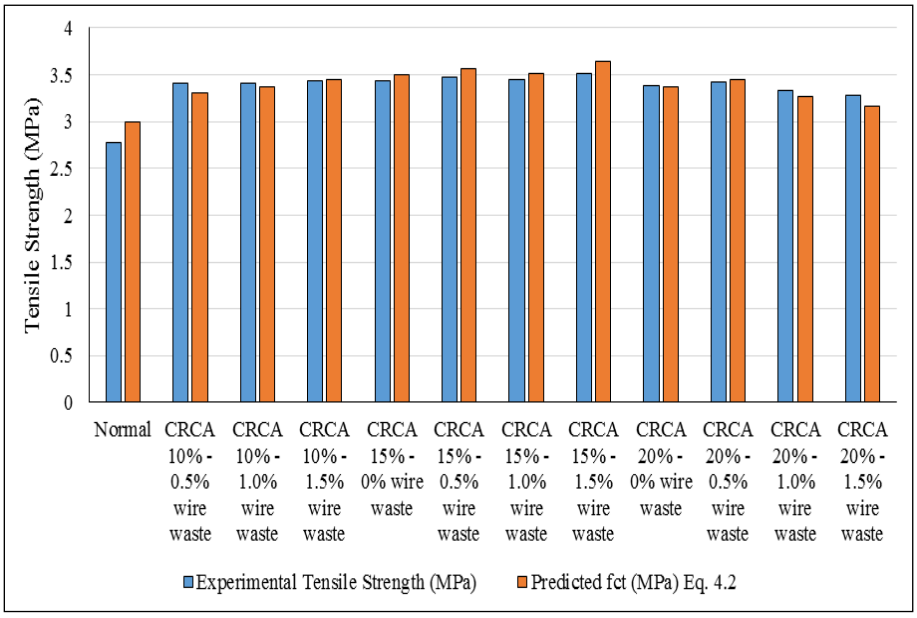


Figure 5. Experimental and Predicted Tensile Strength (Equation 2)

Analysis of relationship between compressive and tensile strength of concrete shows that the optimum percentage of a CRCA and wire waste set to be 15% CRCA and 1.5% wire waste for both of the test. The analysis also shows that when this percentage of CRCA and wire waste set up to 20% CRCA and 1.5% wire waste the strength for compression and tensile were decreased. From that analysis it shows that as a strength for compression increased the strength for tensile also were increased and vice versa. From the result, the relation equation is fct, wire waste = 0.002(fc)2.098 with R2 was 0.786.

CONCLUSION

The significant difference of workability can be seen between normal specimen and specimen that contain CRCA and wire waste. CRCA and wire waste added shows the poor workability as concrete was difficult to mix compared to normal specimen that easily spread. The strength increasing at 15% CRCA and 1.5% wire waste for 9.8% compared to normal specimen. This combination of material in concrete also observed increment of 26.3% tensile strength compared to normal specimen. Therefore, it can be concluded that optimum percentage for CRCA is 15% replacement materials and addition of 1.5% wire waste as fiber. At the end of study was produced relationship between compressive and tensile strength which is fct, wire waste = 0.002(fc)2.098 with R2 was 0.786.

ACKNOWLEDGEMENT

This work was financially supported by University Tun Hussein Onn Malaysia (UTHM) and Minister of Education Malaysia (KPM) through Dana Penyeldidikan Malaysian Technical University Network (MTUN) K 122.

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INTEGRATION OF GEOPHYSICAL INTERROGATION AND GEOMECHANICAL ASSESSMENT FOR SEDIMENTARY ROCK MASS CLASSIFICATION, ISKANDAR PUTERI, JOHOR, MALAYSIA

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Abstract

This paper focuses on the characterization and classification of sedimentary rock mass, located in Iskandar Puteri, Johor, Malaysia by utilizing the combination of geophysical investigation and geomechanical assessment. Resistivity and seismic refraction method were employed for geophysical investigation while the geomechanical classification involves Q system and Rock Mass Rating (RMR). The geophysical methods were carried out on the rock outcrop to estimate the rock mass quality index. For Q System and RMR, parameters that were considered are the point load strength index (PLI), rock quality designation (RQD), degree of weathering, joints characteristics, rock types, lithology, structures, and texture. Rock mass characterization and classification were done based on the correlation between geophysical and geomechanical results. Both Q system and RMR show that higher value of rock mass rating has higher range values of seismic and resistivity. Similarly, result indicates that the increases of weathering grades form Grade VI to II (residual soil to slightly weathered) reducing the resistivity and seismic values from 12,000 Ωm to 0 Ωm and 3,500 m/s to 300 m/s, respectively. As a conclusion, the classification of sedimentary rock mass based on the correlations between geophysical result from seismic velocity and resistivity and geomechanical classification using Q-system and RMR can be used as one of the useful tools in site investigation in order to enhance the subsurface information during site investigation.

Keywords: *Sedimentary rock, rock mass classification, Q-system, rock mass rating, resistivity, seismic refraction.*

INTRODUCTION

The complexity in characterizing weathered rock for excavation purposes has become one of the major issues in tropical countries. An initial effort of Bieniawski introduced the stability assessment of a rock mass classification systems in 1979, a number of researchers have developed rock mass classification systems that are based upon circumstances for easy interpretation by grading, graphical and geophysical method (Bieniawski, 1989). However, little number of studies have been carried out as well as no exact guideline to classify the problematic weathering zone that exist abundantly in the wet tropical region. For example, a guideline to address drastic changes of behaviour of materials in highly and completely weathering zones due to moisture increase that influence the workability of the material (Edy Tonnizam et al., 2010).

Rock quality designation (RQD) is one of the first systems developed consider only for the joint frequency within a rock mass (Deere, 1963). RQD was used in Rock Mass Rating (RMR) (Bieniawski, 1973) and Q systems (Barton et al., 1974) as one of the measurements,

including others parameter such as intact rock strength, joint condition, joint spacing, groundwater and number of joint sets. The onsite observation data were then analysed in terms of RMR and Q system value. Furthermore, numbers of studies described the used of geophysical methods in rock mass characterization. Boadu (2000) studied relationships between seismic wave velocity with permeability and porosity of fractured media. Another study by Kahraman (2002) shows the relationship between P wave velocity with Fracture Rougness Coefficient. Barton (2002) correlated Q values and P wave velocities for hard rock tunnelling project.

Besides wave velocity, rock mass classification has been correlated to electrical resistivity Matsui & Park (1996). Choi et al. (2006) correlated electrical resistivity to the Q system through theoretical analyses. Each technique is individually considered; hence it is difficult to compare the results based on different circumstances. Since the wave propagation and electrical resistivity of rock mass can capture the characteristics of the rock mass that depend on the properties of joint surface and intact rock, this geophysical tool is a key role for evaluating the state of rock mass besides the existing rock mass classification systems.

Geophysical approaches play a significant role in delivering helpful and cost effectives information about subsurface features at the required level of spatial resolution and target definition. Geophysical methods can produce continuous image of subsurface profile with a measurement that provides an improvement in accurate and sufficient information for heterogeneous ground. The combinations of geotechnical and geophysical discipline can be advantageous in classifying the depth of rock layer, detecting voids, cavities and boulders in subsurface works.

Since the early application of seismic refraction method to determine depth to bedrock at 1930s, geophysical methods have been discovered as one of the dependable practices for geotechnical evaluation (McDowell et al., 2002; Anderson et al., 2008; Mahvelati et al., 2018). The greatest verdict when applying the method in early stage of site investigation work is that able to discard other site investigation technique and shortlist those with potential values for improving the overall effectiveness of site investigation in term of cost and time consuming. The purposes of this study focuses on the use of resistivity survey and seismic refraction methods in the selected outcrop and the result are then correlate with the geotechnical result, while potential relation between the parameters are studies to get better interpretation in term of geophysics and engineering. There is a need for better documentation and correlation of geological and geophysical for each grade of weathering profile for surface excavation purposes to implement precise and economical method in the construction.

GEOLOGICAL SETTING

Legoland (LEGO) is located at latitude 103° 37´ 47.4" E and longitude 01° 26´ 04.4"N. The main access to the site is by Second Link Plus Highway. The geology of the study area in Legoland, Nusajaya covers the variable lithology of an older Alluvium as consistent with Burton (1972). The most common variety is coarse feldspatic sand with irregular rounded phenoclasts, but gravely clay, sandy gravel, sandy clay, silty clay, clayey sand and clay are also well presented. The Older alluvium contains phenoclasts of vein quarts, quartzite, sandstone, siltstone, shale, hornfels, granite, granite porphyry, alaskite, aplite, rhyodacite, andesite and tuff. Generally, bedding is moderately well-developed in it, but close

interbedding of various lithological types can be seen in few places. Therefore, the formation is exposed in outcrops surfaces, as weathered sedimentary rock of sandy clay or clayey sand as shown in Figure 1 (red dot) where the Triassic-Jurassic formation are located with sedimentary rock associated volcanism.

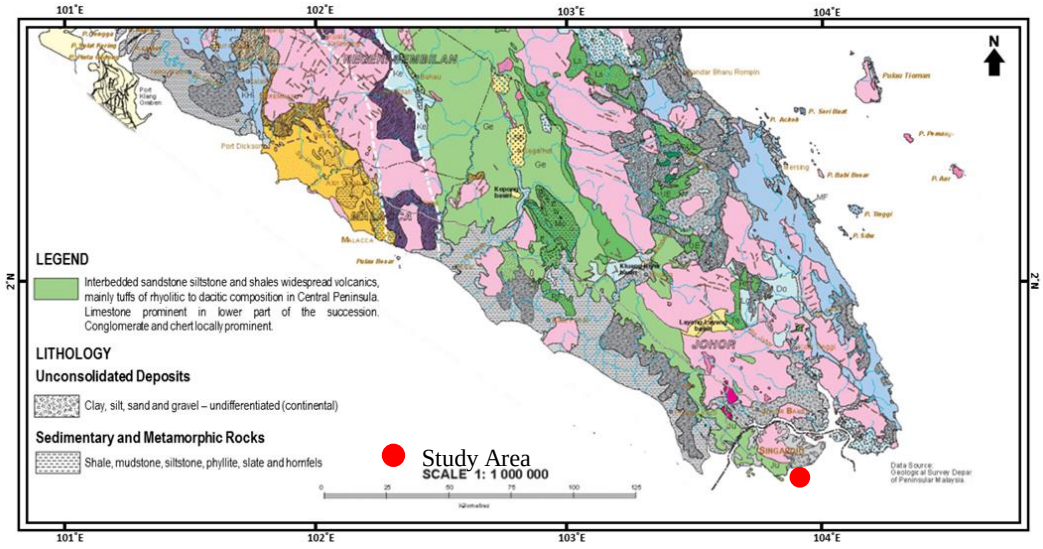


Figure 1. Geology of Study Area Located at Iskandar Puteri (Ng et al., 2008)

The unique weathering profiles and heterogeneous physical deterioration of rock mass in tropical climate country such as Malaysia is widely in studies by Komoo (1995), Zainab (2004), Winn Kar et al. (2018). The weathering profiles differ with rock type. The Jurong formation covers the site with a variety of interbedded sedimentary rocks, including sandstone, shale and siltstone. It was deposited during late Triassic to early or mid-Jurassic (Burton, 1973). This outcrop has a slightly to highly weathered sandstone and shale dominating the rock mass. This study applies the scan line technique as proposed by Priest and Hudson (1976) in the range of 20 m. Figure 2 shows the position map of both resistivity and seismic survey mark in red line on the study area.

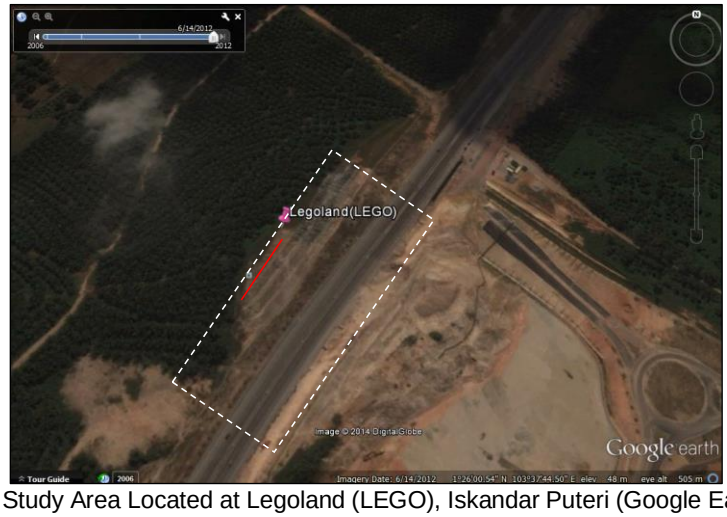


Figure 2. Study Area Located at Legoland (LEGO), Iskandar Puteri (Google Earth, 2014)

METHODOLOGY

This study begin with the distribution of the rock mass outcrop in Figure 3 into panels in which certain features are uniformed and characteristics are similar. Major geological features such as joints were measured in each panel. The field mapping techniques were adopted from a proposal by the Geological Society of London Engineering Group Party (1977). The weathering grade classification has been carried out in order to classify the quantitative description of discontinuities of weathering profile. Determination of weathering grade classification was carried out in accordance to ISRM (2007) with additional information from published journals by previous researchers (Geological Society, 1990; Ibrahim Komoo, 1985). According to ISRM (2007), this classification is divided into six grades namely Grade I for sound/fresh rock, Grade II for slightly weathered, Grade III for moderately weathered, Grade IV for highly weathered, Grade V for completely weathered and Grade VI for residual soil .This study includes weathering zone classification qualitatively based on physical characteristic of the rock mass and material distribution in the weathering profile. Discontinuity evaluation is frequently done by two selection arrangements, whereas scanline and window survey are carried out as in Song (2006), scanline method represents more comprehensive data in measuring and evaluating joint characteristic (Gumede, 2007; Stacey, 2007). The physical characteristics of each divided zone consist of the degree of discoloration, friability of rock material, rock texture and retention of original rock fabric as in ISRM (2007). Indirect method suggested by Priest and Hudson (1976), then reviewed by Zhang (2016) was used to assume RQD in exponential with joint spacing as in RQD = 100 e -0.1λ (0.1λ. + 1) due to the limitation in obtaining RQD value on site. The data of field mapping were then calculated in term of Q-system and RMR to correlate its relationship with the geophysical results. Data include strength, geometry of the exposed faces, lithology, structures, and texture, were obtained based on the selected panel which are divided into different weathering grade.



Figure 3. Selected Outcrop of The Studied Area at Legoland (LEGO), Iskandar Puteri

Geophysical methods in subsurface applications are currently well known because of its effectiveness and cost. The results from two geophysical field measurements were presented in a form of inverted 2D image. The inverse model of resistivity section and seismic refraction section showed the profile of subsurface and display resistivity index and seismic refraction index of ground mass signify by colour scheme. Physical parameters that related to engineering are discussed in detail by considering the geological setting and geomorphology of the study areas.

Two resistivity cables, 41 stainless steel electrodes and 42 jumpers are used with Terrameter SAS4000. A single channel relay matrix switch of Electrode Selector 464 is connected to ABEM Terrameter SAS4000 in the resistivity survey. The electrode spacing

used in this study is 2m and total covered length is 80m in order to obtains the resistivity value based on the exposed outcrop length. Pole-Dipole array was employed and RES2DINV software was used for the resistivity imaging. Pole – dipole array provides deeper penetration and better underground resolution even though survey line is shorter than other arrays (Nordiana et al., 2018).

Seismic survey uses a 12V battery pack, a trigger cable, two seismic cables, ABEM Terraloc Mark 8 (recording equipment), 24 geophones of 28Hertz, 12-pound hammer and a striker metal. The total length of the survey is 115m with geophone spacing of 5m in purpose to obtain detailed and reliable seismic data for the survey area. Optim software was used for seismic imaging. Field survey were conducted simultaneously on the outcrop. On site observation for geotechnical work were carried out and sample were brought back for laboratory test to determine engineering parameters.

RESULTS AND DISCUSSIONS

This section discusses study findings from rock mass classification and geophysical interpretation.

Rock Mass Classification

The site consists of massive sandstone and shale. The total length of studied rock outcrop is 80m with the height of the panels approximately 18m. The massive sandstones layers with average thickness of 5m dominated this area which interbedded with shale in average thickness of 3m. Generally, sandstone and shale with very fine to medium sized in light grey to yellowish reddish-brown colour are identified. The weathering of each layers varies relatively, the upper 5m of the outcrop are completely weathered while the bottom part is interbedded with different weathering grade of sandstone and shale. The bedding dipped toward N 30 ° E with an average angle of 35°. The overview of LEGO site in Figure 4 presents the lithology, bedding orientation and joint for the outcrop. The complexity of sedimentary rock mass having interbeds of several rock types is significant to be addressed. The ground mass was classified from slightly weathered to highly weathered sandstone and moderately weathered to highly weathered shale.

Legoland comprises of interbedded slightly to highly weathered sandstone and moderately to highly weathered shale. Slightly weathered sandstone was identified to be very firm, with intact surface texture and only breakable by several blow of geological hammer along discontinuities. In the meantime, moderately weathered sandstone can be visibly notable as it seemed to be blocky and easily break by hammer. On the other hand, highly weathered sandstone is easily breakable by hand and very friable. It can be seen that the exposed sandstone rock mass has experienced aggressive tropical weathering overtime. weathering process deteriorates the rock strength, resulted in low value of Schmidt hammer readings. It is due to the loss of cohesion between surface grains, which extricate the rock surface as reported by Arikan et al., (2007). Besides that, Schmidt hammer value could not be obtained on completely weathered material, discloses the restriction of using rebound hammer in testing the surface hardness predominantly in weak rocks. This is in with earlier study by Aoki and Matsukura (2007) and Gokceoglu et al. (2009).

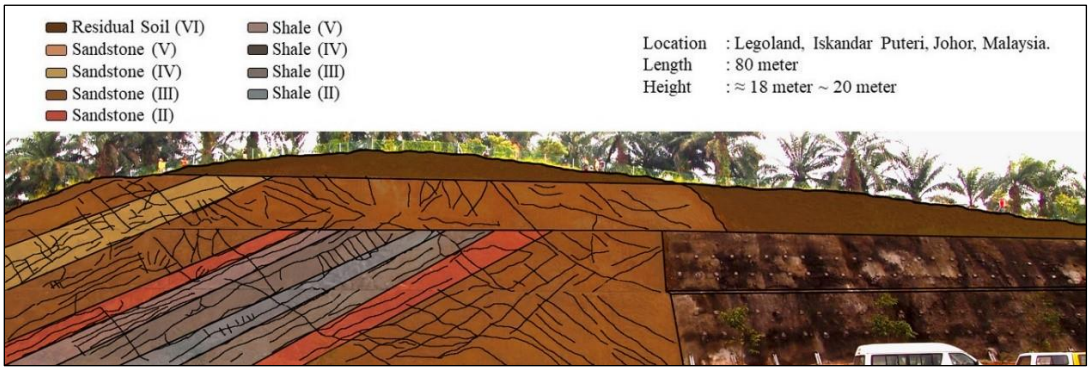


Figure 4. The Lithology and Weathering State of Studied Outcrop at Legoland (LEGO)

The outcrops were classified using Rock Mass Rating (RMR) and Q-system as these systems are widely used for classifying the rock mass. RMR (see Table 1) and Q-system (see Table 2) rating for respective panels are valued based on the parameters measured. Due to the restrains in obtaining the core sample for weathered material, indirect value using point load strength is employed for the estimation of RMR value in the strength parameter (Karaman and Kesimal, 2014; Karaman et al., 2015). Maximum RMR value of 69 is recorded with 27 as the minimum value. Slightly weathered sandstone and boulder logged with value of 69, followed by 50 to 55 for moderately weathered sandstone and 38 to 45 for highly weathered sandstone. Meanwhile for shale, 53 to 55 is calculated for moderately weathered material, 45 to 48 for highly weathered shale and 27 for completely weathered shale.

Table 1. RMR for Studied Area

Panel	PLT $I_{S\ 50}$ (MPa)	Joint Spacing (m)	RQD (%)	R_s	R_{RQD}	R_{dj}	R_{cj}	R_w	R_{oj}	RMR
BLR 1	4.684	1.300	90.980	12	17	5	20	15	0	69
SAIV 1	0.772	0.600	50.367	0	8	5	10	15	0	38
BLR 2	4.286	1.200	89.251	12	17	5	20	15	0	69
SAIV 2	1.003	0.550	45.745	4	8	5	10	15	0	42
SAIII 2	2.269	0.750	75.327	7	13	5	15	15	0	55
SHV 3	1.045	0.350	22.153	4	3	5	0	15	0	27
SHIV 3	1.198	0.550	45.745	7	8	5	10	15	0	45
SHIII 3	2.824	0.100	73.576	7	13	5	15	15	0	55
SHIV 4	2.001	0.580	48.579	7	8	8	10	15	0	48
SHIII 4	2.578	0.100	73.576	7	13	8	10	15	0	53
SAIV 5	1.009	0.550	45.745	4	8	5	10	15	0	42
SAIII 5	2.145	0.100	73.576	7	13	5	10	15	0	50
SAIV 6	1.113	0.550	45.745	4	8	5	10	15	0	42
SAIII 6	2.567	0.750	75.327	7	13	5	10	15	0	50
SAII 7	4.108	1.200	89.251	12	17	5	20	15	0	69

The value of Q was evaluated by panel and calculated based on Barton classification. The maximum value specifies 16.883 for slightly weathered sandstone, and minimum value of 1.525 was observed for highly weathered sandstone. Moderately weathered sandstone was recorded with a range of 6.358 to 6.533. Shale shows no clear trend as highly weathered shale have higher Q-value (5.090 to 5.166) when compared to moderately weathered shale (3.853 to 4.905), while 2.696 was recorded for completely weathered shale. The values of RMR

ranges from 27 to 69, it signifies difficult excavation when the value is higher. The Q-system value range from 1.525 to 16.883. The higher value indicates the quality of rock increases resulted a more difficult excavation required. Overall, it appears that the value of Q-system and RMR decreasing trend as the weathering grade is higher for both sandstone and shale. This is consistent with Bieniawski (1989) that suggests RMR value of more than 60 as 'good rock' and requires blasting, while the lower value can be treated as 'fair rock'.

Table 2. Q-System Rating for Studied Area

Panel	Joint Set Number	Joint Spacing (m)	RQD (%)	J_n	J_r	J_a	J_w	SRF	Q
BLR 1	2	1.300	90.979	4	4	5	1	5	14.557
SAIV 1	2	0.600	50.367	4	4	5	1	2.5	4.029
BLR 2	2	1.200	89.251	4	4	5	1	5	16.883
SAIV 2	2	0.550	45.745	4	2	3	1	5	1.525
SAIII 2	2	0.750	75.327	4	4	5	1	2.5	6.426
SHV 3	1	0.350	22.153	2	4	3	1	5	2.696
SHIV 3	2	0.550	45.745	4	2	2	1	2.5	5.166
SHIII 3	2	0.100	73.576	4	4	3	1	5	4.905
SHIV 4	2	0.580	48.579	4	4	2	1	5	5.090
SHIII 4	2	0.100	70.467	4	4	3	1	5	3.853
SAIV 5	2	0.550	45.745	2	4	5	1	5	1.830
SAIII 5	2	0.100	73.576	4	4	2	1	5	6.358
SAIV 6	2	0.550	45.745	2	4	3	1	5	3.050
SAIII 6	2	0.750	75.327	4	4	2	1	5	6.533
SAII 7	3	1.200	89.251	9	3	2	1	2.5	6.550

Geophysical Interpretation

Figure 5 represents the resistivity image and profile of the Legoland (LEGO) outcrop. The images were synchronized to the outcrop geological map for interpretation (Nordiana, 2013). The total length of the survey line was 80m with 2m electrode spacing using Pole-dipole array. Results indicate that the whole study area were made up of zone with resistivity value range -100 Ωm to 12000 Ωm covered with a thin layer (<5m) of overburden which showed resistivity value of 500 Ωm to 5000 Ωm. Low resistivity zones with value <100 Ωm were detected at bottom part of relatively higher resistivity value speckled near surface with resistivity value >5000 Ωm at the location of 15m to 20m on the scanline.

Thin layer is detected at depth <5m and the resistivity section show that the area is made up of moderate resistivity value range. The study area is located at an excavated hilly slope area where there are unstable loosen soil formed from weathering process of bedrock. As 2D resistivity imaging survey lines were executed across through, there are lots of fractured and weathered materials that have been transported down slopes during the excavation process. Zones with low resistivity value which is <100 Ωm are also identified in this study area. These zones are may be due to the saturated zones where water was seen seeping through the rocks, therefore the resistivity value is lower than surrounding.

The presence of weathered boulders is distinguished scattered near surface of study area with resistivity value > 5000 Ωm. These weathered boulders have the potential to create the

unstable soil condition and could trigger mass movement. Regarding the geophysical interpretation, the results were then compared with onsite observation data of the surface outcrop. Resistivity methods are relatively good in detecting saturated zone in which moisture content is an important key. Besides that, resistivity can detect the existence of boulders where high resistivity speckled zone is detected.

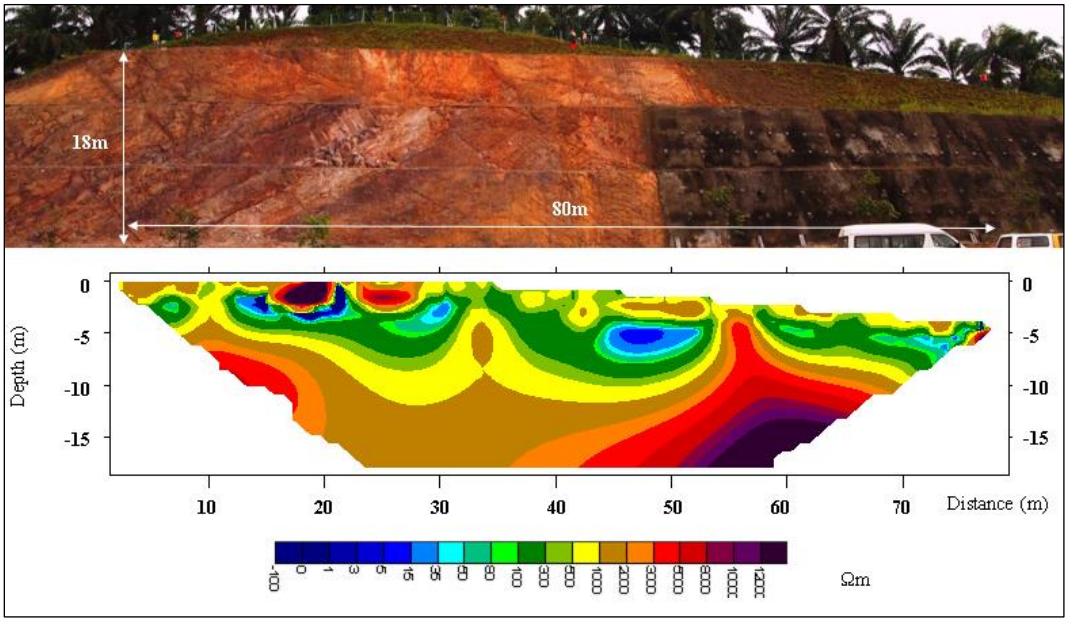


Figure 5. Resistivity Profile of Studied Area

Residual soils exhibit the lowest resistivity value of less than 1000 Ωm. The low value is also shared by completely weathered shale located at 40m to 50m on the survey line with depth less than 5m below ground surface. Highly weathered sandstone and shale of presented by the value between 100 Ωm to 3000 Ωm located 10m below the ground. On the other hand, moderately weathered sandstone and shale with 1000 Ωm to 5000 Ωm has higher range of resistivity compared to highly weathered zone. Slightly weathered zone consists of sandstone and boulder located near surface with speckled value up to 12000 Ωm.

Moderately weathering grade sandstone shows broader range of 1000 Ωm to 5000 Ωm while similar grade of shale exhibit range of 2000 Ωm to 5000 Ωm. Whilst sandstone of highly weathered shows 0 to 3000 Ωm compared to similar weathering grade of shale range from 500 Ωm to 2000 Ωm. The range for shale is narrower compared to sandstone. This finding is supported by Gomaa et al. (2014) which suggested the mineralogy composition, texture and structure, porosity and cementation affected the electrical properties. Dielectric constant decreases with frequency and increases with silt and clay composition. Thus, the conductivity increases with the increases of conductor paths between electrodes.

The survey lines were designed along the slopes to obtain subsurface information of the area and seismic velocity of the underlying layers. For Legoland seismic survey line (LEGOS), the total length is 115m with geophone spacing 5m. The location of first geophone situated at the same location of first electrode in electric resistivity survey. Velocity tomography model (see Figure 6) indicates that the site can be classified as two main velocity

layers which are upper layer and lower layer. The upper layer has velocity values ranges from 300 m/s to 1500 m/s, which can be categorized as surface deposits and mixtures of sands and gravels. The excavation works of the upper part of the site also signify high production rate and normal excavation method are done. Meanwhile, the lower layer has velocity values with range of 1500 m/s to 3500 m/s which is classified as compacted fine material due to high pressure of the overburden and indicated the hard layer of the site. At this layer, the excavation needed a few blasting to loosen the material or otherwise the excavator could not dig the material. It has been observed that the production of the excavations at this part are slow.

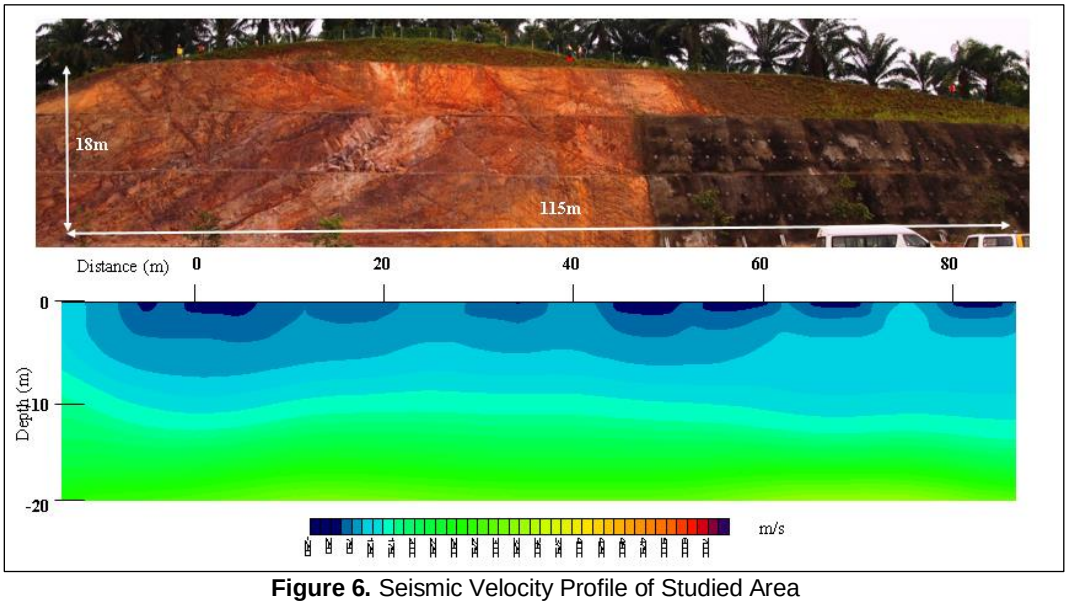


Figure 6. Seismic Velocity Profile of Studied Area

Following both geophysical data collection and analysis, ground proving information was collected at the site to provide partial verification of the geophysical results interpretations. Mass characterization were conducted at the site to gain further information of the subsurface hence assist in the interpretation. In this study, the weathering grade, strength, Schmidt hammer were determined on site. The observations are also performed on the discontinuities of the outcrop surfaces. Panels were analysed by recording the joint spacing, bedding, inflow water and the quality of other discontinuities parameters. The result show good correlation with the weathering grade of the outcrop mass. The summary of geophysical results for different weathering grade of studied rock mass are tabulated in Table 3.

Table 3. Geophyiscal Results for Different Weathering Grade of Sandstone and Shale for Studied Site

Material	Weathering Grade	Resistivity (Ωm)	Seismic Velocity (m/s)
Residual soil with saturated zone	VI	-100 – 50	300 – 750
Residual soil	VI	0 – 1000	300 –750
Sandstone	II	5000 – 12000	2500 – 3500
Sandstone	III	1000 – 5000	1500 – 3000
Sandstone	IV	100 – 3000	750 – 2000
Shale	III	2000 – 5000	2000 – 3000
Shale	IV	500 – 2000	750 – 2000
Shale	V	100 – 500	750 – 1250
Boulder	II	3000 – 12000	Not detected

Geophysical techniques are cost effective that attain generalized subsurface information for geotechnical or geological characterization over relatively large areas. The use of efficient equipment and procedures have been available to the geotechnical engineering profession to develop experience with implementation of its. This study is carried out with the implementation of resistivity and seismic refraction to enhance the interpretation of the result in order to compare with geological data in respect to excavation. The relationship for resistivity/seismic velocity vs Q-system/RMR were plotted in Figure 7, Figure 8, and Figure 9 and Figure 10. The summary of the Legoland data are tabulated below (Table 4). A resistivity survey was carried out on the same survey line as seismic refraction. The data produced significant correlations between the variety of indices. Generally, the trend provides the value of RMR and Q-system increase with the increases of weathering grade.

It is further indicating that the accuracy and reliability of the estimation is highly dependent on the parameters. Hence, the results are particularly depending on parameters such as joint set, joint fractured and moisture content. The results of residual soil (VI) and completely weathered shale has Q value of 0 to 2.296 and RMR ranging from 0 to 27. RMR of highly weathered sandstone range from 38 to 42 while value of 45 to 48 for highly weathered shale whilst the reading of Q-values are 1.830 to 4.029 for highly weathered sandstone and 5.090 to 5.166 for highly weathered shale. Moderately weathered sandstone and shale present their RMR range from 50 to 55 for sandstone and 53 to 55 for shale. Meanwhile, Q values display a range of 6.426 to 6.533 for moderately weathered sandstone and shale (3.853 to 4.905). Lastly, slightly weathered sandstone has RMR of 69 and Q value of 6.550. The RMR ranges between 0 and 69 in this study. It was recommended by Bieniawski (1989) that very good rock have a value of more than 80, whereby rocks having RMR values less than 40 are measured as poor rock.

The data distributions were found to be linear based on panels which designates different weathering grade of sandstone and shale. On the other hand, RMR system places too much emphasis on the discontinuity parameter, where 50% of the total rating are RQD and joints spacing that rated separately as stated by Rawlings et al. (1995). Similarly, Q-systems uses six parameters to classify rock mass. The values of Q range from 0 to 1000 indicating extremely poor to excellent conditions of rock. Generally, the Q-system is not appropriate in estimating the productions in sedimentary rock. Q-system very much rely on the rock structure by taking into account the number of joints set and RQD. Overall, the Q-system can be too conservative when the rock mass is soft and underestimate when the rock mass is hard.

When electrical resistivity results compared with RMR, some pattern could be observed. This phenomenon is obvious on completely weathered shale/sandstone and moderately weathered sandstone. It was found that when RMR less than 40 and Q less than 1 which the value indicated domination of highly to completely weathered sandstone and shale, the resistivity value is in the range of 100 Ωm to 3000 Ωm in contrast with the study by Ryu et al. (2011) which depicted < 1000 Ωm. Ryu et al. (2011) did not specify the weathering grade of the material and this study prove that the higher degree of weathering could results in wide variation of resistivity value.

The case shows different situation when $40 < RMR < 60$ and $1 < Q < 10$, consists of highly/moderately weathered sandstone and shale. It is also interesting to note completely weathered sandstone display similar performances. The resistivity value of sandstone and

shale is in the range of 100 Ωm to 5000 Ωm and 500 Ωm to 5000 Ωm respectively. The range is in agreeable with Ryu et al. (2011) findings which the study was carried out in Korea. Completely weathered sandstone with wider joint spacing (0.4 m to 1 m) cause the mass more difficult to be excavated. Joint properties are the features that impacts the excavation performances as discussed in Tajul Anuar and Ismail Yusoff (2003) and Trisugiwo et al. (2016). Wide joint spacing of < 2 m able to hinder easy excavation even though in the completely weathered sandstone.

Besides resistivity, seismic velocity parameters are also investigated in this study. Nourani et. (2017) and El-Naqa (1996) agree that P-wave velocity have a great relation with RMR and Q. Barton (1991) study the correlation between P-wave velocity and Q-value whereas Cha et al. (2006) and Zafirovski et al. (2012) explore the correlation between RMR with shear wave velocity for subsurface. They found Vp could be corelated with RMR with fair to good relationship. Hence, the comparison is carried out on the parameter for rock mass classification RMR, Q and velocity for example the intact rock relation with P-wave done by McDowell (1993), RQD and compression wave by Bery and Saad (2012). Some variation is noticed when seismic velocity is plotted against RMR and Q especially on completely and moderately weathered sandstone and shale. When RMR less than 40 and Q less than 1, the seismic velocity ranges from 250 m/s to 1250 m/s which conquered by completely weathered sandstone and shale.

Nowadays, the application of seismic velocity and resistivity are widely used in engineering projects, geological engineering, and civil engineering (Carrozzo et al.,2008; Azwin et al., 2015; Lapenna et al., 2003). seismic profile interpretation unaided does not explain whether the velocity variation related to its geomechanical properties. However, this uncertainty can be determined by observing deeper into the correlation between both geophysical method (resistivity and seismic velocity) to its geomechanical properties regarding excavation performances. As agreed by Carrozzo et al. (2008), all of the parameters involved in a classification system such as Q and rock mass rating (RMR) affect the seismic wave velocity as well. Therefore, it is possible to obtain the variations of Q and RMR as a function of the seismic wave velocity parameters.

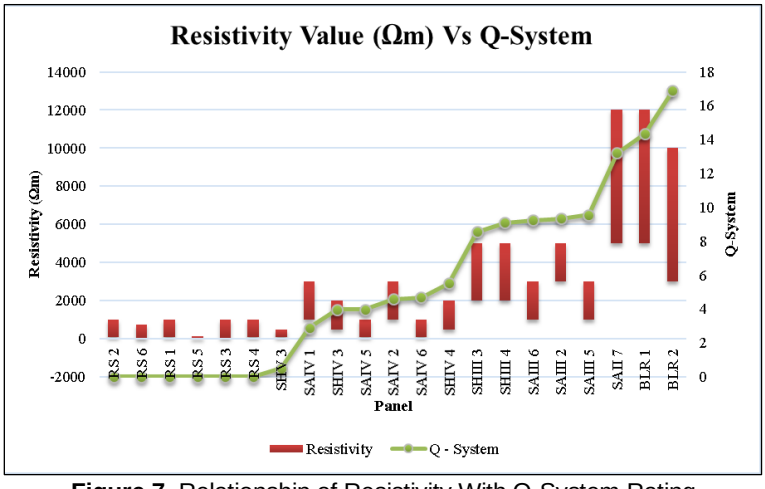


Figure 7. Relationship of Resistivity With Q-System Rating

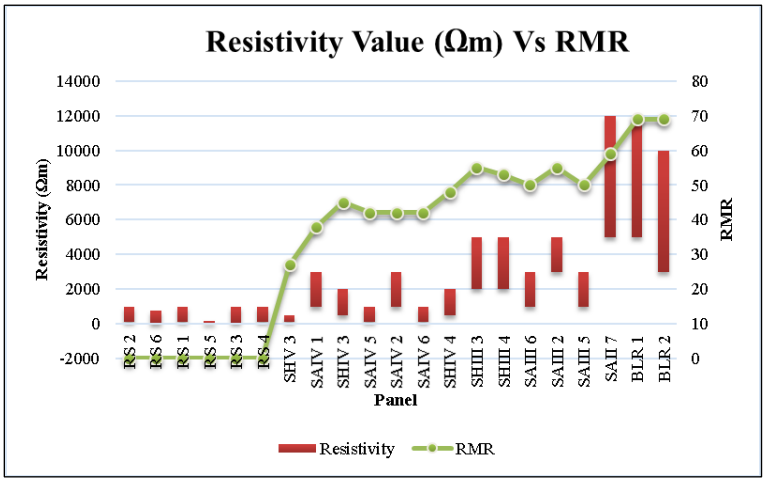


Figure 8. Relationship of Resistivity With RMR

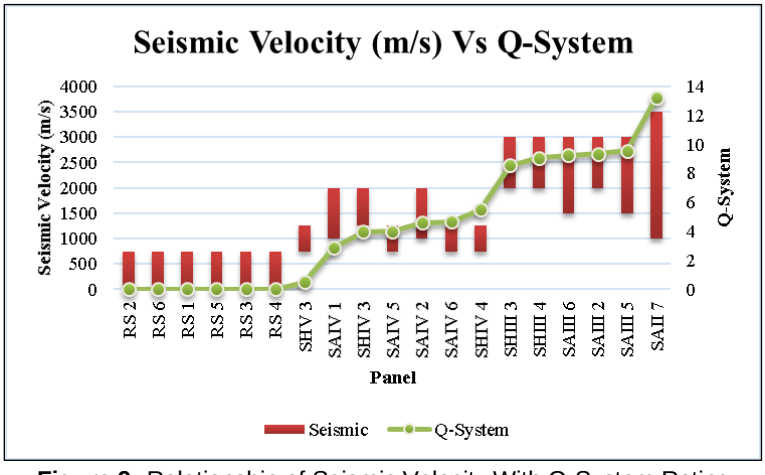


Figure 9. Relationship of Seismic Velocity With Q-System Rating

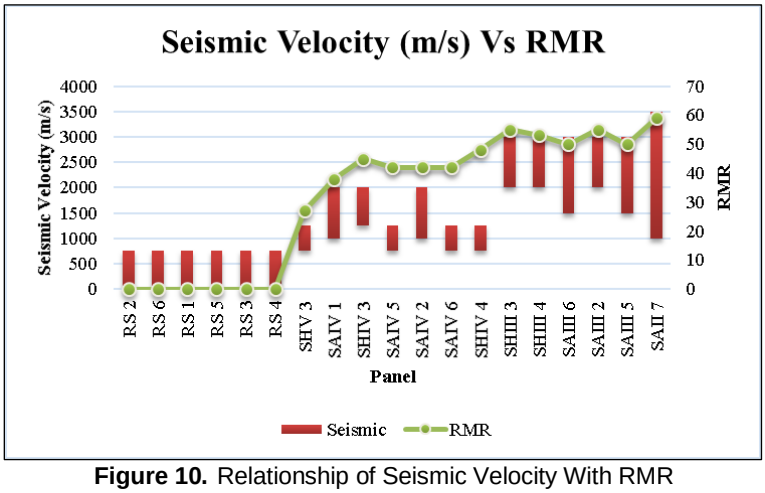


Figure 10. Relationship of Seismic Velocity With RMR

Table 4. Summary of Rock Mass Classification and Geophyiscal Values of Studied Area

Material & Weathering Grade	Rock Mass Rating (RMR)	Q-System Rating	Resistivity (Ωm)	Seismic Velocity (m/s)
Residual Soil (VI)	0	0	< 1000	< 750
Residual Soil (VI) Saturated zone	0	0	< 50	< 750
Sandstone (II)	69	6.550	5000 - 12000	> 2500
Sandstone (III)	50 - 55	6.426 - 6.533	1000 - 5000	1500 - 2000
Sandstone (IV)	38 - 42	1.525 - 4.029	100 - 3000	750 - 2000
Boulder (II)	69	14.557-16.883	5000 -12000	Not detected
Shale (III)	53-55	3.853-4.905	2000 - 5000	2000 - 3000
Shale (IV)	45-48	5.090-5.166	500 - 2000	1250 - 2000
Shale (V)	27	2.696	100-500	750 - 1250

When electrical resistivity results compared with RMR, some pattern could be observed. This phenomenon is obvious on completely weathered shale/sandstone and moderately weathered sandstone. It was found that when RMR less than 40 and Q less than 1 which the value indicated domination of highly to completely weathered sandstone and shale, the resistivity value is in the range of 100 Ωm to 3000 Ωm in contrast with the study by Ryu et al. (2011) which depicted < 1000 Ωm. Ryu et al. (2011) did not specify the weathering grade of the material and this study prove that the higher degree of weathering could results in wide variation of resistivity value.

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Nowadays, the application of seismic velocity and resistivity are widely used in engineering projects, geological engineering, and civil engineering (Carrozzo et al.,2008; Azwin et al., 2015; Lapenna et al., 2003). Seismic profile interpretation unaided does not explain whether the velocity variation related to its geomechanical properties. However, this uncertainty can be determined by observing deeper into the correlation between both geophysical method (resistivity and seismic velocity) to its geomechanical properties. As agreed by Carrozzo et al. (2008), all of the parameters involved in a classification system such as Q and rock mass rating (RMR) affect the seismic wave velocity as well. Therefore, it is possible to obtain the variations of Q and RMR as a function of the seismic wave velocity parameters.

CONCLUSION

Geophysical survey is a useful tool for subsurface study. In this study, both resistivity and seismic surveys are performed on the site. Geophysical equipment is known as portable and relatively inexpensive. The technique uses to interpret the data along the survey line able to provide greater coverage of subsurface profile. The profile can be used to enhance the existing rock mass classification system to ease the site investigation process. Somehow, both geophysical tests have their own advantages and disadvantages. Resistivity have higher potential in detecting boulders besides sensitive in detecting high weathering grade material and saturated zone which cannot be obtain by seismic refraction survey. Besides, it also can detect different type of material in small hard layer. The results show significant changes of range in the velocity and resistivity in different weathering grade of the materials. It can be concluded that there is a strong correlation between seismic velocity and resistivity value with the rock mass classification system. Linear relationship between weathering grade and rock mass classification systems are used to improve the confident level in applying geophysical survey. The higher the rock mass classification rating, the lower the weathering grade of the material. Seismic profile interpretation unaided does not explain whether the velocity variation related to its geomechanical properties. However, this uncertainty can be determined by observing deeper into the correlation between both geophysical method (resistivity and seismic velocity) to its geomechanical properties regarding site investigation parameter. Therefore, it is possible to obtain the variations of Q and RMR as a function of the seismic wave velocity parameters.

ACKNOWLEDGEMENT

The authors are grateful to Office for Research, Innovation, Commercialization and Consultancy Management (ORICC), Universiti Tun Hussein Onn Malaysia for research and innovation fund, TIER 1 VOT H174. The authors also express their sincere thanks to all technical staffs of Department of Transportation & Geotechnic, Universiti Teknologi Malaysia (UTM), all technical staffs and postgraduate of School of Physics, Universiti Sains Malaysia (USM) for the data collection during the study.

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PERFORMANCE OF PALM OIL FUEL ASH (POFA) IN PEAT SOIL STABILIZATION

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Abstract

A study on fibric peat soil at Pontian, Johor was conducted to determine its physical properties, chemical properties and engineering properties. Due to low strength of peat soil, cement stabilization was widely used to enhance the soil performance. However, abundant amount of cement was required in peat stabilization compared to other types of soil and cement production would lead to air pollution. Palm Oil Fuel Ash (POFA) as a profitless industrial by-product was recommended to substitute some of the cement as it possesses cementitious behaviour when subjecting in cement hydrating process. 300 kg/m^3 of optimum binder dosage of cement (C100) used as control specimen in peat stabilization. While the partial replacement of cement dosage in four others mixing proportions were replaced with 5 %, 10 %, 15 %, and 20 % of POFA. Scanning Electron Microscope Test (SEM) and Energy Dispersion Spectroscopy X-Ray (EDX) were applied to observe the changes in chemical composition and particles arrangement before and after treating using POFA. At 28 curing days, C85P15 achieved the highest value in unconfined compressive strength (UCS) test which was 107.6 kPa. In comparison to C100 and C80P20, C85P15 had lower carbon content, higher calcium content and lower silica and alumina content. Therefore, the ability of POFA to enhance the peat strength and effectively reduce the cement usage in peat stabilization is proved.

Keywords: *Peat stabilization; Palm Oil Fuel Ash; peat strength.*

INTRODUCTION

Peatlands is one of the major land types in Southeast Asia. Approximately 35 million ha in the region with the majority in Indonesia, Malaysia, Brunei Darussalam, Thailand, Vietnam and smaller areas in Myanmar, Lao PDR and the Philippines covered with this kind of land (Adon, Bakar, Wijeyesekera & Zainorabidin, 2012; *Distribution and Status of Peatlands in Malaysia*, 2019; Wahab, Talib & Rohani, 2019). These wetland forests mainly developed in the coastal lowland plains in between major rivers. According to Hooijer et al. (2010), a more precise estimation was attempted in a Southeast Asian regional study which assumes an average thickness value of 3m for peatlands in Malaysia. In Peninsular Malaysia, peat soil mostly was found in the states of Selangor, Johor, Perak, and Pahang. Despite Pahang has the largest peatlands area, Johor has the greater peatlands area used for agricultural purposes as shown in Figure 1.

According to Hamzah et al. (2018), peat soils or known as soft soil is a problematic soil refers to a highly organic soil with an organic content more than 75 % which low strength and high compressibility (Basri, Wahab, Talib & Zainorabidin, 2019) and which is always in waterlogged condition that cause less decomposition of organic matter from plant liter and the latter is then accumulates as peat over thousands of years with insufficient oxygen (Adon et al., 2012; Raghunandan & Sriraam, 2017). Tang, Bakar and Chan (2011) regarded peat as a problematic deposit with poor geotechnical properties, which invariably requires

stabilization if it is to support any structure or infrastructure in the context of construction engineering. Abandoned sites usually happened to the peat soil areas during the site selection for construction uses. However, this option might lead to land scarcity and thus many researchers proposed various solutions to improve the bearing capacity of the problematic soil. Normally the engineers will solve this problem by the removal of such organic soil and its replacement with mineral soil. This is not an economical method as there is wastage of soil and the cost for cut and fill will increase the whole project cost. Besides, compaction associated with drainage system conducted to reduce the water level of peat area. Rogers and Glendinning (1996) and Makusa (2012) stated that adding binder to the weak soils could efficiently improve the gradation of particle size and eventually achieved further improvement of the soils. Other than that, cement stabilization is broadly used recently to improve the strength of soil. Cement could efficiently enhance the geotechnical properties of soft soil such as clay and peat soil. However, the humic acid in peat soil lead to abundant amount of cement needed in soil stabilization compared to other types of soils. The adverse impact of this solution is concerned to the environmental issues emerged in cement production. According to Andrew (2018), cement production is the third largest anthropogenic emissions of carbon dioxide in global.

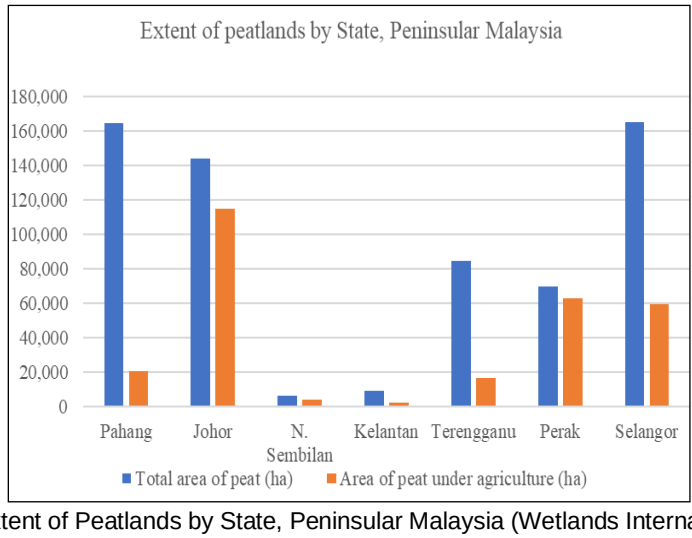


Figure 1. Extent of Peatlands by State, Peninsular Malaysia (Wetlands International, 2010)

On the other hand, approximately 100 million tonnes of oil palm fruits, oil palm and palm kernels produced annually in Malaysia based on data generated in FAOSTAT. Indonesia is the largest palm oil producer in world but in term of yield production, Malaysia is the first in world as shown in Table 1 and Figure 2. 4.5 million tons of waste per year produced by the palm oil industries Malaysia as they are being the second largest palm oil industries in the year 2010 (Zarina, Al Bakri, Kamarudin, Nizar & Rafiza, 2013). According to Muthusamy, Zamri, Kusbiantoro, Lim, and Ariffin (2018), POFA in palm oil mill area is disposed at open environment so that it can be easily carried by the wind. These abundant profitless solid waste or industrial by-product is known as Palm Oil Fuel Ash (POFA) that may contribute to serious environmental problem and harmful to mankind if not disposed properly.

According to Hassan, Ismail, Ismail, Lee and Hussin (2020), POFA has a potential in enhancing the properties of concrete which are able to generated secondary calcium silicate

hydrate (C-S-H) and act as the supplementary cementitious material (SCM) which increase the density and durability of the cement mortar (Hamada, Jokhio, Yahaya, Humada & Gul, 2018). Ahmad, Rahman, Ali and Rahman (2011), it is found that POFA contains pozzolanic materials which possess cementitious behavior which can substitute cement in peat soil stabilization. Pourakbar, Asadi, Huat and Fasihnikoutalab (2015) declare the utilization of POFA in the micro and nano form turn out to be high-performance concrete which provided an excellent improvement in the compressive strength.

Results obtained recorded peat soil added with 20 % of Ordinary Portland Cement and 10 % of POFA achieved higher Unconfined Compressive Strength than peat stabilized with only 30 % cement. Ahmad et al. (2011) conducting a study on the effect of partial replacement of cement to the peat stabilization by applying the by-product (POFA). The results record the strength was increased with replacement cement 20 % + 10 % POFA compared to peat stabilized with cement only. It also states that POFA is good to replace with cement but still cement percentage is a dominant. Otoko, Fubara-Manuel, Chinweike, and Oyebode (2016) presented that 5 % lime together with 3 % POFA could achieved the highest CBR values in Chikoko subgrade stabilization compared to 0 %, 2 %, 4 % and 5 % of POFA used. Next, Fitri Amzar and Suwandi (2006) also found that 14% of 300 µm sieved POFA could improve the shear strength of clay soil up to 225.23 kPa.

Table 1. Production of Palm Oil in Malaysia and Indonesia (FAOSTAT)

Country	Production (million tonnes)			Area Harvested (million ha)		
	2015	2016	2017	2015	2016	2017
Malaysia	98.3	86.3	101.7	4.9	5.0	5.1
Indonesia	148.8	156.9	158.3	8.6	9.2	9.3

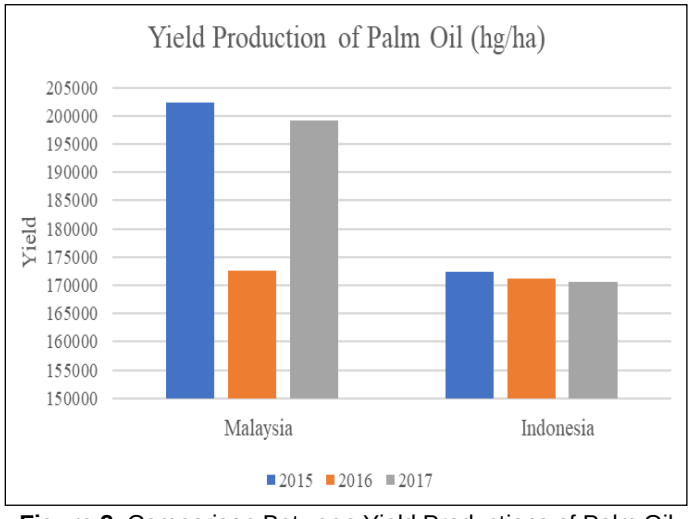


Figure 2. Comparison Between Yield Productions of Palm Oil

Regarding to land scarcity and environmental problem due to cement production and POFA dumping methods, it is significant to develop a sustainable binders to overcome these obstacles. In this study, different percentages of POFA will be used to replace cement in peat soil stabilization in order to obtain the optimum mix proportion of cement and POFA. The main objectives of this research are to determine basic properties of Palm Oil Fuel Ash

(POFA) and peat soil, investigate the effect of POFA in peat soil stabilization with cement, and evaluate peat strength mechanism based on the chemical analysis such as Scanning Electron Microscopy (SEM) with Energy Dispersive X-Ray Analysis (EDX).

EXPERIMENTAL STUDIES

The three main materials utilized in this research were peat soil, Palm Oil Fuel Ash (POFA) and Ordinary Portland Cement (OPC). Peat samples were collected from Medan Sari, Pontian in the Johor region, Malaysia. While POFA samples were brought from FGV Holdings Berhad, Kluang. POFA was then subjected to ball mill process before stabilizing and mixing process to fulfill the standard of pozzolanic material. Total of 4500 rotations (5 x 900 rotations) were applied on 5 kg POFA using 18 steel balls where the process took around 2.5 hours. After that, POFA were sieved through the sieve with opening aperture size of 150 µm to obtain finer particles and achieve uniformity of POFA which may affect the results of engineering properties.

All the laboratory test regulation and standards were adopted from Bristih Standard (BS) and American Society for Testing and Materials (ASTM) Standard. The basic properties of Pontian peat were analysed using various laboratory tests according to specified standards as shown in Table 2. These tests were significant to classify the types of peat and its geotechnical behaviors.

Table 2. Standards for Laboratory Tests

Test	Symbol	Standard
Moisture Content	MC	BS1377: Part 2:1990:3.2
Organic Content	OC	ASTM D2974-87
Fiber Content	FC	ASTM-D 1997-91 (2001)
pH Value	pH	BS1377: Part 3:1990:9
Liquid Limit	LL	BS1377:Part 2:1990:4.3/4.4
Specific Gravity	G_S	BS 1377: Part 2:1990:8.3
Particle Size Analysis	PSA	ASTM-D 422
Unconfined Compressive Strength	UCS	ASTM-D 2166

The physical properties of the entire samples in this study were conducted at the beginning before further with engineering properties. The natural water content is determined by drying a peat sample in oven under 105˚C temperature for 24 hours. Ash content is obtained by igniting the oven-dried sample in a Digital Muffle Furnace (model FHX-63) at 440˚C about 4 to 7 hours or until no change in mass. The ash content is defined as the percentage of inorganic materials content in peat after removing all the organic materials and water. Then, subtract the percentage of the ash content from one hundred to get the value of organic content for peat. To understand the percentage of fiber content of peat soil, the specimens were stirred approximately about 100 g of undried peat thoroughly with 5 % sodium hexametaphosphate and let it stand for at least 15 hours. Poured over a 150 µm stainless steel sieve and washed the peat using rubber hose attached to a water faucet until water passing through the screen is clear. Washed all the fiber off the screen after inverting screen over a large funnel containing a piece of filter paper that known mass. Removed the filter paper contains the fiber fragment and dried it in the oven at 105°C about 24 hours. Last, deducted the mass of filter paper to get mass of the fiber and expressed it as a percentage of the oven-dried mass of the original sample.

Meanwhile, for specific gravity test it were starts by transferring 10 g of oven dried sample (sieved 2 mm) into pycnometer. In this study, kerosene is used instead of distilled water as the kerosene had lower density than peat soil and thus peat soil would sink at the bottom. 10 mL of kerosene is poured slowly into the bottle until the soil soaked completely and leaved for 1 hour. Again filled the bottle fully with kerosene, put into dessicator for an hour and recorded the mass. Lastly, emptied the bottle and thoroughly cleaned it. Filled again the bottle with only kerosene and recorded the mass. Eutech pH 2700 is used to measure the pH of peat by placing the electrode to the moist peat and begin reading. On the other hand, liquid limit test is conducted by mixing the 425 microns sieved sample with distilled water until it reached appropriate moisture level. Adjust the tip of cone penetrometer to the surface of peat and released with the time set of 5 seconds. Several readings are taken to obtain the average cone penetration readings. Penetration graph produced by plotting penetration readings versus moisture content and the projection of the line on the penetration of 20 mm is the liquid limit of Medan Sari peat.

In this study, the unconfined compressive strength (UCS) tests as the fastest, simplest, easiest and least expensive method had been operated on all samples in order to demonstrate the stabilized peat strength improvement. Scanning Electron Micrographs (SEM) was conducted to illustrate the micro visual evidence on cementation and void enhancement of the stabilized peat (Talib, Yasufuku and Ishikura (2015). Combined with the application of Energy Dispersive X-ray (EDX), the percentage of chemical composition of raw materials and peat samples were obtained.

Table 3. Binders Proportion in Peat Soil Stabilization

Binder Proportion	Symbol	Quantity of Samples
Cement 100%	C100	3 samples x 3 curing days = 9 samples
Cement 95% + POFA 5%	C95P5	3 samples x 3 curing days = 9 samples
Cement 90% + POFA 10%	C90P10	3 samples x 3 curing days = 9 samples
Cement 85% + POFA 15%	C85P15	3 samples x 3 curing days = 9 samples
Cement 80% + POFA 20%	C80P20	3 samples x 3 curing days = 9 samples
Total		45 samples

Table 4. Quantity of Materials Needed

Sample Code	Peat Soil, g	OPC, g	POFA, g
C100	300	88	0
C95P5	300	83.6	4.4
C90P10	300	79.2	8.8
C85P15	300	74.8	13.2
C80P20	300	70.4	17.6

Totally 5 mixture proportion were prepared in this research as shown in Table 3. While the quantity of materials needed for each mixture proportion was shown in Table 4. Natural water content of peat soil was maintained during the preparation of all treated peat samples. 300 kg/m3 of OPC dosage was used for the control peat samples (C100) which was similar to the OPC dosage implemented by (Dehghanbanadaki, Ahmad & Ali, 2013; Talib & Noriyuki, 2017). The 300 g of peat and 88 g of OPC (C100) used based on the threshold of optimum binder dosage had done by the previous researcher (Talib et al., 2015) which in the range 300- 350 kg/m3 respect on the unit weight and water content of original peat. Chen and

Wang (2006), Sing, Hashim and Ali (2009), and Talib and Noriyuki (2017) described the insufficient of cement binder dosage in the peat stabilization will retard the hydration and pozzolanic reaction since peat is widely known as highly acidic.

All the mixing and sampling process were followed the specifications in (EuroSoilStab (2002). In making the control admixture, 300 kg/m3 of OPC was mixed thoroughly with untreated peat for approximately 5 minutes using laboratory mixer. Mixing time for fibrous peat could not be long as it might destroyed fibers in soil. After that, inserted the well mixed peat into PVC pipe for 7 layers and each layer tamped for 20 times. The PVC pipe used as mould was 50 mm in diameter and 150mm in height. To ease the samples removal, PVC cylinder tubes were split into two half and then taped and tied together using binding tapes and cable ties. Next, all the peat specimens subjected to wet curing processes to represent the real peat site condition. After wet curing for 7, 14 and 28 days, peat samples were taking out and cut to 100mm height for UCS test.

RESULTS AND DISCUSSIONS

Materials Properties

After conducting all the laboratory tests, the results were analysed and summarized in Table 5. The basic properties ranges for fibric peat soil in Malaysia were collected from published data (Deboucha & Hashim, 2009; Islam & Hashim, 2010; Kolay, Aminur, Taib & Zain, 2011; Rahman & Chan, 2014) to enhance the liability of results and used for guidance purpose.

Table 5. Medan Sari Peat Soil Properties

Laboratory Test	Value	Published Data
Moisture content, %	938.321	350 - 850
Ash content, %	3.530	-
Organic content, %	96.487	88.61 – 99.06
Fiber content, %	72.319	68.2 – 90.49
Specific gravity, Gs	1.208	0.95 – 1.4
pH value	4.12	3.27 – 4.6
Liquid limit (LL), %	419	173.75 – 208.39
UCS, kPa (raw peat)	8.65	-

At Medan Sari site during site visit, Von Post was carried out on the peat soil and it was found to fulfil the description for H3 (fibric peat type). Figure 3 shows the peat soil during squeezing and after squeezing. Muddy brown water was released when peat soil squeezed but there was no peat passes between the fingers. H3 is considered as fibric peat soil which has very high fiber content as it is very slightly decomposed. After conducting fiber content test, it is found that the peat has fiber content more than 66 % and pH value at 4.12. According to Jarrett (1995), peat soil that has fiber content more than 66 % is classified as fibrous peat with low degree of decomposition and contains easily recognized plant structures. High water content and liquid limit accompanied with low unconfined compressive strength indicates that Medan Sari peat has high compressibility and instability characteristics. The unconfined compressive strength of cohesive soil, qu means the load per unit area recorded once the unconfined cylindrical soil specimen fails under a simple compression test. The reading taken

is either the maximum load attained per unit area or the load per unit area at 15 % axial strain, depends on which one obtained first during the unconfined compression test. The peak value of raw peat at 20 % of axial strains was 9.744 kPa but the value at 15 % was taken as its UCS value. UCS of peat rises steadily over the increasing strain and peat does not show any crack during deformation due to its elastic properties.

Before testing for SEM-EDX tests, the samples were sieved to 150 micron and proceeded to gold coating using FISON SEM Coating System as shown in Figure 4. Sputtered coating could reduce microscope beam damage, increased thermal conduction and improve the conductivity of the sample surfaces. The magnified scale used was X2000. SEM-EDX tests are significant to observe the changes in both microstructure arrangement and chemical composition of treated and untreated peat could be compared and the effect of POFA. Hitachi VP-SEM SU 1510 is used to carry out SEM-EDX tests.

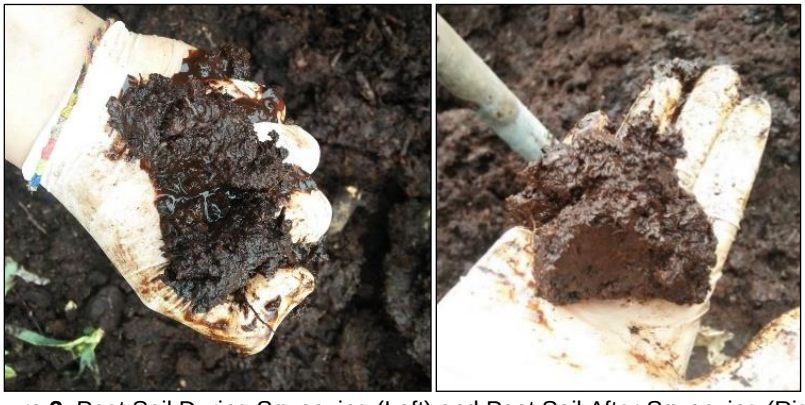


Figure 3. Peat Soil During Squeezing (Left) and Peat Soil After Squeezing (Right)



Figure 4. FISON SEM Coating System (Left) and Hitachi VP-SEM SU 1510 (Right)

Table 6. Oxide Compounds of Peat and Materials

Oxide Compound (%)	Peat	OPC	POFA
C	75.22	-	-
O	23.56	-	-
Si	0.33	16.98	70.27
Al	0.22	3.85	4.39
Fe	0.30	1.84	2.74
Ca	0.14	70.70	7.25
$SiO_2 + Al_2O_3 + Fe_2O_3$	0.85	-	77.4

The contents of pozzolanic minerals (sum of SiO2, Al2O3 and Fe2O3) in peat soil are very low with only 0.85 % as shown in Table 6. According to Isaia, Gastaldini, and Moraes (2003) and Isaia et al. (2003), the dominant element in OPC is CaO2, SiO2, Al2O3 and Fe2O3. The cement used in this study has the calcium content up to 70.70% which is much higher than the range collected from other researchers. For POFA, it was found that the summation of crucial pozzolanic oxide compounds (SiO2, Al2O3 and Fe2O3) is 77.4 % of the total oxide compounds and thus it satisfied the requirements in ASTM C618. ASTM C618 is widely used in determining the required specification for natural pozzolan and indicating the suitability of materials as a pozzolan. POFA fall in the range as Class F pozzolan.

The particle size distribution of raw peat and other materials are illustrated in Figure 5. The particle size distribution curves for peats were obtained by wet sieve method accompanied with particle size analyzer model CILAS 1180. This machine is aided with laser diffraction and CD camara and thus it able to measure the whole size range without any mechanical adjustment or optical realignment. It is also applied on POFA and OPC which has pass through 150 micron sieve. More than 95 % of peat is finer than 4.75 mm and around 86% has passed through 75 micron sieve. The percentage of cement particles that passes through 45-micron sieve is 97.2 %. According to ASTM C150, cement that retained on 325 mesh opening must not more than 24 %. Thus, the percentage retained in this study is only 2.8 %, means that the cement used has fulfilled the requirement in ASTM C150. Percentage of POFA that passed through 45 microns was 100 % which was more than 66 % as required in ASTM C618. The POFA utilized in this study had been undergone ball milling process and thus it has finer particle size. The particle size of POFA ranged from 0.04 microns until 36 microns.

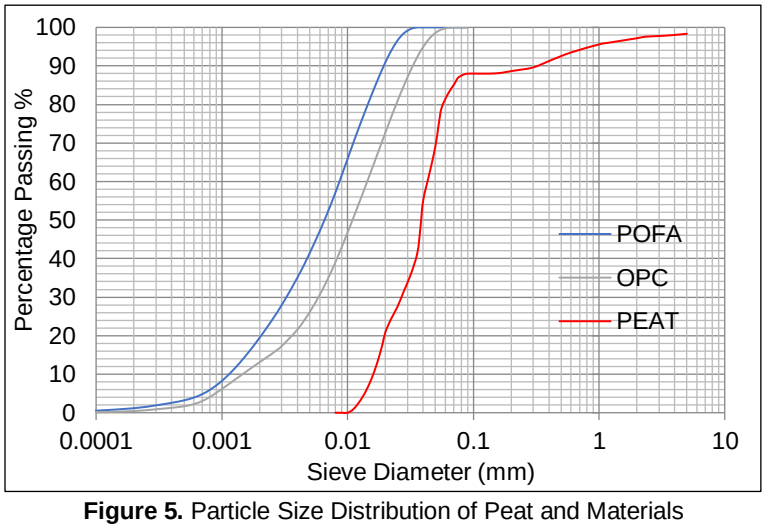


Figure 5. Particle Size Distribution of Peat and Materials

Effect of Partial Replacement of OPC With POFA on The Strength Characteristics

Table 7 presented the results of unconfined compressive strength of treated peat samples with different mixture ratios and curing days. For each curing days, C85P15 attained the highest value for unconfined compressive strength compared to the other mixture ratios and thus it is the optimum mixture. While C100 obtained the lowest strength for 7, 14 and 28 days

among all the treated peat specimens where it is 1.3 times lower than C85P15 after curing for 14 and 28 days. The UCS value decreases when 20 % of POFA is used in the stabilization. Hamada et al. (2018) have been reported some researchers conducting various proportions ranging from 10 % to 20 % of cement replacement by POFA to be optimum for the compressive strength of concrete. Based on Pourakbar et al. (2015), 15 % binder dosage of POFA substituted with cement shows the treated soil led to an increasingly dense from the microstructure properties. POFA itself can slightly improve the strength of treated soil. However, the partial replacement of POFA in cement can boosting the strength because of cement hydration, which serves as a POFA activator. It is discovered by Sooraj (2013), that POFA is an excellent pozzolan and can be applied as a partial substitute for cement in concrete.

Table 7. Unconfined Compressive Strength of All Treated Peat Samples

Mixture Ratio	Unconfined Compressive Strength (UCS), kPa		
	7 Days	14 Days	28 Days
C100	31.544	55.730	82.089
C95P5	35.152	62.887	97.986
C90P10	35.537	64.622	99.013
C85P15	38.713	71.758	107.590
C80P20	35.400	69.347	102.230

C100 has lower unconfined compressive strength than the samples stabilized using both POFA and cement as the peat soil consists of high organic content and less solid particles. Huttunen, Kujala and Lehto (1996), and Rahmi, Taib and Sahdi (2018) stated that humic acid is one of the factors that influence the stabilization effectiveness where it can retard the strength development of cement-stabilized peat. Axelsson, Johansson, and Andersson (2002) and Talib and Noriyuki (2017) indicated that quantity of binder used must be exceeded a threshold to neutralize the humic acid in soils with high organic contents. According to Chen and Wang (2006) and Zulkifley et al. (2014), the combination of humic acid with calcium ions generated during cement hydration process will retard the calcium crystallization which is significant for the increase of peat soil-cement mixture strength to take place.

The chemical reactions between cement and pozzolan with peat water are shown in Equation 1 and Equation 2. For C100, only reactions in Equation 1will take place as there is no any pozzolanic materials exist. Cement hydration is defined as the process of mixing cement with water and other components for a desired application resulting into hardening phenomena. The hardening of cement will not change the soil structure, but it only encloses the soil as glue. Cement hydration can be affected by the chemical composition of cement, cement type, sulphate content, fineness of cement, presence of foreign matters or impurities, water-cement ratio, curing temperature, presence of admixtures and the specific surface of the mixture. C-S-H gel or also known as tobermorite gels forms a continuous layer that binds together the original cement particles into a cohesive whole during cement hydration, thus enhancing the engineering properties of cement paste. Calcium hydroxide is produced during the Portland cement hydration process where it might reduce the strength of peat soil.

$$C_3S \text{ @ } C_2S + H_2O \rightarrow C - S - H + Ca(OH)_2 \quad (1)$$

$$Ca(OH)_2 + \text{Pozzolan} \rightarrow \text{Secondary C-S-H (Gel) @ C-A-S-H} \quad (2)$$

As shown in Table 7, unconfined compressive strength of all treated peat samples increases with the increase curing days. Talib et al. (2015) mentioned that an increase in the unconfined compressive strength at 7 curing days was mainly contributed by the combination of filler effect and pozzolanic activity of pozzolan material and hydration reaction of the cement. Optimal effect of hydration reaction has occurred when 15 % of POFA is utilized probably due to filler effect and secondary pozzolanic reaction. Filler effect of the smaller particles in the mixture will significantly affect the pozzolanic effect beside the pozzolanic reaction (Isaia et al., 2003). When pozzolanic material such as POFA is added into the sample mixtures, silica or alumina oxide in POFA will react with calcium hydroxide to form secondary calcium silicate hydrate or calcium alumina silicate hydrate that contribute to strength of peat soil as shown in Equation 2. High content of silica and alumina in POFA also act to counterbalance the humic acid in peat soil and then create an alkaline condition and ease the secondary pozzolanic reaction to take place within the cemented soil. Due to the increased of C-S-H compound, peat soil becomes stronger and denser. Pozzolanic reaction mostly depends on the calcium hydroxide that released during hydrations of calcium silicates. The results in Table 7 indicates that optimal mix design (C85P15) could stabilize the peat effectively as the fine particles of pozzolan fill up pre spaces of the cemented soil, making it closely packed and more tobermorite gels produced during cement hydration.

Wet curing process ensures the cement could fully hydrate. Figure 6 revealed that moisture content for every mixture ratio is reduced gradually from being cured for 7 days until 28 days. The higher the percentage of POFA used, the higher the moisture content. This is probably due to higher percentage of cement used up more water in cement hydration process. According to Mohamed, Wurochekke, Chan and Kassim (2014), utilization of agricultural wastes such as POFA as stabilizer will tend to have higher water content compared to control specimen.

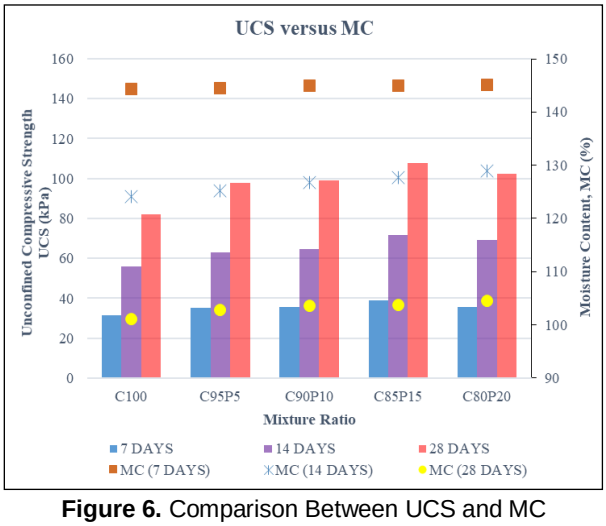


Figure 6. Comparison Between UCS and MC

SEM-EDX tests was conducted on the treated peat samples with curing days of 14 days. After subjecting to moisture content tests, the oven-dried samples were sieved to 150 micron and used for SEM-EDX tests. Figure 7 illustrates the results of SEM test on untreated and some stabilized peat Ssamples. The untreated peat is loosely composed of coarse organic particle and fibres in random manner. According to Talib et al. (2015) and Wong, Hashim

and Ali (2013), these coarse organic particles were hollow and spongy which lead the untreated peat to be highly compressible and high water holding capacity when fully saturated. Significant pore improvement can be observed in Figure 7 for stabilized peat compared to untreated peat. Pozzolanic activity of POFA leads to tiny pore spaces in well cemented peat. White shiny spots in C80P20 postulated that the POFA percentage is too high for the cement replacement, causing the silica to be excessive in peat samples.

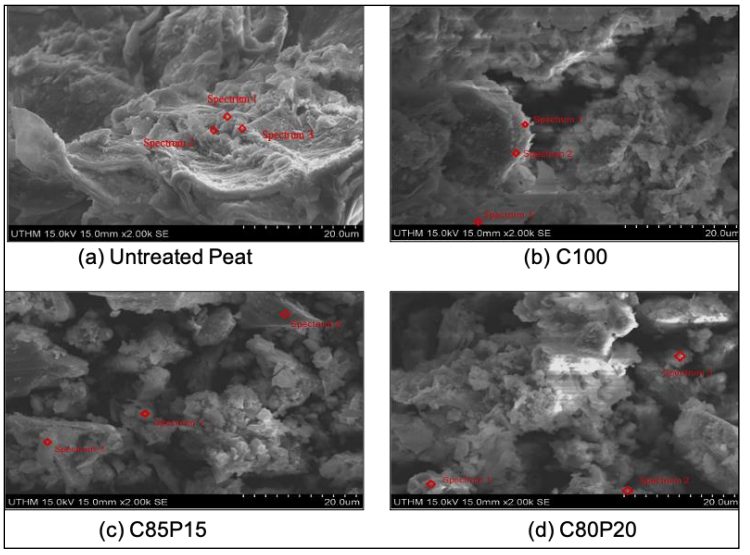


Figure 7. SEM Analysis on, (a) Untreated Peat; (b) Stabilized Peat C100, (c) Stabilized Peat C85P15; (d) Stabilized Peat C80P20

The summary of oxide compound percentages of untreated and stabilized peat from EDX test is tabulated in Table 8. C85P15 has the lowest percentage of silica and alumina compared to C100 and C80P20 most probably because of there are more pozzolanic reaction occurred. According to Sing et al. (2009), the secondary tobermorite gels will able to densify the stabilized peat and further improving its strength and thus C85P15 achieved the highest strength. C85P15 also has the highest calcium content which means cement hydration successfully happens. On the other hand, for C80P20, the percentage of calcium drops while silica and alumina content increase and it has lower UCS value than C85P15. Based on the research of (Talib et al., 2015), this might because of the limited formation of primary cementation products and thus insufficient amount of calcium hydroxide (CH) that could react to form secondary tobermorite gels during pozzolanic reaction. Excessive of pozzolanic mineral exist in C80P20 however there is limited amount of calcium hydrate ions available for secondary pozzolanic reaction to take place.

Table 8. Summary of Oxide Compound Percentages of Untreated and Stabilized Peat from EDX Test

Element	Raw Peat	C100	C85P15	C80P20
Carbon, C (%)	75.22	39.37	20.74	27.65
Oxygen, O (%)	23.56	37.12	46.82	42.75
Silica, Si (%)	0.33	4.84	3.09	4.21
Aluminium, Al (%)	0.22	0.81	0.58	0.71
Iron, Fe (%)	0.30	0.54	0.67	1.11
Calcium, Ca (%)	0.14	16.36	27.72	22.01
UCS (14 Days) (kPa)	9.744	55.730	71.758	69.347

CONCLUSION

Utilization of Palm Oil Fuel Ash (POFA) as sustainable binder to partially replace cement in peat soil stabilization has been examined. POFA is proved to have significant effect in achieving higher unconfined compressive strength in peats. As a summary of the findings attained, the following conclusion could be drawn:

i. Average moisture content of Medan Sari peat was 938.12 %. Organic content was 96.487% while fiber content was 72.319 %. All these values drew inference that peat soil at Medan Sari was categorized as fibric peat. Fibric peat is slightly decomposed compared to sapric and hemic peats.
ii. Liquid limit obtained in cone penetration test was 419 % which is much higher than results presented by other researchers. High fiber content would lead to high water holding capacity and thus causing high liquid limit.
iii. According to Von Post scale, Medan Sari peat was classified as H3 which is represented the types of fibric peat.
iv. Specific gravity was 1.208 Mg/m^3. Higher percentage organic constituents will affect the peat soil to have lower specific gravity.
v. POFA has finer particle size after subjecting to Los Angeles abrasion machine for ball milling process. Results from CILAS showed that both cement and POFA had fulfilled the requirements in ASTM C150 and ASTM C618 respectively. Around 86 % of untreated peat had passed through No.200 sieve.
vi. PH value of fibric peat used in this study was 4.12 which meant that raw peat was a kind of acidic soil.
vii. C85P15 achieved the highest unconfined compressive strength which was 107.59 kPa at 28 curing days. Untreated peat had very low compressive strength which was only 8.65 kPa at strain of 15 %. C80P20 started to drop in strength but it still had greater strength than C100. Therefore, POFA was proved to have significant effect in reaching higher compressive strength in peat.

Through SEM-EDX, the results showed that C85P15 had higher percentage of calcium than C100 and C80P20. Besides, it had lower silica and alumina content which meant the pozzolanic reaction had took place in the peat. In C80P20, the silica and alumina content were high probably due to excessive POFA in peat soil without pozzolanic reaction.

ACKNOWLEDGEMENT

The authors would like to thank the Universiti Tun Hussein Onn Malaysia (UTHM) and Ministry of Education Malaysia for their sponsorship (FRGS/1/2019/TK08/UTHM/03/1) and research funding grant vote (H009) GPPS from Research Management Center (RMC) on this study. The authors also would like to extend their gratitude to Research Centre for Soft Soil (RECESS) for allowing the use of research equipment and facilities. Finally, authors would also wish to express gratitude to FGV Holdings Berhad for providing POFA.

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PLANNING IMPACT AND MEDIATIONS ON THE QUALITY OF CONSTRUCTION PROJECTS: CASE STUDY IN JORDAN

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Abstract
Many factors affect the quality of construction projects. One of the significant factors influencing the quality of the project is planning. This paper develops a conceptual model to identify the impact of planning on construction companies' quality by referring to three mediator factors: the concept of planning, the importance of planning, and the basics of efficient planning. Additionally, the model aims to quantitively measure the impact of planning on the quality of projects. A semi-structured questionnaire was distributed randomly to 159 Jordanian Engineers. The results indicate that there is a medium level of planning in Jordanian construction companies. The respondents and statistical analysis showed that there is a significant impact of planning on the quality of construction projects. Finally, some recommendations were made to improve construction quality and planning.

Keywords: *Construction project; impact; Jordan; planning; quality.*

INTRODUCTION

In the construction industry, one of the main objectives is to obtain a high-quality product, which could be achieved through efficient planning, design, and construction. The American Society of Civil Engineers defined quality as "the totality of features, attributes, and characteristics of a facility. The product, process, component, service, or workmanship that bears on its ability satisfy a given need: fitness for purpose. It is usually referenced to and measured by the degree of conformance to predetermined standards of performance" (Ebel, 2019).

One of the main factors that influence quality is the degree of planning conducted by construction companies through the realizing of the concept of planning, recognizing the importance of planning, and adopting the basics of efficient planning (Rumane, 2017). Though that, focus on planning and quality is a significantly researched topic, this research will adopt the knowledge-based view to quantify the impact of planning on the quality of construction projects mediated by those three factors. Additionally, the researcher aims to identify the factors that moderate the relationship between planning and quality.

LITERATURE REVIEW

Planning

Planning is considered as the major principle of the principles of management – planning, organizing, leading, and controlling (Morden, 2017). It also represents the starting point in managerial work. According to Noe, Hollenbeck, Gerhart, and Wright (2017), planning is an essential function because it contributes the most to the other functions. Consequently, the success or failure of any managerial process depends on planning; no organization could succeed without proper planning, and no organization or manager can prepare for the future

without planning. Planning forces the organization and management to think about the actions that should be taken now to be prepared for the future (Oliver & Parrett, 2018).

Ramlawati and Kusuma (2018) indicate that proper planning is where the decisions that have to been taken today would lead to valuable results in the future. It also indicates that the planning process is a dynamic process that consists of lots of variables and elements that should be predicted by analyzing the present situation.

Omusi (2018) conducted research to identify factors that influence the planning process. They indicated that strategic management, leadership, organizational culture, and human resources contribute to the success of any strategic planning. It was found that human resources in an organization contribute to the most significant percentage. Consequently, and based on a comprehensive literature review, three main factors were identified for effective construction planning; identifying the concept of planning (Appiah, 2017; Inuwa, Joseph, & Musa, 2019), realizing the importance of planning (Hiyassat, Hiyari & Sweis, 2016), and adopting the basics of effective planning (Kiptoo & Mwirigi, 2014; Lines, Sullivan, Hurtado, & Savicky, 2015). Those factors will be used as mediators and are discussed as follow:

The Concept of Planning

Identifying and applying the concept of planning is classified as one of the main factors that affect the planning process (Kiptoo & Mwirigi, 2014). There are many definitions for the term "planning", each researcher has their definition; the most common definitions are:

- Planning is the process by which an individual or organization decides in advance on some future course of action (Kerzner, 2017)
- Planning is the process of determining how the organization can get where it wants to go. (Certo, 2000)
- Planning involves selecting from among alternative future courses of actions for the organization as a whole and every department or section within it. (Cook & Hunsaker, 2001)

Construction companies should apply the concept of planning while conducting projects. In the beginning, construction companies should hire experienced people to plan and conduct the plan. Also, Companies should ensure that there is full coordination between them. Thus, it leads to an accurate definition of the characteristics of the project. Therefore, Managers should define the objectives of the firm and the project; then those objectives should be explained to both planners and executive staff (Fewings & Henjewele, 2019).

Importance of Planning

Realizing the importance of planning is as essential as the planning itself (Kiptoo & Mwirigi, 2014). The planning process has four main goals: 1) to minimize uncertainty and change, as for reducing the risk and increase success, 2) to define objectives and focus on them, 3) to be economically beneficial; leads to minimizing the costs by directing effort towards production activities, and 4) to facilitate control; by measuring the accomplishments with the plans (Kerzner, 2017).

The importance of planning should be realized by construction companies. It will be evident if construction companies lack the realization of the importance of planning. Thus, it would lead to random and chaotic while executing projects and misuse of resources. Huo, Ann, Darko and Wu (2019) indicated that for construction companies planning is essential to predict the time and cost correctly. They recommended that future conditions would be assumed by experts, and evolutions should be conducted based on known standards.

The major factor that strengthens the importance of planning in companies is the availability of a network diagram for projects. This diagram helps the execution team differentiate between critical and non-critical activities (Huo et al., 2019).

Basics of Effective Planning

To achieve an efficient planning level, some basics should be adopted by companies. Kerzner (2017) recommended that for efficient planning, planning should be effectively done and closely allied to the main point. Also, planning should lead to decision making. Kerzner (2017) also, recommends basics that lead to efficient planning, among them: the forecast should be accurate, the plan should be accepted, the plan is sound and objective, the flexibility of the plan, and plans should be revised to be sure that it fits the situations.

Quality

Many researchers throughout the years attempted to create a definition for quality in general or by formulating quality through quality assurance systems. Thus, they depend on judgment, experience, assumptions, and opinions. Below are somedefinitions and assumptions of quality:

“The totality of features and characteristics of a product or service that bear on its ability to satisfy stated and implied needs” as defined by the British Standards Institution. (McCabe, 2014). Some researchers like Juran and Godfrey (1999) assume that quality is determined by customers' satisfaction, what consumers want, and what consumers are willing to buy. So, they define quality as the "Fitness for purpose/use". The ISO 8402 also adopted this theory; their definition is "Quality relates to the evaluation of a product or service to its ability to satisfy a given need" (Karim & Islam, 2019).

Crosby and Free (1979) definition: “conformance to agreed and fully understood requirements.”. Thisdefinition was based on a manufacturing basis. In a standerd view for quality, it was amplified that there is no such thing as high or low quality. (McCabe, 2014).

All definitions are essential, and it is necessary to connect them to set an overall definition of quality. This could be done by understanding the factors affecting quality (Castello, De Castro, & Marimon, 2019).

Factors Affecting Quality

Many researchers have attempted to determine the critical factors affecting quality. The literature has various lists of variables that measure the quality of a construction project. Some

variables are repeated in more than one list and became common, but there is no agreed list in general (Mutha & Ghadge, 2018).

Hussain, FangWei and Ali (2019) have found that the generally perceived factors that affect the quality of the project can be classified under the headings of the client, project environment, project team leaders, project procedures, and project management procedures.

A research was conducted in India by Shobana and Ambika (2016) to evaluate the factors affecting quality in construction projects. The factors were identified from the literature. A questionnaire survey, including 138 respondents, was conducted. Based on statistical analysis, the top ten ranking factors were: coordination, resources, inventory level, labor shortage, material delay, heavy work, inspection, environment, labor experience, and meetings. The researchers were able to evaluate the factors affecting quality in construction and determine the top-ranked factors.

Factors influencing the quality of construction projects were identified and then ranked differently from the perceptive of the contractor and consultant. The contractor ranked them as follows: conformance to codes and standards, selection of a contractor, financial issues, top management support, material, labor, equipment, management factors, execution, selection of designer, lack of communication, system, the co-operation of parties, design and contract documents (Joy, 2014).

Relationship Between Planning and Quality

Time, cost, and quality were recognized as the "triple constraint" or essential elements of the success of a project (Fewings & Henjewele, 2019). Planning was identified as a significant factor that influences the performance of construction projects (Larsen, Lindhard, Brunoe, & Jensen, 2018). Additionally, The top critical success factors within the project life cycle were identified to be: project mission, top management support, the competence of project manager, project plan, the competence of project team, quality of suppliers and subcontractors, technical tasks, client acceptance, monitoring, communication, and troubleshooting (Babu & Sudhakar, 2015).

Moreover, Project plans affect the outcome of construction projects. Principally, the client's assessments of the project’s duration, cost, and quality are used as the subjective indicators of the project's outcome. Therefore, Plans can be used to reduce the high time and cost overruns. It is recommended that stakeholders be sure that the project plans are prepared (Gunduz & Yahya, 2018).

Design changes and the lack of skills were ranked as a significant factor that causes delays in building construction projects by large and small contractors (Zidane & Andersen, 2018). On the other hand, slowness in making decisions was ranked the highest by large contractors. In Ramli et al. (2018) research, “Poor planning and scheduling” was found as one major delay cause.

Poor planning was also identified as one of the essential reasons for cost overrun. Fifteen most essential causes were identified from a list of forty-nine different reasons. They were: delays of schedule, changes in design, owner interference, design mistakes and errors, poor

planning and scheduling, inflation, scope changes, drawings, workers skill, construction mistakes, and defective works (Bekr, 2015).

A literature review for research in the Jordanian construction sector indicated a significant relationship between planning and quality. For instance, Hiyassat et al. (2016) noted that Poor safety, inferior working conditions, and insufficient quality had been identified as factors affecting construction's productivity and the company's performance. Furthermore, "poor planning and scheduling" was classified as one of the critical causation variables.

R. J. Sweis, Bisharat, Bisharat, and Sweis (2014) also conducted research in Jordan, to identify factors affecting the performance in construction projects, then classify them according to the consultant's and contractor's perspective. The statistical analysis yielded that financial difficulties, workforce shortage, and excessive owner change orders were the main factors affecting the contractor's performance. Also, Poor "planning and scheduling" were classified by both the consultant and owner as one of the most critical factors.

Similarly, Odeh and Battaineh (2002) conducted research in Jordan, to identify the causes of construction delay from the viewpoint of contractors and consultants. Their study indicated that both consultant and contractor agreed that interference of the owner, payments and financing, delay in decision making, subcontractors, the experience of the contractor, productivity of labor and poor planning are among the top ten most important factors.

Theory Underpinning This Research

The knowledge-based theory (KBT) of the firm considers knowledge as the most strategically significant resource of the firm. Its proponents argue that knowledge bases and capabilities among firms are the major determinants of sustained competitive advantage and superior corporate performance (Flöthmann, Hoberg & Gammelgaard, 2018).

A review of theories related to this research problem suggests that the KBT is a valuable method to assess the change in companies' quality due to planning. The level of planning can play an essential role in the knowledge-based view of the firm in that planning can be used to synthesize, enhance, and expedite knowledge management (Yu & Yang, 2018).

Research Gap and Added Value

To provide cheaper and high-quality products for their customers, it is crucial for construction companies to "rethink their business models and to innovate with the ways they deliver and receive services" (Thuesen & Hvam, 2012). As the poor planning promising for declining project quality, it was significantly looked in the literature. On the other hand, a review of the existing construction literature indicated that such an impact had not been quantified. Therefore, this study attempts to cover this gap by quantifying the impact of planning on construction projects' quality adopting KBT; such knowledge would enhance the understanding of construction management and policy-makers on when, where, and how to conduct efficient planning.

Hence this study is aimed at investigating the impact of planning on construction projects' quality in developing countries, namely Jordan. The findings are expected to, besides adding

to literature, open the door for further research on the use of planning for business purposes and its influence in the context of organizational quality, more specifically to the construction industry.

Research Questions and Research Model

This research depends on a descriptive-analytical methodology by using a survey. This paper proposes a conceptual model based on the KBT, relating planning with the quality of construction projects (Figure 1). Planning, the independent variable, represents the degree of planning adopted by construction companies. The quality of construction projects, the dependent variable, is defined as the degree of conformance to predetermined standards and specifications of performance. The concept, importance, and basics of planning are the mediators.

The main question of this research can be formulated as: "what is the impact of planning on the quality of construction projects in Jordan?" Arising from the main question are the following secondary questions:

1. what is the degree of the planning adopted in Jordanian construction companies?
2. To what level does the sample estimate the impact of planning on the quality of the project?
3. Is there any statistically significant impact of planning (concept, importance, and basics) on the quality of construction projects?
4. Which of the control factors “gender, age, qualification, experience, firm size, and course attendance” moderate the relationship between planning and quality?

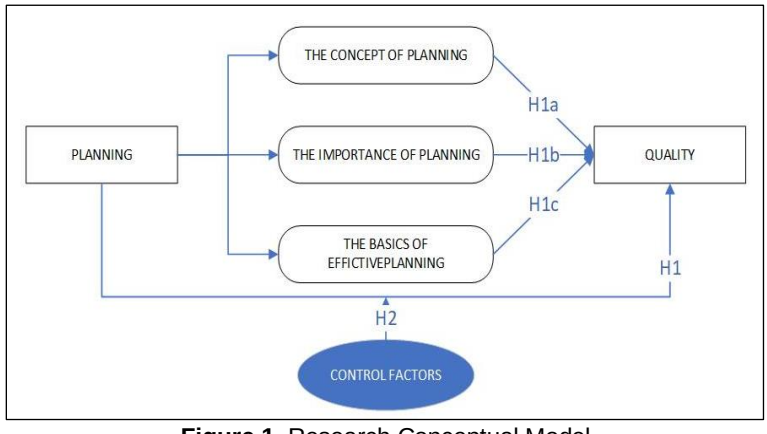


Figure 1. Research Conceptual Model

RESEARCH METHODOLOGY

Study Tool

The study tool (questionnaire) consisted of three sections. The first section of which dealt with the demographic data for the sample: gender, age, academic qualification, representation (work for), experience, firm size, and planning course attendance. The second section consisted of three parts which are the factors that measure the degree of planning in a

company: The realization of the concept of planning (13 items), Realizing the importance of planning (11 items), and Adoption of the basics of effective planning (11 items). The third section consisted of (18) items that measure the quality of a construction project (Table 3).). In sections two and three, a 5-points Likert scale was used, as shown in Figure 2, respondents were directed to show the extent to which they agree with the provided statements.

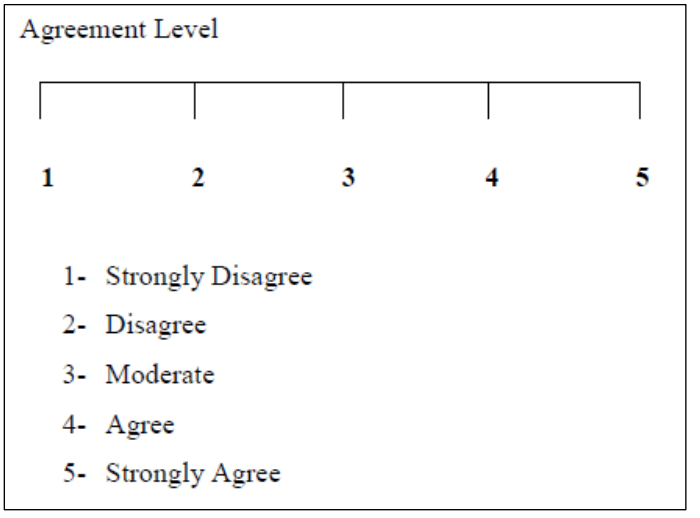


Figure 2. Likert's Scale Used in The Study Tool

Sample and Data Collection

The population of the questionnaire survey was limited to Jordanian engineers who work in the construction industry in Jordan. Due to the difficulty of defining the numbers of those engineers who are geographically distributed in Jordan, the challenge of reaching them, and saving time, money, and effort, a convenience sample was selected. A convenience sample refers to the collection of information from members of the population who are conveniently available to provide it (Sekaran & Bougie, 2016).

Questionnaires were distributed by social media, to return the higher number of responses, for four weeks. One hundred and seventy (170) surveys were retrieved. After checking and auditing, eleven (11) responses were suspended due to their lack of validity for statistical analysis. Thus, the results were extracted for 159 responses. Table 1 summarizes the characteristics of the sample.

Reliability and Validity of The Questionnaire

Cronbach's alpha was used to test the reliability of the scale. According to Sekaran and Bougie (2016), Cronbach's alpha is a standard test for data reliability. The analysis indicated a Cronbach's alpha value of (0.94). Since reliability coefficients that are above 0.6 are acceptable, the questionnaire is internally consistent (Hair, Black, Babin, Anderson, & Tatham, 2006). Additionally, the researcher examined the validity of measures by presenting the questionnaire to professional experts and academics who refereed the tool and gave their notes and comments

Table 1. The Characteristics of The Sample

Characteristics	Categories	Frequency	Percent
Gender	Male	106	66.7
	Female	53	33.3
Age	20-30	122	76.7
	31-40	20	12.6
	41-50	5	3.1
	Over 50 Years	12	7.5
Qualification	BCS	126	79.2
	Higher Degree	33	20.8
Work for (represent)	Consultant office	43	27.0
	Contractor	74	46.5
	Owner	42	26.4
Experience	Less than 2 years	45	28.3
	2-5 years	60	37.7
	5-10 years	33	20.8
	10-20 years	5	3.1
	Over 20 years	16	10.1
Firm/Company Size	Less than 10 employees	24	15.1
	10-49 employees	42	26.4
	50-200 employees	24	15.1
	Over 200 employees	69	43.4
Attend planning courses	Yes	113	71.1
	No	46	28.9

DATA ANALYSIS, RESULTS AND DISCUSSION

Research Question 1: What is the Degree of the Planning Adopted in Jordanian Construction Companies?

To answer this question, the standard deviations (SD) and the means were computed (Table 2 and Figure 3). Items were measured using a five-point Likert scale. To classify the level of usage, five ranges for estimates were used (1.00-1.79 = Very Good, 1.80-2.59 = Good, 2.60-3.39 = Medium, 3.40-4.19 = Poor, 4.20-5.00 = Very Poor). The Descriptive statistics for "the degree of the planning adopted in Jordanian construction companies?" indicated that the overall mean of the results in section "degree of planning" was (2.72) with a standard deviation of (.755) which is considered medium compared to standards of this study.

Table 2. The Means and Standard Deviations (Degree of Planning)

No.	Articles	Mean	Standard Deviation	Rank*	Level*
C	Basics of planning	2.90	0.780	1	Med
B	Importance of planning	2.70	0.717	2	Med
A	Concept of planning	2.56	0.770	3	Good
-	Overall average of Planning	2.72	0.755		Med

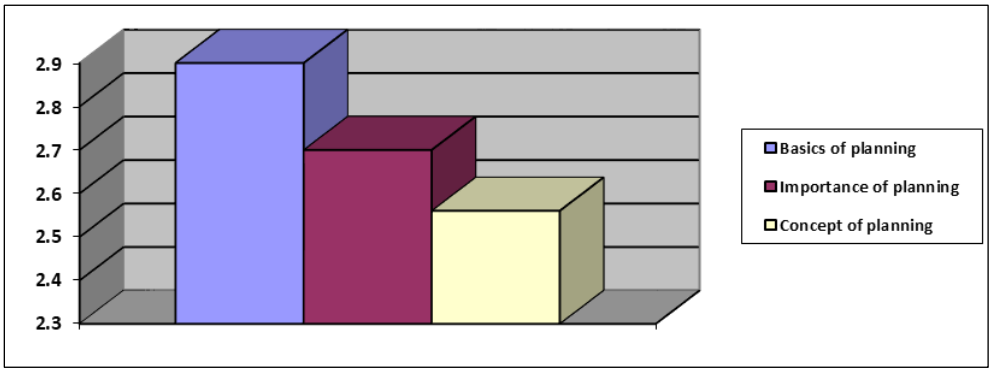


Figure 3. Degree of Planning in the Construction Firms

Research Question 2: To What Level Does the Sample Estimate the Impact of Planning on the Quality of the Project?

Table 3 and Figure 4 point out the overall mean for the sample, which was (3.57) with SD of (0.965), it is considered high compared to the standards of this study tool.

Table 3. The Means and Standard Deviations (Impact on Quality)

No.	Articles	Mean	Standard deviation	Rank*	Level*
D14	Financial flow is organized to keep the project's progress	3.81	1.214	1	High
D1	Finishing the project on time	3.68	1.370	2	High
D9	Drawings are obvious and include all details	3.68	1.333	2	High
D2	Finishing the project within the budget	3.65	1.217	4	High
D8	Design drawings are comprehensive and complete	3.64	1.338	5	High
D6	Equip the consultant with the information needed about the project	3.60	1.164	6	High
D10	Materials are supplied according to schedule	3.60	1.258	6	High
D7	Equip the contractor with the information needed about the project	3.58	1.342	8	High
D15	Final submission is made according to precise procedures	3.58	1.127	8	High
D4	Determine the required consultant experience	3.56	1.225	10	High
D5	Determine the required contractor experience	3.55	1.291	11	High
D11	Coordination between contractor and suppliers	3.55	1.215	11	High
D13	Equipment is used in an organized manner	3.55	1.053	11	High
D12	Applying quality standards upon conducting tests	3.48	1.247	14	High
D17	Determining standards for tender assignment	3.48	1.135	14	High
D16	Assisting the owners to take decisions in a convenient time	3.46	1.146	16	High
D18	Determining a management system to control labor	3.45	1.156	17	High
D3	Determine the project's location (wide, accessibility and storage area)	3.43	1.280	18	High
-	Overall average of Quality of the project	3.57	0.965		High

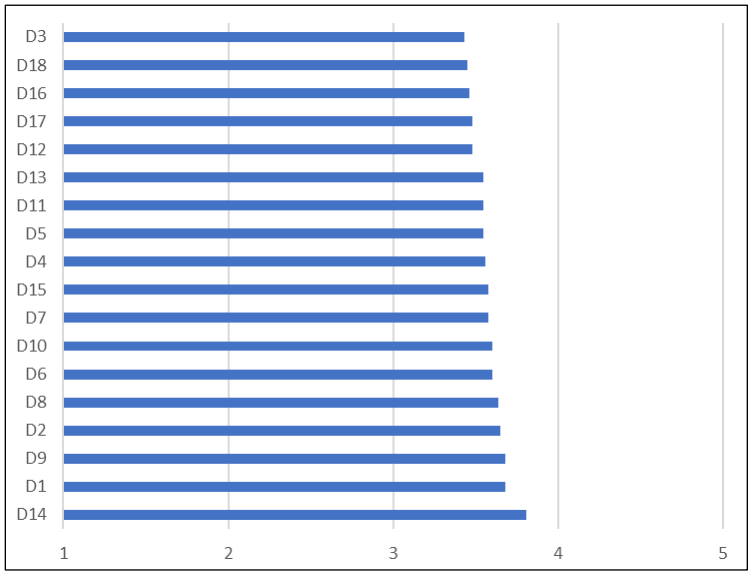


Figure 4. Estimated Impact of Planning on the Quality

Research Question 3: Is There Any Statistically Significant Impact of Planning (Concept, Importance, and Basics) on the Quality of Construction Projects?

A simple regression test was used to answer research question number 3. The results can be seen in Table 4. The analysis of normality was conducted where the results of the Kolmogorov-Smirnov statistic were obtained. A non-significant result (Significant value of more than 0.05) indicates normality. In the case of this study, the value was 0.092, suggesting an approximately normal distribution of scores. To answer this question, hypotheses were formed and tested.

H1: *Planning has a Significant Impact on Quality.*

Table 4. The Results of the Regression Test for Planning

Concept of Planning					
F	Sig	R	R^2	α	β
75.816	.000	0.610	0.372	3.252	0.135
Importance of Planning					
F	Sig	R	R^2	α	β
72.981	.000	0.603	0.363	3.181	0.154
Basics of Planning					
F	Sig	R	R^2	α	β
59.739	.000	0.564	0.318	3.100	0.182
Planning					
F	Sig	R	R^2	α	β
59.495	.000	0.563	0.317	3.058	0.184

The analysis revealed a statistically significant impact of planning on the quality, as supported by the F value (59.495) with a (0.000) significant value. Thus, the null hypothesis was rejected. Additionally, R^2 (0.318) indicates that planning explained a percentage of (31.8%) of the changes that occur to the quality.

H1a: *the concept of planning has a significant impact on quality.*

Table 4 indicated a statistically significant impact for applying the concept of planning on quality, as supported by the value of F (75.816) with a significant value of (0.000). Thus, the null hypothesis was rejected. Additionally, the value of R^2 (0.372) was increased (from 0.317), which indicated that the impact of planning on quality is more significant when mediated by the concept.

H1b: *the importance of planning has a significant impact on quality.*

There is a statistically significant impact of realizing the importance of planning on the quality. Supported by the F value (72.981) with a significant value of (0.000). Thus, the null hypothesis was rejected. Additionally, the value of R^2 (.363) was increased (from 0.317), which indicated that the impact of planning on quality is more significant when mediated by the importance.

H1c: *the basics of planning have a significant impact on quality.*

There is a statistically significant impact of adopting the basics of good planning on the quality. Supported by the F value (59.739) with a significant value of (0.000). Thus, the null hypothesis was rejected. Additionally, the value of R^2 (0.318) was increased (from 0.317) which indicated that the impact of planning on quality is more significant when mediated by the basics

Research Question 4: Which of the control factors moderate the relationship between planning and quality?

To answer research question number 4, the means and the SD were obtained according to the demographic characteristics.

To point out the significance of the statistical differences, hypotheses were formed. A *t*-test was applied to the (gender, qualification, Training courses) while, a one-way ANOVA test was applied to the (age, work for, Experience, Firm size).

H2a*: **Gender** Moderates the Relationship Between Planning and Quality*

Table 5 shows that there are no statistically significant differences at a significance level of (0.05) where the t value was (-0.612) with a significance of (0.541). What supports that result is that the means for both males and females were convergent. Thus, the null hypothesis was accepted.

H2b*: **Age** Moderates the Relationship Between Planning and Quality*

Table 5 shows that there are no statistically significant differences at a significance level of (0.05) where the F value was (2.179) with a significance of (0.093). What supports that result is that the means for all age ranges were convergent. Thus, the null hypothesis was accepted.

***H2c*: *Qualification** Moderate the Relationship Between Planning and Quality*

Table 5 shows that there are no statistically significant differences at a significance level of (0.05) where the t value was (-1.594) with a significance of (0.113). What supports that result is that the means for both BCS and higher degrees were convergent. Thus, the null hypothesis was accepted.

***H2d*: *Representation** Moderate the Relationship Between Planning and Quality*

Table 5 shows that there are statistically significant differences at a significance level of (0.05) where the F value was (4.870) with a significance of (0.009). The differences were between the "contractor" and the "consultant" samples, which mean that the consultant indicated that there is a higher impact of planning on the quality of the project. Also, between "consultant" and "owner" samples, means that the consultant indicated a higher impact of planning on the quality. And the null hypothesis was rejected.

***H2e*: *Experience** Moderates the Relationship Between Planning and Quality*

Table 5 shows that there are statistically significant differences at a significance level of (0.05) where the F value was (4.008) with a significance of (0.004). Differences were between "less than 2 years" and "2-5 years" samples sided to “less than 2 years”, between "less than 2 years" and "10-20 years" samples sided to “10-20 years”, and between "less than 2 years" the "more than 20 years" samples sided to “more than 20 years”. And the null hypothesis was rejected.

***H2f*: *Firm Size** Moderates the Relationship Between Planning and Quality*

Table 5 shows that there are no statistically significant differences at a significance level of (0.05) where the F value was (0.952) with a significance of (0.417). What supports that result is that the means for all firm sizes were convergent. Thus, the null hypothesis was accepted.

***H2g*: *Training Courses** Moderate the Relationship Between Planning and Quality*

Table 5 shows that there are no statistically significant differences at a significance level of (0.05) where the t value was (0.194) with a significance of (0.846). What supports that result is that the means for both “attended” and “did not attend” were convergent. Thus, the null hypothesis was accepted.

Table 5. Results of T and ANOVA Test

Age					
	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	5.960	3	1.987	2.179	.093
Within Groups	141.304	155	.912		
Total	147.263	158			

Work For					
	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	8.654	2	4.327	4.870	.009
Within Groups	138.609	156	.889		
Total	147.263	158			
Experience					
	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	13.886	4	3.471	4.008	.004
Within Groups	133.377	154	.866		
Total	147.263	158			
Firm Size					
	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	2.665	3	.888	.952	.417
Within Groups	144.598	155	.933		
Total	147.263	158			

Gender						
Sig	T	Female		Male		
		St. deviation	Mean	St. deviation	Mean	
0.541	-0.612	0.879	3.64	1.008	3.54	Quality
Qualification						
Sig	T	Higher Degree		BCS		
		St. deviation	Mean	St. deviation	Mean	
0.113	-1.594	0.908	3.81	0.974	3.51	Quality
Training Courses						
Sig	t	Did not attend		Attended		
		St. deviation	Mean	St. deviation	Mean	
0.846	0.194	1.002	3.55	0.945	3.58	Quality

CONCLUSIONS

The results indicated that planning has an impact on the quality of construction projects. This impact was indicated from the respondents' estimates and then was statistically tested using the regression test. Those results support the previous efforts in the literature; A study conducted in India by Shobana and Ambika (2016) to evaluate the factors affecting quality in construction projects, where coordination and material delays were classified as top-ranked factors. These elements can be controlled by good planning. Another study was conducted in India as well. It used time, cost, and quality to measure the success of projects. It indicated that the project's mission is a critical success factor (CSF), which is a major phase in planning (Gunduz & Yahya, 2018). Similarly, in a study aimed to identify the factors affecting the quality of construction projects, both contractors and consultants agreed that planning affects the quality of the project (Hussain, Fangwei, Siddiqi, Ali & Shabbir, 2018).

In Jordan, a study was conducted by Sweis et al. (2014) to determine the factors affecting performance. Both contractors and consultants agreed that poor planning and scheduling is one of the major CSFs. Also, poor planning was among factors leading to construction projects failure in Khlaifat, Alyagoub, Sweis, and Sweis (2019) study.

Hasan, Baroudi, Elmualim, and Rameezdeen (2018), Gunduz and Yahya (2018), and Moh'd Al-Abbadi and Agyekum-Mensah (2019) studies concluded that planning is classified

as one of the major factors that affect the quality and productivity of construction companies. Though, none of the past studies quantitively measured the impact of planning on the quality of the project. However, most of them pointed out that planning affects the quality of projects.

In simple linear regression, the outcome or dependent variable Y is predicted by only one independent or predictive variable. Their relationship can be expressed in a math equation as follows: $Y = \alpha + \beta X + e$. Where: **Y** is the dependent variable; **α** is a constant amount; **β** is the coefficient; **X** is the independent variable, and **e** is the error or the 'noise' term that reflect other variables to have an effect on Y. Thus, based on Table 4, the following equations were formulated.

Quality = 3.058+ 0.184 Planning (1)

Quality = 3.525 + 0.135 Planning Concept (2)

Quality = 3.181 + 0.154 Planning Importance (3)

Quality = 3.100 + 0.182 Planning Basics (4)

Most of the control variables were found to have no impact on the relationship between planning and quality: gender, age, qualification, firm size, and training courses. While experience indicated a significant impact, that's because those who have more experience tend to have more knowledge and skills. Additionally, representation indicated a significant impact; that is because consultant and owner engineers tend to focus on quality more than contractor engineers. Those results support G. J. Sweis, Sweis, Al-Shboul, and Al-Dweik's (2015) conclusions regarding the moderate factors between IT and quality of construction projects.

RECOMMENDATIONS

The findings imply academic and managerial perspectives. This research filled the gap by quantifying the impact of planning on the quality of Jordanian construction projects. The results confirm that planning explained a percentage of (31.8%) of changes that occur to the quality, and the remaining percentage of (68.2%) is caused by other variables. This percentage is relatively high. Moreover, the impact of planning on quality will be increased when construction companies realize the concept of planning, recognize the importance of planning, and adopt the basics of effective planning.

It is recommended to have full coordination between planners and the executive team, so plans are explained to workers to enhance the realization of the concept of planning. Similarly, construction companies are recommended to conduct an evaluation based on known standards with coordination between the execution team, top management, and planners to embrace the importance of planning. Additionally, rewarding the staff, keeping the outcome of planning for preceding projects, and ensure all staff contributions are basics of effective planning.

LIMITATIONS AND FUTURE RESEARCH

Using a convenience sample in this type of research is a limitation. Therefore, increasing sample size and choosing a more specific sample type are recommended to ensure more findings' generalization. Further studies should be directed towards finding the challenges of effective planning in the construction companies. Besides, the aspects of this research could be improved by applying the methodology used in this research to other developing countries. Moreover, the use of more advanced statistical analyses (i.e., multiple regression and Structural Equation Modeling (SEM)) is recommended for future research.

ACKNOWLEDGMENT

The corresponding author would like to express very profound gratitude to Prof Tahseen Mansour; for providing him with unfailing support and continuous encouragement throughout his years of study, and also through the process of research and writing this article.

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THE EFFECT OF W/C RATIO OF COMPACTED BORED PILE TO FRICTION RESISTANCE OF UNJANI CLAY

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Abstract
The installation of bored pile in clay soil disturbs the properties of surrounding soil in the vicinity of pile until certain distance from outer skirt of bored pile. Once disturbance halts, a clay regain it's strength under thixotrophy to a certain degree normally less than 100 %. On bored pile installation, under the effect of various w/c ratio, the capacity of bored pile shows different value. To obtain the effect of w/c ratio to pile capacity and original soil, the small scale model drum was set-up. Clay soil, bored pile model and procedures of simulation were also prepared. The simulation of pile loading test (PLT) tests were carried out in various w/c ratio in ambient condition on first liquidity index (LI_1). To obtain LI_2 , the stiff steel plate was then positioned to cover clay soil, then the surcharge was laid on top to allow consolidation. After enough time to consolidate to obtain LI_2, the similar simulation of PLT were performed. Similar sequence were done to obtain third LI_3 with different loading and consolidation time. The direct shear test (DST) of clay soil (at LI1 , LI2 and LI3) and at combination of clay + cement mortar at various w/c ratio were also performed to support and synchronize the simulation work. The results reveal that there are changes of clay properties due to migration of water into void or crack inside surrounding clay, mixing cement mortar with clay mineral as well as compaction from hammer drop. The higher water content of original clay the bigger adhesion, α value as clay soil is softer and the optimum w/c-ratio (w/c = 0.4) provides more pile capacity. It is understandable that water plays an important role, water from cement mortar migrates to surrounding clay at various degree of moisture content, which contribute to the change of surrounding clay properties. The slight improvement of Su ($\pm 75\%$ increase) due to compaction of concrete in bored pile, migration of moisture and reaction with surrounding clay leads to take benefit of design and construction of bored pile in clay soil for the sake of efficiency and stability.

Keyword: *Clay; **c**ompaction pile; undrained shear strength; water-cement ratio; bored pile; ground improvement.*

INTRODUCTION

To get a great benefit in construction of deep pile foundation, any type of pile should contribute to increasing properties as improvement of soil surrounding pile on short and long term condition. In the current practice of deep foundation technology, there are five categories of ground improvement pile (Clayton, 1983):

Category A: Ground Improvement without admixtures in non-cohesive soils or fill materials (dynamic compaction, vibrocompaction etc.)

Category B: Ground Improvement without admixtures in cohesive soils (Replacement, preloading, vertical drains, vacuum consolidation etc.)

Category C: Ground Improvement with admixtures or inclusions (Vibro replacement - stone columns, sand compaction piles, rigid inclusions etc.

Category D: Ground Improvement with grouting type admixtures (Particulate and chemical grouting, Deep mixing, jet grouting etc.)

Category E: Earth reinforcement (Geosynthetics or MSE, ground anchors, soil nails etc.).

In an ordinary theory mentioning that Bored pile type is non-displacement pile instead of driven pile as displacement type. To change the bored pile become displacement type, need an effort to displace surrounding soil in the vicinity of bored pile, the compaction of concrete during installation is one of the solution. During compaction, the soil displaced and excess pore water pressure increase, then by allowing pore water pressure to dissipate, the ground improvement is happening.

The ultimate capacity Q_u of bored pile can be described in equation (1),

$$Q_u = Q_p + Q_f \tag{1}$$

where, Q_p and Q_f are Point capacity and friction resistance respectively. The term Q_p consists of Ap 9 Cu and p L α Cu for friction capacity as mentioned in equation (2).

$$Q_u = Ap\ 9\ Cu + p\ L\ \alpha\ Cu \tag{2}$$

The term α Cu is friction resistance qf , Cu is undrained shear strength and α is adhesion factor. According to many references, the value of adhesion factor, α normally in the range of 0.3 to 0.8 to original sub-soil of undrained shear strength (Hoque & Alamgir, 2014).

By the improvement, the impact of those improved sub-soil can be as high as two fold to original shear strength even some reported higher. As mentioned in eq. (3), the improvement of Su after grouting pile can be in the range of 2 fold of Su before grouting as described by Liao, Su and Chen (2006).

$$\left\{\frac{S_{u-r}}{S_u}\right\} max = 1 + \frac{2S_u}{\sigma'_p} Ln(0.5\sqrt{I_r}) \tag{3}$$

where:

Su = undrained shear strength
Ir = Rigidity index
$\sigma' p$= is the vertical effective consolidation pressure before ground improvement

Installation of displacement types of pile by driving or by compaction on concrete, causes stress changes in the surrounding soil. The properties will decrease in surrounding soil, recover, and increase after pile column installation. The philosophy of stabilization methods either in clay or sandy soils consist of displaced soil in radial or horizontal direction. The construction of compacted bored pile leads to reach horizontal displacement as well as raising excess pore water pressure (pwp). Eventually, the undrained shear strength Su, will increase if enough time is permitted for excess pwp to go away gradually from surrounding soil (Liao et al., 2006; Hoque and Alamgir, 2014).

Majority the causes of improvement in this category closely related with moisture migration, due to the facts that concrete in bored pile construction is made of in-situ or casting in place, there are some portion of water will move to surrounding soil. The percentages of moving water is not known accurately, but some researcher have introduced their findings as follow : First prominent researcher conclude that water moves in certain clay reachs to 2,5 to 3 inches. Secondly, Meyerhof revealed that water migration caused 6 to 7 % increases than original clay. Then Skempton finally described that deduction of Su due to migration of water from fresh concrete (Das, 2007; Liao et al., 2006). Ratio of weight of water to weight of cement in a concrete mix is labelled as w/c ratio is the key element to trigger migration of water. If the ratio is small leads to high strength and durable of concrete and contrary, if w/c ratio is large leads to many weakness such as : segregation, highly porous, crack, shrinkage and finally goes to lowering strength of concrete (Chang & Zhu, 2004; Zhu, 2016). The hydration process in producing concrete is a chemical reaction between cement and water normally needs 35 % from its weight to finish hydration process. Nevertheless, this ratio is not sufficient to complete the process of producing concrete, it still needs additional water. The typical w/c ratio of 0.45 to 0.6 is normally used with some necessary additive materials.

By combination of increasing water content which leads to lowering Su and chemical reaction of water + cement which leads to increasing Su, the possibility of final condition is important to be investigated. There are at least seven major factors causing the property changes in surrounding soils during and after installation of piles. These factors are soil thixotropy (Chuang & Reese, 1969), soil fracturing, cement penetration and diffusion, cementation, consolidation, heating. and pozzolanic reaction. The aim of this study is to investigate the optimum water cement ratio in the installation of bored pile in clay soil in various Liquidity Index, LI (different water content) after compaction by hammer blow which is simulated in small scale drum lab. model as shown in Figure 1. From the concept of small scale modelling in normal gravity (Narong, Win, Pornpot & Thiruchelvam, 2000), the scaling factors is neglected. Instead, in the modelling with enhanced gravity, the scaling factors is a must. So, the result from this data can be used directly in the analysis and calculation.

METHODOLOGY

The works consists of setting up the model tank, sample preparation, simulation of bored pile and loading test as shown in Figure 1. First stage, the work was done as follow:

1. Determination of properties of original clay
2. Setting up clay in LI-1 (slightly more water in it)
3. Installation of 8 bored pile models under 2 nos. of w/c-1; 2 nos. of w/c-2; 2 nos. of w/c-3 ; 2 nos. of w/c-4. During concreting, every pile model was compacted in similar energy (10 blows each)
4. Allowing 3 day curing time then load test to 4 model piles with w/c-1 until w/c-4 as mentioned in Figure 2
5. Allowing 7 days curing time then load test to 4 model piles similar as 3 days curing.

Second stage was to obtain LI_2 , The clay soil should be consolidated under certain loading and drainage valve was opened in a couple of days as shown in Figure 3. Then do the similar steps as no.3 to 5. Third and Fourth stages were done with different loading and consolidation

time to obtain LI_3 and LI_4. To support and synchronize the above work, direct shear test was performed to original clay soil and to a combination of clay + cement mortar with various w/c ratio as mentioned in Figure 4 and 5.

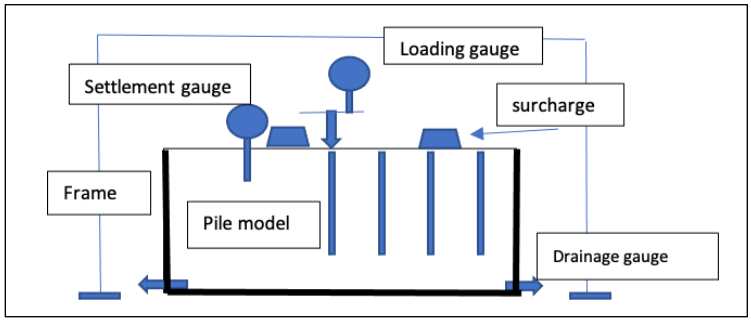


Figure 1. Schematic Diagram of Drum Model



Figure 2. Under Pile Loading Test



Figure 3. During Loading and Consolidation

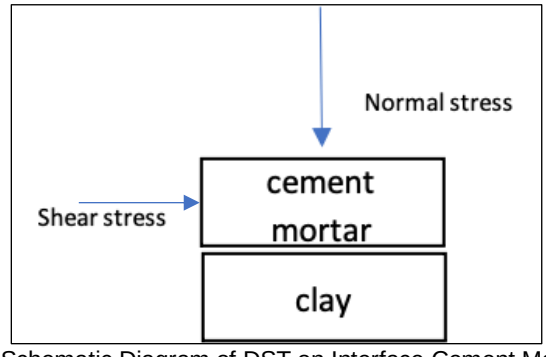


Figure 4. Schematic Diagram of DST on Interface Cement Mortar - Clay



Figure 5. Sample of Clay – Cement Mortar

DATA AND ANALYSIS

Result of physical properties of UNJANI clay soil was as follows: LL = 140; PL = 48; PI = 92. Classification soil as clay high placticity, CH. Cu from Unconfined Compressive Test, UCT = 3.8 kPa. Comparing to the some result compiled at previous work, this Cu value is relatively small and can be classified as extremely soft clay as indicated by high PI and LL. Data for bored pile model simulation is shown in Table 1. Diameter of bored pile model = 22 mm, pile length model = 115 mm, L/D = 115/22 = 5 – 6.

Table 1. Data for Bored Pile Simulation

Soil condition Liquidity Index, LI	Cement mortar w/c-ratio
w_1 = 82 % LI-1 = 0.37	w/c-1 = 0.35
	w/c-2 = 0.51
	w/c-3 = 0.72
	w/c-4 = 0.91
w_2 = 61 % LI-2 = 0.14	w/c-1 = 0.35
	w/c-2 = 0.51
	w/c-3 = 0.72
	w/c-4 = 0.91
w_3 = 54 % LI-3 = 0.07	w/c-1 = 0.35
	w/c-2 = 0.51
	w/c-3 = 0.72
	w/c-4 = 0.91

The load settlement curve of 3 days curing periode from pile loading test were plotted in the Figure 6 , Figure 7 and Figure 8. The curves show that failure occur on 8 – 10 % of pile diameter as mentioned by Terzaghi. At the w/c = 0.40 the pile capacity is the highest, as the low w/c ratio, only small water migrate from cement to surrounding soil to change properties of clay. Application of compaction with 10 blows to each piles were effective by small w/c ratio to improve clay, and it can be monitored during observation in installation. In higher w/c ratio local softening is high as water migrate from cement mortar, compaction showed cement mortar spilled out because of overwhelming of water in it.

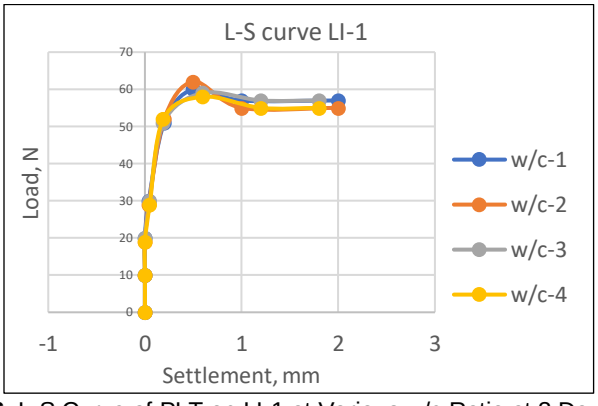


Figure 6. L-S Curve of PLT on LI-1 at Various w/c Ratio at 3 Days Curing

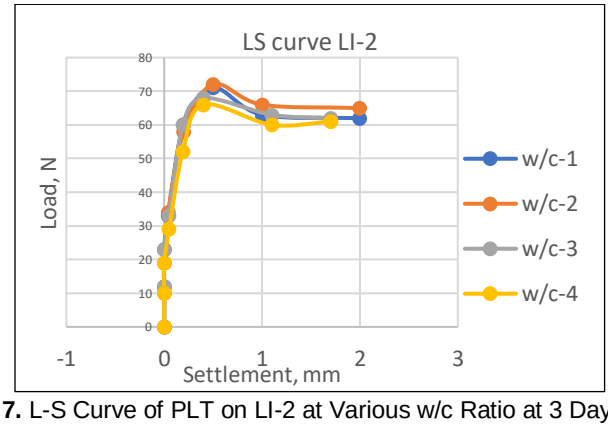


Figure 7. L-S Curve of PLT on LI-2 at Various w/c Ratio at 3 Days Curing

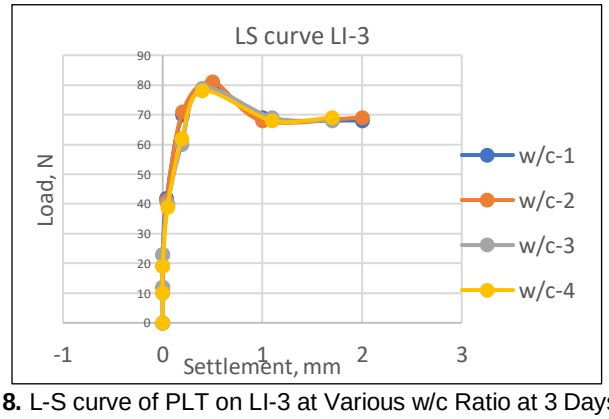


Figure 8. L-S curve of PLT on LI-3 at Various w/c Ratio at 3 Days Curing

The similar result in 7 days curing periode reveals the slight increasing of pile capacity (Figure 9). Eventhough slightly increases, this is an indicator that chemical reaction occurs in there due to some portion of cement pasta concentration penetrates into surrounding clay soil.

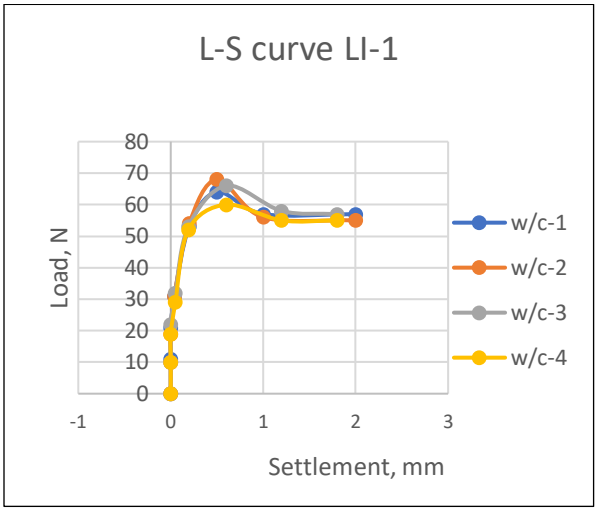


Figure 9. L-S Curve of PLT on LI-1 at Various w/c Ratio at 7 Days Curing

This investigation is also agreeable with the findings by Chuang and Reese (1969), he found that by w/c = 0.2 the local softening of the soil had been avoided. The L-S result of original clay soil with high water content which is soft indicates low pile capacity compared with less water content as can be seen in Figure 10. This is due to migration of water from cement mortar leads to worsen soil condition.

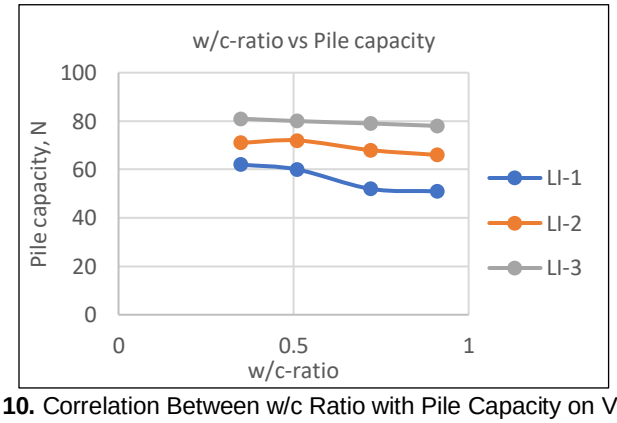


Figure 10. Correlation Between w/c Ratio with Pile Capacity on Various LI

The α values is linearly corresponds with pile capacity, Data results show the correlations between α values and moisture content (α value is conversely dependent with undrained shear strength). When initial moisture content is high the α value is low and vice versa as can be seen clearly in Figure 11.

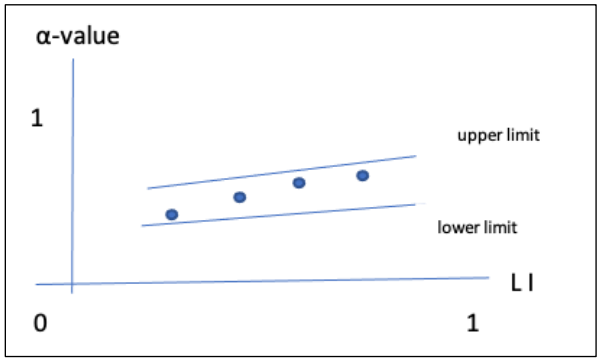


Figure 11. Correlation Between α with LI

DST result on Figure 12, 13 and 14 showed confirmation and support the pile simulation test. Under LI-1 , strength parameter of original clay without cement mortar c = 3.6 kPa and Ø=5^{o} almost similar with UCT result. As layer of mortar with w/c ratio decrease, strength increase. It was comparable with simulation. Also in Figure 12 and 13 which LI-3 < LI-2<LI-1 indicated increasing strength of LI-3 and LI-2 compared to strength in LI-1.

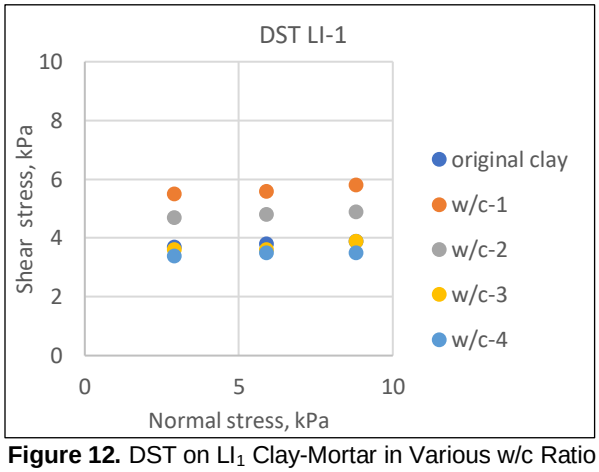


Figure 12. DST on LI_1 Clay-Mortar in Various w/c Ratio

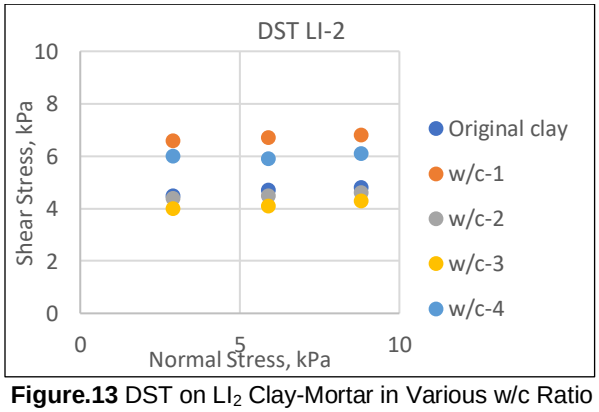


Figure.13 DST on LI_2 Clay-Mortar in Various w/c Ratio

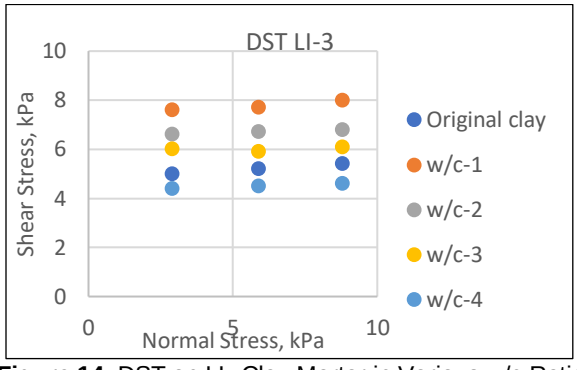


Figure 14. DST on LI_3 Clay-Mortar in Various w/c Ratio

To encourage this compaction pile has an element of ground improvement, let analyze the observation result.

Data of pile model:

Pile diameter = 2,2 cm ; Area of base,
Ap = 0.000298 m^2 ; perimeter, p = 0.069 m
Pile length = 11,5 cm

Loading Test Result

0.06 kN = Cu (0.002682 + 0.007925), So Cu = 0.06 / 0.00903 = 6.64 kPa

Meanwhile, original clay soil has strength properties Cu = 3.8 kPa from UCT and C = 3.6 kPa; Ø = 5o from DST. From this calculation it shows improvement of strength properties is clear (75 % increase).

Another encouraging observation is shown in the Figure 15 and Figure 16 reveals that there are chemical reaction between cement with clay as a result of moisture migration, Some mm thickness of clay-cement mix was produced. Diameter of failure slip surface is slightly bigger than original diameter of bored hole, which in turn increasing pile capacity. The real picture also shown in the Figure 16 on comparison of some extracted pile model (dark color) and the size of bore hole (white color in the middle), clearly showed the dark color size is bigger.

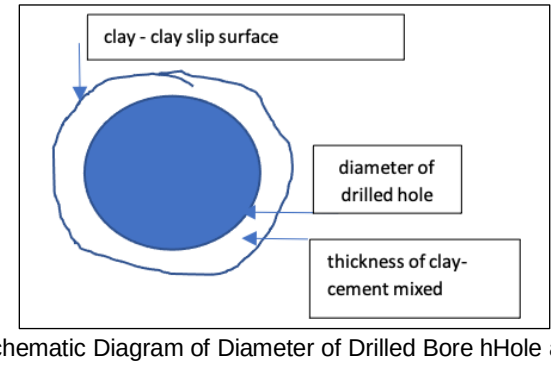


Figure 15. Schematic Diagram of Diameter of Drilled Bore hHole and Slip Failure



Figure 16. Pile Model was Being Taken Out After Tested

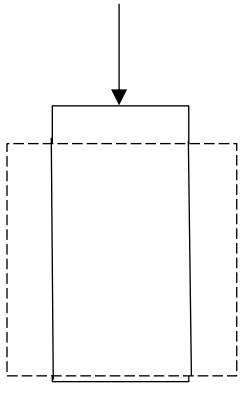


Figure 17. Typical Failure Mode of UCT

Figure 17 indicates the pattern of failure during UCT test to sample of Unjani clay, theoretically it is a condition of failure of soft clay or NC clay. During testing, the rate of loading was set as such to represent undrained condition.

CONCLUSION

Bored pile in clay soil is able to perform as ground improvement pile by imparting energy of hammer blow (or pressure injection, soil displaced, compaction etc.) during installation, selection of water cement ratio of concrete mix as well as allowing pwp to dissipate.

The average increasing capacity is 75 % higher than those without compaction. There are some aspects should be considered in compaction bored pile on clay: water content of original soil, time, water - cement ratio of concrete mix, and energy from hammer blow.

1. Water content of original soil.
   The more water content of original clay the bigger α value as clay soil become softer. This is indicated by lower value of pile capacity of L-S curve from LI-3 to LI-2 and to LI-1.

2. Time.
   Sufficient time is needed to allow dissipation of excess pore pressure after hammer blow. Normal clay needs 30 days to fully dissipation. In this context, fully loading from upper structure is not allowed before complete dissipation.
3. Water cement ratio of concrete mix.
   The lower w/c-ratio provides more pile capacity, minimum w/c – ratio of 0.40 was selected in this type of clay, the other reference shows different value. To obtain minimum w/c – ratio for certain case, It is recommended to do some tests prior to begin installation of bored pile.
4. Energy from hammer blow.
   Hammer blow numbers was not studied thoroughly, too much hammer blow will produce uncontrolled compaction. Insufficient compactive effort will result in inefficient improvement. To control effectiveness of improvement, this blow count should be thoroughly investigated.

The Cu after pile installation become 6,64 kPa instead of 3,8 kPa of original clay, As mentioned in the eq.3 that increased Cu on various radius from it's center is dependent to effective stress, rigidity and Cu before improvement. Therefore, the magnitude of increasing Cu after pile installation is not the fixed value.

RECOMMENDATIONS

Ordinary bored pile installation is without compaction by hammer to the concrete. To optimize energy during bored pile installation, research to obtain optimum hammer blow in clay soil which contribute to maximum improvement is necessary. Due to very rare record of the field implementation to this compacted bored pile concept in clay soil, observations in this area is encouraging.

ACKNOWLEDGEMENTS

This work was financially supported by the LPPM-UNJANI, the authors is grateful to the fully support and encouragement to this project.

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EFFECT OF GROUND GRANULATED FURNACES BLAST SLAG ON KAOLIN PROPERTIES

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Abstract
Kaolin is widely known to possess high compressibility and low strength that leads to infrastructure collapse. Nevertheless, it is crucial to produce soil with such properties because of its immense demand by the infrastructure section amidst the rapidly growing population in Malaysia. The soil must be stabilised before construction so that several critical characteristics can be enhanced. Soil stabilisation finds extensive use in the treatment of the subgrade layers concerning road building. The objective of the study is to ascertain how the physical characteristics of kaolin change upon treatment with different concentrations of granulated ground furnaces blast slag (GGBS); changes in particle size and compaction properties are some areas of interest. Furthermore, the study assesses how a specific quantity of GGBS may be used for enhancing kaolin strength when it is cured for up to 28 days. Seven days curing time for 20% and 25% GGBS-kaolin blend leads to strength enhancement that exceeds the subgrade layer requirement. Considering the case of 25% GGBS-kaolin blend cured for 28 days, the maximum strength was determined to be 8.4 MPa. The inclusion of GGBS leads to changes in crystal phase and structure which can be observed using microstructure analysis.

Keywords: *Kaolin; soil stabilization; ground granulated furnaces blast slag; chemical stabilization.*

INTRODUCTION

Such soil is distributed globally, and it is the primary cause of premature infrastructure failure. Clay is at the top of the list of such soils because varying water content leads to change in volume; furthermore, it has inadequate strength that cannot provide sufficient support to structures (Muhmed & Wanatowski, 2013). Enhancement of soil strength comprises the use of chemical processes that use pozzolanic substances comprising rich proportions of silica, calcium, and aluminium; such treatment is especially beneficial when there is a substantial amount of water in the soil (Walker & Pavia, 2011). Lime and cement are among the widely used pozzolanic substances. Most Asian nations use cement because it is cost-effective compared to lime (Raftari, Rashid, Kassim & Moayedi, 2014). There is widespread use of limestone in Malaysia and that makes limestone the material of choice in Malaysia where it is available in a hydrated form (Saeed, Kassim, Nur & Yunus, 2013). Nevertheless, lime and cement production are associated with copious amounts of carbon dioxide; furthermore, these substances are considered unsustainable. During the late 1940s, the requirement to identify sustainable substances became critical (Muhmed and Wanatowski, 2013).

Solid waste collection rate in Malaysia stands at 76% of which only 5% is recycled; the remaining waste ends up in landfills. It is required that a larger portion of this material be recycled in order to provide better environmental friendliness and sustainability (Jamaludin et al., 2018). It has been established that specific substances are a feasible replacement for lime or cement fully or partially; fly and bottom ash (Jamaludin et al., 2018) and ground granulated blast-furnaces slag (GGBS) are some examples (Saravana, Udhayakumar, Dinesh, Venkatasubramanian & Muthu, 2017; Md Nayan, Ling, Harahap & Ho, 2019).

Kaolinite clay, widely called kaolin, may be divided into primary and secondary classes. The former is produced by the in-situ changes of metamorphic or igneous protoliths rocks; the latter comprises sedimentary rock having high clay content. This material has several applications including paper, plastic, and rubber production (Baioumy & Ibrahim, 2012). Kaolin has widespread availability across Malaysia. Perak, Johor, Selangor, Kelantan, Pahang and Sarawak states are especially rich in kaolin, and the soil reserves in these states are estimated to be one hundred twelve million tons.

Ground granulated blast-furnaces slag (GGBS) is among the primarily accessible by-products in Malaysia; GGBS is produced during the production of iron (Islam, Alengaram, Jumaat & Bashar, 2014). During the reduction of iron ore to elemental iron, GGBS is obtained as a by-product. During the process, the slag is captured and rapidly cooled and dried; it is crushed into a fine powder that closely resembles cement (Yi, Liska & Al-Tabbaa, 2012). The material has a fine texture and off-white colour (Suresh and Nagaraju, 2015) that consists of silica, calcium, aluminum, magnesium, and oxygen (Islam et al., 2014; Jiang, Du & Liu, 2018). GGBS has slow hydration characteristics upon water contact which is because of its low pH; hence, it exhibits a comparatively low strength. GGBS can be utilised separately (Manjunath, Shekhar, Kumar, Kumar & Kumar, 2012; Pathak, Pandey, Murari & Singh, 2014; Chandra & Lavanya, 2017; Mohammadia, Mallikarjuna & Hussain, 2017; Narendra & Prasad, 2017; Mujtaba, Aziz, Farooq, Sivakugan, & Das, 2018) or with other materials (Yi et al., 2012; Maneli, Kupolati, Abiola, & Ndambuki, 2015; Mandal & Singh, 2016; Sharma & Sivapullaiah, 2016; Priyanga, Krishnan & Ravichandra, 2018; Md Nayan et al., 2019; He, Wang, Su, Li & Shi, 2019) to stabilize soil; however, utilising it alone causes an initial low strength gain (Sharma & Sivapullaiah, 2016). It has several benefits compared to cement; one benefit is the reduced production of carbon dioxide – one ton of GGBS production causes 0.07 tons of carbon dioxide emission; this level is about 10% of the amount produced during a similar quantity of cement (Islam et al., 2014; Du et al., 2020). Research concerning waste-treated kaolin is scarce. This research explores the effects of kaolin treated using GGBS under ambient environmental conditions and long curing durations.

MATERIAL AND METHODOLOGY

Kaolin clay was selected as the problematic soil because it is weak and lacks strength; at the same time, it has high compressibility. Brown coloured kaolin comprising very fine particles was used for the present study; these characteristics indicate the existence of iron. This form is widely present in Malaysia (Ipoh, Bidor, Sarawak and Mersing area). The SDN BHD factory KAOLIN (MALAYSIA) was selected for procuring soil. The factory is located at Jalan Tapah – Bidor, 35000 Tapah, Parak, Malaysia. It should be noted that the off-white form of GGBS was procured from a local production facility in Johor. Table 1 lists the characteristics of the GGBS and kaolin employed for the study.

Table 1. Properties of Kaolin and GGBS

Material	Kaolin	GGBS
Liquid Limit (%)	40.5	36.6
Plastic Limit (%)	22.5	-
Plasticity Limit (%)	18	-
Clay (%)	0.65	1.876
Silt (%)	92.6	98.124

Material	Kaolin	GGBS
Sand (%)	6.75	0
Specific Gravity	2.52	2.83
Optimum Moisture Content (%)	18	-
Maximum Dry Density (kg/m^3)	1640	-
UCS (kPa)	115	-
pH	4.33	10.6

The present study assesses the influence of different concentration of GGBS in the 0% to 25% (Table 2) range considering kaolin dry weight on the kaolin characteristics like particle size distribution and compaction. Firstly, raw materials were placed in the oven and cured for 24 hours; subsequently, kaolin and different GGBS concentrations (5, 10, 15, 20, and 25%) were mixed. A laser-scattering based particle size analyser (LA-960V2) was used in conformance to the BS 1377:1990 standard. The compaction test comprised 10% distilled water addition to kaolin, which was then put inside a plastic bag for one day. Different quantities of GGBS were added, and the testing process conformed to the BS 1277:1990 standard. Distilled water was added in 4% increments until the final value of 26% was reached.

Table 2. Properties of oncentration GGBS

	Particle size		
Content of GGBS (%)	Sand	Silt	Clay
0	6.75	92.59	0.652
5	4.07	94.12	1.80
10	3.76	94.20	2.029
15	1.67	97.26	1.071
20	1.46	97.64	0.89
25	0.98	97.99	1.02

The BS 1377: Part 7: 1990 standard was used as a reference to perform the unconfined compressive strength test (UCS). A specific quantity of GGBS was added to Kaolin (5%, 10%, 15%, 20%, and 25% dry-weight kaolin) along with distilled water (at 18% OMC dry mass). The specimens were mixed thoroughly and placed inside cylindrical moulds having 76 mm height and 38 mm diameter. Compaction was continued until the samples reached the required bulk density. Sample weight was taken after the specimens were removed from the moulds; they were subsequently sealed in plastic containers to ensure no loss of moisture. Additionally, the specimens were treated inside a humidity apparatus that was set at 27 ± 2^0C temperature. Curing durations were 0, 7, 14, and 28 days. After the necessary curing was achieved, the samples were weighed and the samples having weight loss in excess of 4 grams were not considered for analysis. The samples were subsequently tested using a shear UCS apparatus that subjected the specimens to axial loading till the failure point (if there is reduction in load or the strain level is 20% or more). The load level at the failure point is recorded; subsequently, water content was assessed by placing the specimens in an oven for 24 hours.

RESULTS AND DISCUSSION

It is crucial to understand soil particle size distribution; consequently, there is a need to ascertain the amount of sand, gravel, silt, and clay so that the most suitable stabilisation

process can be established. It is understood that fine soils react appropriately to chemical stabilisation. Soil classification process also requires the knowledge of soil type.

Figure 1 depicts how different amount of GGBS used for treating kaolin affected particle size distribution. The results indicate that the addition of GGBS to kaolin caused a left-shift in the graph that meant a movement from coarse to a relatively-fine area; the shift was continuous as GGBS content was increased. Therefore, if a fine substance (GGBS) lacking coarse particles (sand) substitutes kaolin comprising of coarse particles, the process can be understood. There is a reduction in sand particles corresponding to an increase in GGBS. This phenomenon is indicated in the below table, whereby the silt, sand, and clay levels at distinct GGBS levels are indicated. Nevertheless, GGBS addition does not lead to significant changes in soil particle size.

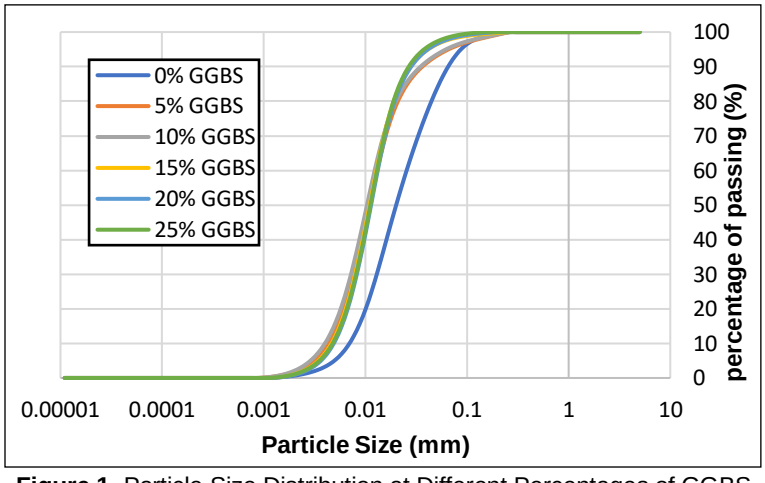


Figure 1. Particle Size Distribution at Different Percentages of GGBS

The treated kaolin was subjected to a compaction evaluation using five variants having different GGBS percentages. The first variant had 5% concentration and the others had successive increments of 5%. This test was performed to evaluate how optimum moisture content (OMC) and maximum dry density (MDD) changed. The results are depicted in Figures 2 and 3.

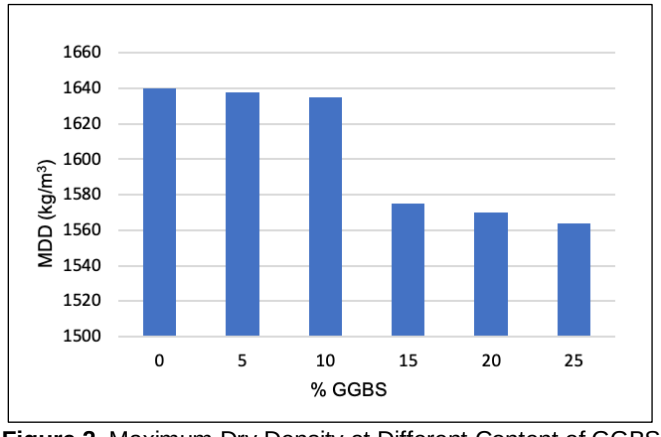


Figure 2. Maximum Dry Density at Different Content of GGBS

Figure 2 presents the highest dry content in the kaolin specimens treated using different GGBS concentrations. Treated kaolin was found to have a maximum dry density of 1640kg/m3. The results indicate that an increase in GGBS is associated with a reduction in MDD. At a 10% or less GGBS concentration, the decline was steady. The slope of the line became higher when GGBS concentration was increased to 15%; consequently, MDD reduced to 1575 kg/m3 from 1635 kg/m3. Till 25% GBSS concentration is reached, MDD reduces slowly to its final value of 1565 kg/m3. Untreated kaolin is associated with reduced MDD as a result of the addition of 25% GGBS (from 1640 kg/m3 to 1565 kg/m3).

Observations concerning stabilising kaolin clay using chemicals based on pozzolanic reactions from previous studies indicate behaviour similar to the present study. The inclusion of pozzolanic reaction substances is associated with MDD reduction. The addition of pozzolanic material such as lime (Muhmed & Wanatowski, 2013; Saeed et al., 2013), rice husk ash (Gupta, Priyadarshee, Sharma & Singh, 2018), cement (Karim, Rashid, Noor & Yaacob, 2014), calcium carbide residue (CCR) (Latifi, Vahedifard, Ghazanfari & Rashid, 2018) decreases the maximum dry density. The primary cause for the decrease in MDD is flocculation and accumulation processes causing the soil to take up more space (Muhmed & Wanatowski, 2013; Saeed et al., 2013). Kaolin possesses such phenomena because the surface is hydrogenous that comprises negative charges around the basal face and positive charge loading at the edges. Particle aggregation is the consequence of the interaction of these charged particles (Hasan & Fatehi, 2019).

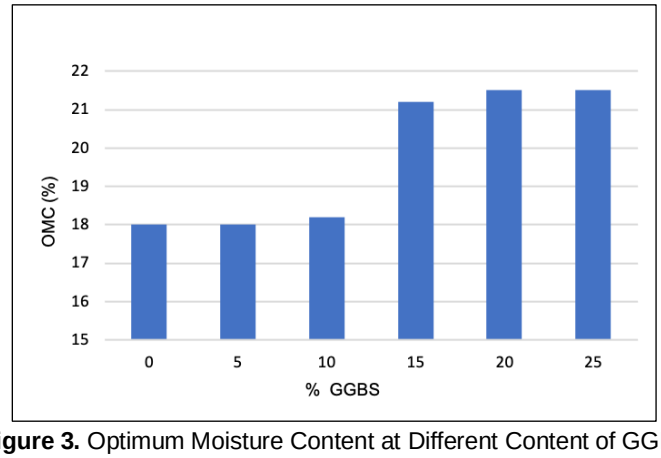


Figure 3. Optimum Moisture Content at Different Content of GGBS

Figure 3 depicts the changes in optimum moisture content corresponding to different GBSS levels. Kaolin clay reaches OMC at 18% moisture, and this value shows minute increment with higher GGBs levels. 10% GGBS level corresponds to 18.2% moisture. Subsequently, the increase in OMC levels becomes more significant in the 10% to 15% range, and finally to the 21.2% level. Subsequently, the gains are not lost once the GGBS levels reach 20% and the corresponding moisture content is 21.4%. There is a consistent increase up to 21.5% moisture till the GGBS levels reach 25%. OMC increased by an aggregate of 3.5% corresponding to a GGBS increase from 0 to 25%.

Previous findings on the treatment of kaolin (Muhmed & Wanatowski, 2013; Saeed et al., 2013; Karim et al. 2014; Saeed, Kassim, Yunus & Nur, 2015; Hasan, Alrubaye, Seng, Ideris

& Marto, 2016; Gupta et al. 2018) using several additives like lime, rice husk, cement, and rick husk ash (RHA) indicates a similar trend of OMC increase corresponding to an increase in the levels of such pozzolanic substances. An increase in additives in the 3-5% range is associated with such a trend. The observed increases can be attributed to the pozzolanic materials needing more water for completing the reaction and the fact that flocs particles accumulate water.

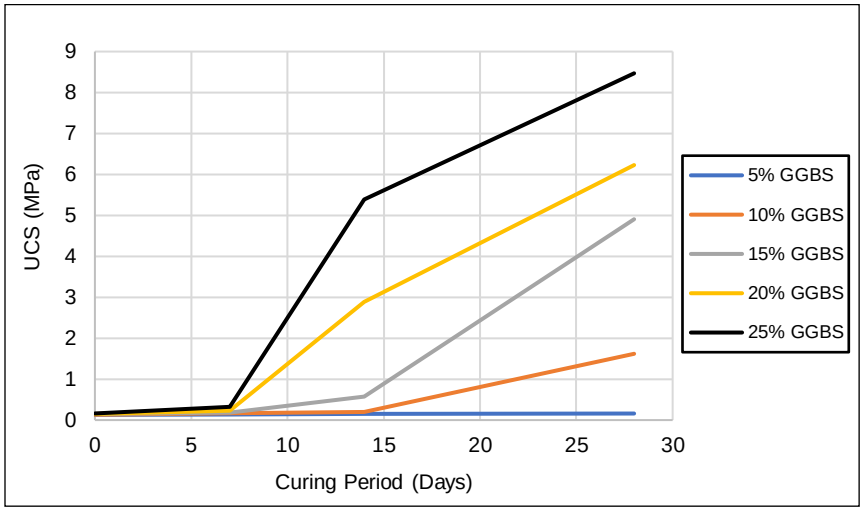


Figure 4. Effect of UCS on Curing Period with Different GGBS Content

Figure 4 depicts how UCS changes curing duration at different GGBS concentrations. The UCS value for untreated kaolin is approximately 115 kPa. Kaolin gains considerable strength when GGBS is added until the 25% concentration level. There is an immediate change in strength from 115 kPa to 162 kPa at 25% GGBS level. When the curing duration was 28 days, 25% GGBS addition led to the highest strength level of 8.45 MPa. The addition of 5% GGBS does not affect strength substantially, considering the 0 to 28-day curing period. Specific to the addition of 10% GGBS, there is a minute increase in strength till 14 days curing; however, a 28-day curing period leads to a substantial increase in UCS to 1.6 MPa.

The addition of 15% GGBS, followed by a 14-day curing duration, is associated with a significant gain in strength that reaches the 0.5 MPa level and keeps rising till the 4.9 MPa level. At 20% and 25% GGBS levels, seven-day curing is associated with appropriate values at 228 and 320 kPa, respectively, which is seen as sufficient for a very stiff subgrade, as indicated by Sharma, Swain and Sahoo (2012). Kaolin treated with 20% to 25% GGBS levels and a curing duration of 14 days led to 2.89 and 5.89 MPa strength levels, respectively. A 28-day curing duration leads to the final strength values of 6.22 and 8.47 MPa, respectively.

The findings indicate the addition of 20% GGBS is associated with more than 200 kPa strength during the initial curing days. The use of 5% GGBS is associated with a minute increase in UCS for up to a 28-day curing duration. There are two potential reasons for this development: firstly, the calcium content in GGBS is lesser compared to cement and lime; secondly, as depicted in Table 3, the pH levels are acidic. It is crucial to have an alkaline environment in order for stabilisation to be effective. Considering a 28-day curing period, a 10% GGBS level is optimal for an adequate UCS value. Kaolin treatment using this level of

GGBS is appropriate if the primary objective is not to have strength gain during initial stages. In case there is a need to have strength gain, it is suggested to use a 20% GGBS level.

The increase in strength can be attributed to the pozzolanic interaction between the calcium contained in GGBS and the aluminium and silica present in kaolin; consequently, calcium silicate aluminium hydrate (CSAH) and calcium silicate hydrate (CSH) gel are produced. When these bonds form, the soil particles are bound together due to which soil strength increases. Hence, considering the specified facts, it is established that GGBS leads to a substantial impact on kaolin strength.

It may be concluded that a relatively long curing duration is required in order to have kaolin attain the requisite strength using 25% GGBS and the subgrade layer. Fourteen days or more is an appropriate curing period.

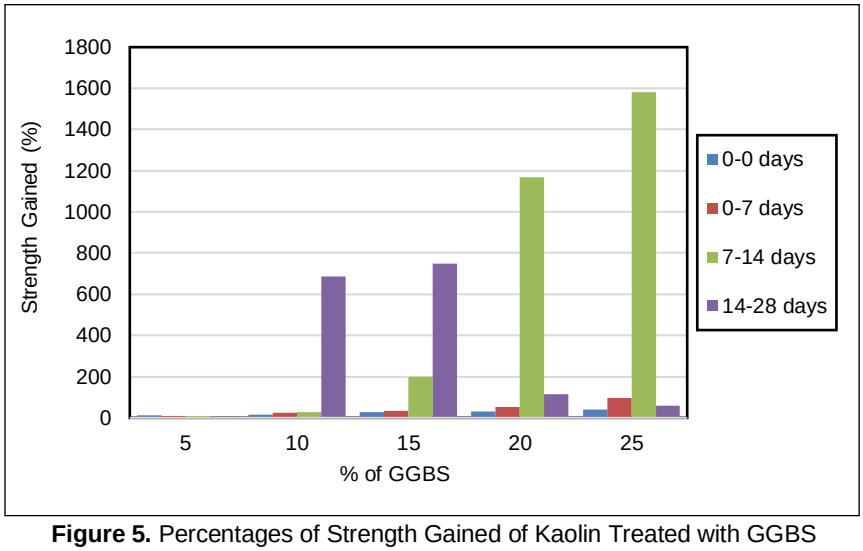


Figure 5. Percentages of Strength Gained of Kaolin Treated with GGBS

Figure 5 displays the proportions of strength gained at distinct stages due to the kaolin treatment with different GGBS content. 0-0 curing represents the strength gained from unprocessed kaolin to the processed kaolin at 0-day curing (sample was examined immediately after de-moulding). The percentage of increase in the strength was determined at the first point; for instance, the strength increased between 14 and 28 days was calculated against the strength gained at 14 days.

For 5% content of GGBS, the strength gained was not too high as it did not strengthen more than 30 percent for all the curing periods, thus indicating that the calcium amount was not sufficient for the reaction. It is true, as explained earlier, that 5% of GGBS content would not cause an increase in the strength. 10% GGBS content demonstrates a very low gain in strength, slightly greater than 5% at initial stages. The gain in strength was very high at the phase between 14 and 28 days as it strengthened by 680%. Likewise, 15% of GGBS content demonstrates a trend similar to that of the 10% content. The gain in strength recorded between 7- and 14-days treatment was 200% which further increased by 780% for treatment between 14 and 28 days. Then again, 20% and 25% GGBS indicates a different pattern since the

percent gain in the strength was considerably high; it reached 1180 and 1600%, respectively. The gain in strength began decreasing for treatment of 14 to 28 days since it indicates an increase of just 100% and 70% for 20% and 25% of GGBS content, respectively.

As mentioned earlier, it can be deduced that 5% content of GGBS is shown to be not enough to treat kaolin at any treatment period. On the other hand, the addition of 10 and 15% GGBS content contributed to a delayed strength gain, at 28 and 14 days, respectively. Therefore, it can be deduced that the optimal content of GGBS that successfully stabilises kaolin is considered to be at least GGBS of 20% content.

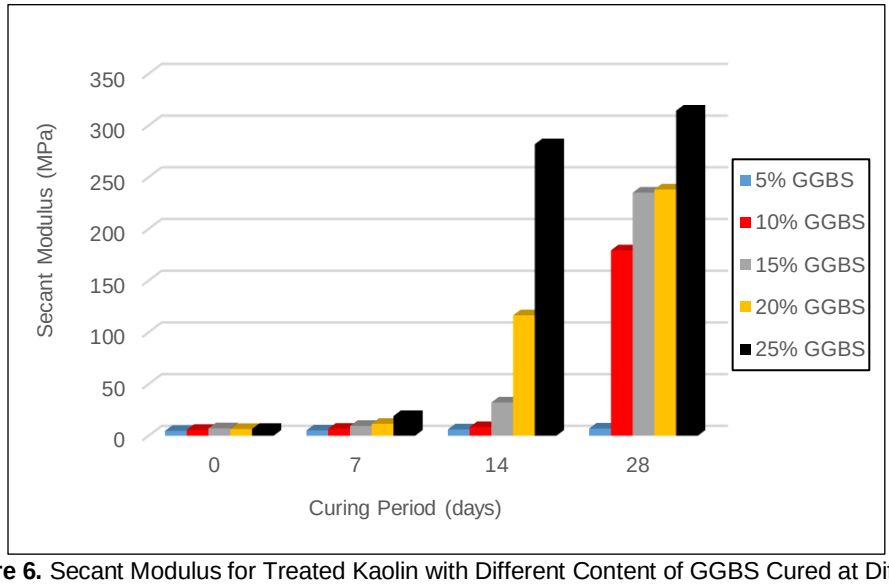


Figure 6. Secant Modulus for Treated Kaolin with Different Content of GGBS Cured at Different Period

Figure 6 displays the secant modulus (E50) computed from the stress-strain plot at half of the strain value at failure for the processed kaolin with different GGBS content under natural fastened condition at various periods. The secant modulus values' trend indicates behaviour similar to the UCS values as it gets increased with an increase in the GGBS content and the treatment duration. It was trivial initially at 0 days treatment, the secant modulus values were between 4 and 8 MPa. On the other hand, for 7 days treatment, the values were a little higher compared to 0 days treatment (5 to 20 MPa). As per Ranjan and Rao (2000), the soil considered for the study was identified as very soft to soft clay on the basis of the values of secant modulus.

At 14 days, the kaolin processed with 5% and 10% GGBS content displayed a very low E50; which was 6 and 8 MPa, in order. The soil was then categorised as soft soil. Moreover, kaolin treated with 15% GGBS displayed slight progress as the soil density was classified as medium according to the same classification; the value of secant modulus was 32 MPa. Treating kaolin with 20 and 25% GGBS for 14 days has demonstrated a substantial improvement in stiffness of soil as the processed kaolin becomes hard (value of secant modulus above 50 MPa). As for 28 days treatment, only processed kaolin with 5% is categorised as soft, whereas for 10% and more GGBS content, the soil has demonstrated very high values of E50, and it becomes hard.

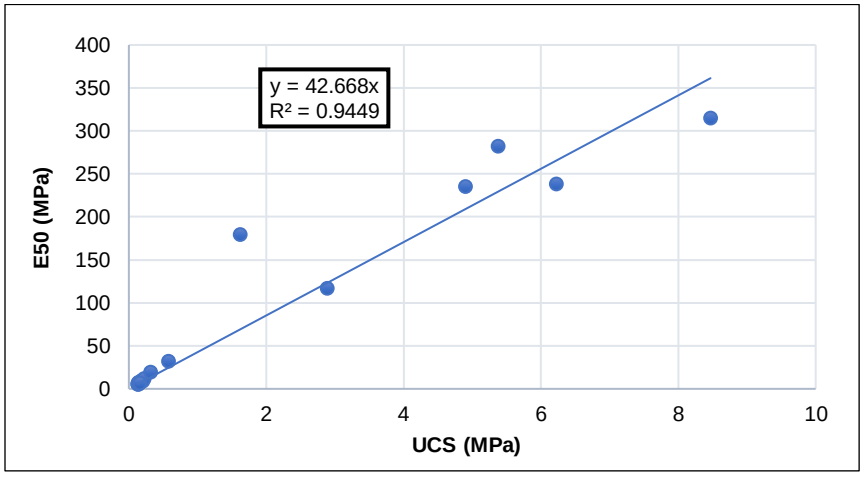


Figure 7. Relationship Between Secant Modulus and Unconfined Compressive Strength Under Natural Condition

Figure 7 displays the correlation between secant modulus computed at half of the strain during failure and UCS for processed kaolin with GGBS under natural treatment condition for up to 28 days. This relationship can be stated as:

$$E50 = \eta \cdot qu \tag{2}$$

E50 represents the secant modulus, qu represents the unconfined compressive strength, and η represents a dimensionless fitting variable (Cai & Liu, 2017). For this study, 20 samples (5 GGBS content for 4 curing periods) were employed to find the correlation between UCS and E50. The outcomes indicate that the secant modulus is nearly 42.67 times of UCS value (η = 42.67) having a correlation coefficient of 0.917, implying that this equation can be considered reliable.

CONCLUSION

This study intended to define the effect of different GGBS content in kaolin characteristics (compaction and particle size distribution). It demonstrates the basic impact of GGBS on soil characteristics in order to establish a base for additional conditions for soil stabilisation. The outcomes are summarised as follows:

a) Particle size analyser demonstrates that addition of GGBS makes the GGBS and kaolin mixture finer (the curve was shifted towards the left, implying the fine part) which is because of the replacement of fine substance GGBS (having no coarse particles) with kaolin (having some coarse particles).

b) The addition of GGBS reduces the maximum dry density; 25 % content of GGBS reduces the MDD (maximum dry density) from 1640 kg/m3 to 1565 kg/m3. It was decreased by 4.6% whereby the reduction is because of the agglomeration and flocculation phenomena.

c) The GGBS addition increases the optimal moisture content for the processed kaolin, 25% content of GGBS raises the OMC by 3.5%. This increase is due to the reason that the pozzolanic reaction uses more water and the fact that water gets accumulated in the flocculation particles.

d) Adding 5% GGBS content to kaolin was shown to have a minor effect on the gain in strength for up to 28 days treatment; this was probably because the pH value did not reach 7%, which is the requirement for an alkaline environment.

e) 10 and 15% content GGBS addition demonstrated very high gain in strength for a treatment of 14 days and more duration; nonetheless, the gain in strength is delayed which is not beneficial for engineering applications. 20% as well as 25% content GGBS addition experience a substantial strength at treatment for lower duration (7 days). Therefore, it makes 20% content of GGBS the ideal amount for kaolin stabilisation.

f) Addition of 20% and 25% content GGBS has demonstrated an enormous gain in strength at 14 days and 28 days treatment; the strength was not less than 3 MPa. The increase in the strength gain is because of the pozzolanic gels formation.

g) At 0-days treatment, the strength gain did not increase considerably for all the mixtures; nonetheless, at 14 days treatment, the majority of the mixtures have demonstrated a good strength gain. Treatment for 14 days is regarded as the effective treatment period for kaolin processed with GGBS.

h) The behaviour of the values of secant modulus is similar to UCS values since it keeps increasing with time and the increase in the GGBS content.

ACKNOWLEDGEMENTS

The authors are grateful for the support by MOE, UTM & VOTE NO. R.J130000.7851.5F197. and Q.J130000.2522.20H06.

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eISSN 2590-4140

