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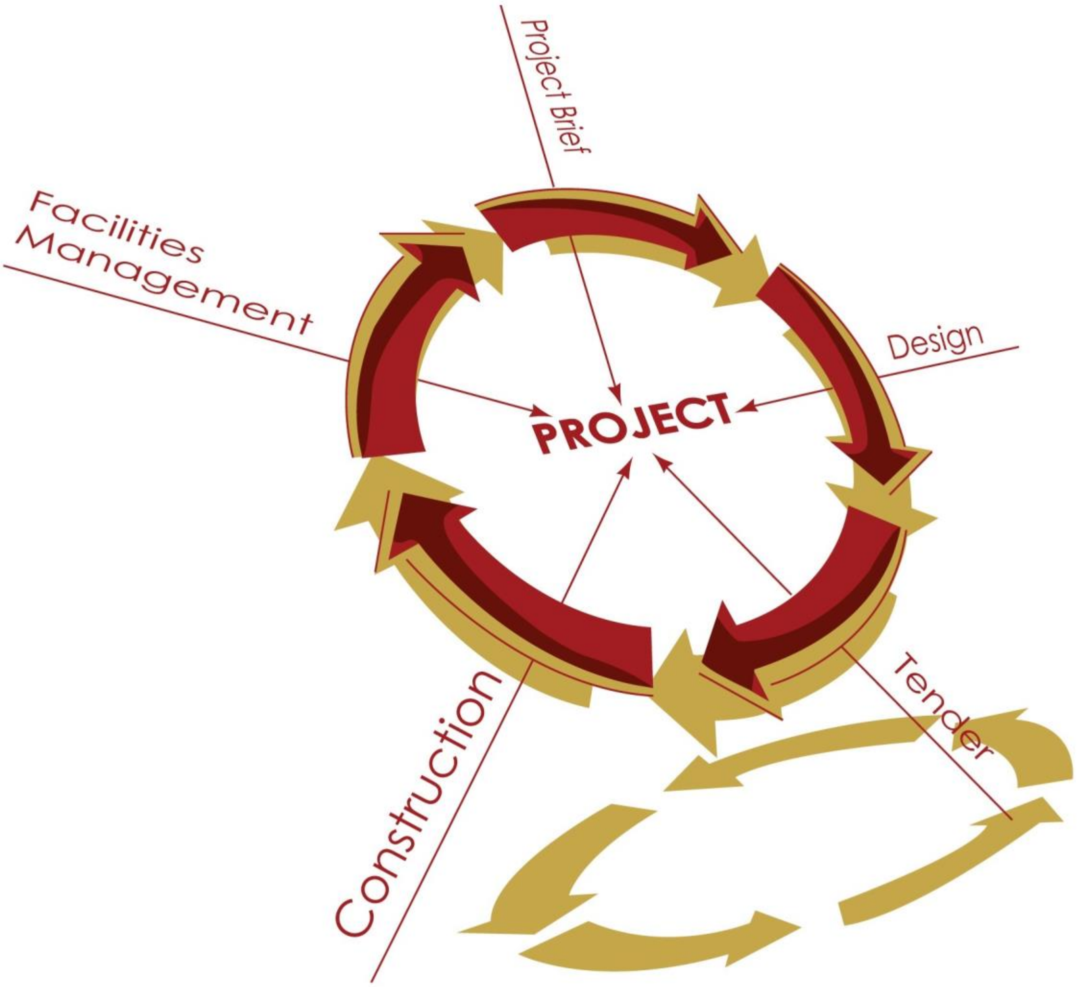

CREAM CONSTRUCTION RESEARCH INSTITUTE OF MALAYSIA
CIDB MALAYSIA

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Contents

INTRODUCTION

Welcome to the Special Issue of Malaysian Construction Research Journal (MCRJ) in conjunction with The Global Congress on Construction, Material and Structural Engineering (GCoMSE) 2017. This special issue is a platform for researchers and engineers to share their knowledge and experiences in creating sustainable and environmental friendly world for the present and the future. As we all know that this world is borrowed from the next generation and in other for us to ensure that it is well looked-after. Therefore, sustainable in construction, material and structural building are important keys to reduce direct impact on the environment. Sustanability in construction building and structural engineering are the optimization of construction activities in way that does not have harmful effects on resources, surroundings and living ecosystem.

This Special Issue of Malaysian Construction Research Journal theme which is ‘Green Technology for Global Awareness and a Better Future’ indicates the needs of preparing the world for future generation to continue their responsibility to make it a better place to live. This theme was divided into 3 major areas which are construction, material and structural engineering. These majors demonstrate the cornerstones of today’s construction engineering and structural research in promoting new ideas, developments and innovations that focuses on new current problems.

This Special Issue of Malaysian Construction Research Journal is significance to the academics and the professionals with research works in construction sustainability by introducing the reduction usage of natural materials and increasing the use of recycled and sustainable materials. By this efforts, pollution and negative environmental impacts can be reduced. This initiative must be followed and well planned towards the sustainable construction and environment. In addition, this special isues also brings opportunity for all authors to share ideas as well as stimulating awareness of this important field. Therefore,this Special Issue of Malaysian Construction Research Journal are consist of 20 selected papers by the conference committee and expert reviewer from the papers submitted at the GCoMSE 2017.

Editorial Advisory Board

Editorial

Welcome from the Editors

Welcome to the Special issue of Malaysian Construction Research Journal (MCRJ) for The Global Congress on Construction, Material and Structural Engineering. The editorial team would like to extend our gratitude to all authors and reviewer for their contributions and valuable comments. The purpose of this special issue is to share their knowledge and experiences in creating sustainable and environmental friendly world for the present and the future. It is hope that the readers will find beneficial information from this special issue edition. Twenty (20) papers are discussed in this issue.

Abdellah et al., determined benefits of NZEBs practices through preliminary investigation among architects. Several existing energies works of literature and pilot studies by using semi-structured interviews were conducted. The findings reveal that, although the term 'NZEBs design strategies' is not being used broadly across construction industry in Malaysia, some green practices related to the design stage of construction has been carried out. The findings are also encouraging in increasing awareness, practices and implementation of NZEBs design strategies by the practitioner in Malaysia.

Mohamed Sulzakimin et al., carried out a comprehensive review on FSM within the context of Malaysian public universities and sustainability from relevant databases by examining the possibility of reducing incidents that could lead to expending unbudgeted resource to rehabilitating property destroyed by fire on campus, in addition to limiting risk to life and interruption of academic and business activities. Several types of research had been conducted on FSM, nevertheless very few consider Higher Education Institutions (HEI) holistically. The paper reveals that FSM is critical to preventing fire disaster in buildings. It identifies the need for an investigation into the implementation of FSM to developing effective FSM framework for assisting facilities managers and other stakeholders to preserve the university buildings against fire risks.

Haqeem Hassan et al., investigated the suitable waste materials to be used in solidification/stabilization (S/S) technique using palm oil fuel ash and quarry dust. In this research, different percentages of Palm Oil Fuel Ash (POFA) and quarry dust (0%, 10%, 20%, 30% and 40%) are used to replace the Portland cement and sand respectively. The results indicated that 10% of POFA and 20% of quarry dust are the best percentages to be incorporated in S/S matrices. The 10% replacement of POFA recorded a considerable value of density ranging from 1500 kg/m^3 to 1660 kg/m3.Meanwhile the compressive strength 10% of POFA achieved the strength of 22.60 MPa. Besides that, 20% replacement of quarry dust in natural sand recorded a value of density ranging from 2080 kg/m^3 to 2147 kg/m^3 with the highest strength is 24.91MPa.

Nurain Izzati Mohd Yassin et al., presented an overview of the performance of sustainable concrete masonry containing industrial waste as cement replacement which exposed to high temperature. Based on previous researches, the performance of these sustainable concrete have been investigated in terms of its residual compressive strength and physical properties at elevated temperature. Large numbers of research on pozzolanic concrete at elevated temperature have been conducted where pozzolanic concrete have better performance than OPC concrete at temperature less than 500°C. In terms of physical properties, surface cracks and discolouration of concrete has been observed. Studies show that, surface cracks on pozzolanic concrete were less than OPC concrete. Meanwhile, for discolouration impact, it was observed that, both OPC and pozzolanic concrete have similar changes in colour where the colour of concrete became lighter as the firing temperature increased.

Faisal Sheikh Khalid et al., established sustainable material properties for sand cement composite brick containing Recycles Concrete Aggregate (RCA) and Polyethylene Terephthalate (PET). This study replaces the fine aggregates with 25%, 50% and 75% of Recycled Concrete Aggregate (RCA) and 0.5%, 1.0% and 1.5% of waste PET in the production of brick. The best compressive strength was recorded at 24.9 MPa for brick contains 75% RCA and 0.5% PET. Meanwhile, an increment of RCA contents in brick increased the water absorption and Initial Rate of Suction (IRS) percentages compared to a normal brick. As a result, the use of RCA and PET in the production of sand cement composite brick has provided better strength with tolerable results on water absorption and initial rate of suction compared to normal bricks.

Mohamad Syamir Senin et al., investigated the changes of the properties for rubber ash cement mortar with respect to different content of rubber ash as cement and as sand substitution and filler. Cubic specimens were produced by adding 10% volume rations of rubber as fillers, sand replacement and cement replacement into M30 quality cement mortar. The physical and mechanical tests were conducted at the end of day 7, 14 and 28. Tire rubber ash is more suitable to be used as sand replacement. It also works as an alternative to cement replacement and fillers. The compressive strength of the mortar specimen reduced when rubber ash was added in the cement mortar where reduction of density occurred. At the early stages, better compressive strength for tire rubber ash used as fillers however at 28 days the use of tire rubber ash as cement replacement had the strongest compressive strength.

Bassam. A. Tayeh et al., investigated the performance of cement mortar containing Glass Powder (GP) as a partial substitution for cement. Portland cement was partially replaced with 10%, 20% and 30% GP. The unit weight and compressive strength of the cement mortar after a curing period of 7 and 28 days were measured. Although the compressive strength of the cement mortar decreased, its unit weight had no significant change and remained equal to those of the control samples. Therefore, GP can be considered as a viable replacement for cement and is thus an economical construction material

Mohd Khairy Burhanudin et al., conducted a study to determine the strength of concrete containing Coal Bottom Ash (CBA) as cement replacement material. The original CBA were sieved passing 150 um sieve size then ground for 20, 30, and 40 hours using ball mill machine. The normal concrete were designed for grade 30 based on Department of Environmental method (DOE). The concrete were produced by replacement level of cement 10%, 20% and 30%. The fresh concrete were tested using slump cone to determine the workability of concrete. It is found that slump height was decrease with increasing the replacement level of CBA. Besides that, from the result, 20% replacement level of cement with 30 hours grinding time contributed high compressive strength compared to others. The presence of CBA in cement as a binder was improves the strength of concrete.

Utilizing powder waste produced from asphalt batching plant so called Asphalt Dust Waste (ADW) was an alternative solution to reduce this waste material that cause pollution to surrounding dumping area. **Isham Ismail et al.,** evaluated the optimum percentage of ADW as filler in coarse aggregate and the effect of ADW as cement replacement in develops Self Compacting Concrete (SCC). According to the results obtained, utilizing ADW as filler and cement replacement in develop SCC was benefit for high workability, stability and segregation resistance in fresh state conditions. As conclusion, the utilization of ADW for the production of self-compacting concrete is perfectly possible and also potentially reducing amount of cement in concrete mixture with high workability.

This research had been conducted by **Nik Mohd Zaini Nik Soh et al.,** to explore the suitable proportion of Sodium Hydroxide (NaOH) treatment for Oil Palm Empty Fruit Bunch (EFB) fibre to increase the compatibility of cement with EFB fibre. The fibre treated with NaOH has shown a significant different on the hydration temperature for EFB fibre- cement mixed compared with the untreated fibre. The higher NaOH concentration, the greater hydration temperatures obtain. Besides that, the increment NaOH concentration applied, the rougher EFB fibre surface is observed with lesser silica body remain. The tensile properties of individual fibre treated with NaOH (0.4%, 1% and 4%) has shown significant increment as compare to the untreated fibre.

Teddy et al., conducted a study on different Sulphate Reduction Bacteria (SRB) concentration (0%, 3%, 5% and 7%) and water cement ratio (0.4w/c, 0.5w/c and 0.6w/c) in term of compressive strength and water penetration. Test results indicated that the best SRB concentration to be mixed in bio-concrete was 5% while 0.5w/c for water cement ratio. Both values enhanced significantly the compressive strength and reduced porosity and water penetration of bio-concrete. These improvements were due to deposition on the SRB cells within the pore of the concrete cube as cured in chloride water. Results of this study demonstrated the role of SRB induced calcium carbonate precipitation in improving the concrete structure cured in extreme condition in term of compressive strength and water penetration.

From previous studies, the strong acid leaching treatment was carried out to remove metallic impurities in Coal Bottom Ash (CBA). The usage of strong acid could significantly hazardous to human and environment. **Yahya et al.,** conducted a study on the leaching process by replacing the usage of strong acid with citric acid which acted as weaker acid. Experimental result shows that the acid leaching treatment has potential to be used in reducing metallic elements in CBA under optimum solution temperature of 400^{o}C with 4% acid concentration at 60 min reaction period. The characteristic of CBA was also determined by comparing the SEM, particle distribution, specific gravity, water content and setting time of raw CBA with treated CBA.

Interlocking Compressed Earth Brick (ICEB) are cement stabilized soil bricks that allow for dry stacked construction. **M.M. Zamer et al.,** used Ureolytic Bacteria (UB) in this study as a partial replacement of limestone water with percentage of 1%, 3% and 5%. Enrichment process was done in soil condition to ensure the survivability of UB in ICEB environment. This paper evaluates the effect of UB in improving the strength and water absorption properties of ICEB and microstructure analysis. The results show that addition of 5% UB in

ICEB indicated positive results in improving the ICEB properties by 15.25% in strength, 14.72% in initial water absorption and 14.68% reduction in water absorption.

Mohamad Azim Mohammad Azmi et al., investigated the addition of cement and bagasse ash in contaminated landfill soil in term of its engineering properties such as the Atterberg limit, specific gravity, linear shrinkage and soil compaction. Tests were conducted on three different groups of soil samples: landfill soil as the control sample, samples which contained cement only (5%, 10%, 15% and 20% of cement) and samples which contained a mixture of cement and bagasse ash (BA) (2.5% cement + 2.5% BA, 5% cement + 5% BA, 7.5% cement + 7.5% BA and 10% cement + 10% BA). The addition of cement and bagasse ash increased the optimum moisture content and reduced the maximum dry density of soil. As conclusion, the use of cement and bagasse ash in this study improved the engineering properties of contaminated landfill soil.

Lightning Protection System (LPS) plays a vital role to protect the structure by dissipate the lightning current to underground safely. New installation method of LPS has implement in Malaysia which to embed the lightning protection cable in a concrete structure. **Mustaqqim Abdul Rahim et al.,** carried out a study to determine the natural frequency of additional reinforcement concrete beam by using impact hammer test. The results shown that the natural frequency of additional reinforcement concrete beam has slightly decreases after strike of electric current. From the results also found out that the location of lightning protection cable to attach in the concrete beam will influence the natural frequency of concrete beam.

Shahrul Niza Mokhatar et al., explored the impact damage of reinforced concrete beams subjected to low velocity impact loading at the ultimate load range. In this study, an impact tests is carried out on reinforced concrete beam consisting Modified Artificial Polyethylene Aggregate (MAPEA) as the replacement of coarse aggregate. There are twelve beam specimens of size 120 mm x 150 mm x 800 mm are categorized into three groups which denoted as Normal Reinforced Concrete (NRC), reinforced concrete with MAPEA concrete block infill (RCAI) and reinforced concrete with 9% of MAPEA as a coarse aggregate (RC9A). The result of the laboratory test showed that the RC9A beams produced less crack and low value of residual displacement.

Composite slab usually consists of corrugated steel deck and concrete topping. Therefore, **Zainorizuan Mohd Jaini et al.,** introduced foamed concrete as concrete topping to reduce the self-weight of composite slab. More focus was given on the effect of thickness toward the natural frequency, damping ratio and energy dissipation of composite slab. The composite slab was cast with the size of 840mm width, 1800 length and five different thicknesses range between 75mm to 175mm. Foamed concrete was deliberately designed for the density of 1800kg/m^3 with the utilization of Rice Husk Ash (RHA) as partially sand replacement and Polypropylene Mega-Mesh (PMM) as fiber reinforcement. It was observed that the natural frequency decreases with the increment of thickness. The damping ratio and energy dissipation indicate a significant increment correspond to the thickness of composite slab.

Ma Chau-Khun et al., presents a critical review on the state-of-the-art of the current research development on pre-tensioning steel straps confinement. The paper was started with highlighting the features of such confining technique, then an extensive description of the connection clip systems was presented. The experimental and numerical investigations performed on steel-strapped concrete structures were discussed. This paper also covered the potential usage of pre-tensioning steel straps confinement in increasing the bond properties of confined concrete and as a repairing technique of damaged concrete.

Shahrin Mohammad et al., presents a study on the development of a nonlinear numerical analysis that is capable of simulating the quasi-static semi-rigid behaviour of structural steel frames under variable loadings for reliable design. The beam element shape functions with semi-rigid connections are formulated and the overall stiffness matrix is derived. The theory to solve the nonlinear analysis is also presented. The developed procedures also revealed the complex finite element calculations in order to identify the structural performance of nonlinearity for steel structures. The major modifications that include the revised solution procedure, the variable load pattern capability, the quasi-static connection model, the modification of input data to cater for initial imperfections and the inelastic constitutive model for steel are presented. From the results, the modified procedures are capable to predict the behaviour of the structural steel frames under varying loading conditions with various connections configuration and material properties.

Concrete-Filled Double Skin Steel Tubular (CFDST) columns were often used in outdoor construction where fire is not a main concern. Therefore, this series of research, **Sharifah Salwa Mohd Zuki et al.,** deals with the behaviour of Concrete-Filled Double Skin Steel Tubular (CFDST) columns after fire exposure, residual strength and method of repairing fire-damaged columns. This particular paper focused on the effectiveness of Hybrid Fiber Reinforced Polymer (FRP) repairing method. It was found that by using Hybrid FRP, the ultimate strength at failure of repaired specimens greatly increased when compared to fire-damaged specimens to the extent of exceeding the control specimens. In addition, FRP is also effectively confined thinner outer steel tube than thicker outer steel tube.

THEME 1:

CONSTRUCTION

INVESTIGATING BENEFITS OF NET ZERO ENERGY BUILDINGS (NZEBS) FOR HEALTHCARE BUILDINGS: PRELIMINARY FINDINGS FROM ARCHITECT'S PERSPECTIVE

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Abstract
In recent years, Net-Zero Energy Buildings (NZEBs) concept has received incrementing attention especially since European Union Parliament are progressively moving towards regulation in which all new buildings to be "nearly Zero-Energy" Buildings by 2020. In the context of Malaysia construction industry, the government has a significant concern about energy consumption and the negative impacts of inefficient of energy usage. Although this concept provides promising benefits, however, previous studies found that the benefit of NZEB is still doubtable to the Malaysian construction industry, particularly for healthcare buildings. The NZEBs concept implementation is crucial in healthcare building as it seen as a key part of the needed transition towards sustainable energy efficiency as well as co^2 emissions control in its energy dimension. This paper aims to determine benefits of NZEBs practices through preliminary investigation among architects. Several existing energies works of literature and pilot studies by using semi-structured interviews were conducted. The findings reveal that, although the term 'NZEBs design strategies' is not being used broadly across construction industry in Malaysia, some green practices related to the design stage of construction has been carried out. The findings are also encouraging in increasing awareness, practices and implementation of NZEBs design strategies by the practitioner in Malaysia. It is envisaged that the paper will provide a basic knowledge for future research in NZEBs design strategies practices for healthcare buildings construction in Malaysia.

Keywords: *Net zero energy buildings; Healthcare buildings; Benefits; Architect's perspective; Malaysia*

INTRODUCTION

Net-Zero Energy Buildings (NZEBs) is a concept that uses renewable energy technology to produce a mix of renewable electricity and other renewable heat resources like biomass and solar to generate as much energy as they consume (Antonin et al., 2014). Similarly, Sartori et al. (2010) emphasized that NZEBs is conceptually a building with a lower of energy demand and are balanced by the on-site generation of electricity, or other energy carriers, from renewable sources. Moreover, another study by Kurnitzki et al. (2011) and Voss et al. (2011) highlighted that Net-Zero Energy Buildings (NZEBs) are typically a grid-connected building with very high energy performance. On the other hand, NZEBs are buildings with the concept of cost-optimal in which ensuring the minimum cost of the estimated economic life cycle of the building, by taking the package of efficiency measures.

Despite today's buildings energy consumption demand increasing significantly, NZEBs concept practices enable to develop a network of clean domestic energy assets. The timeline for the implementation of NZEBs according to the Energy Performance of Buildings Directive (EPBD) recast is illustrated in Figure 1.

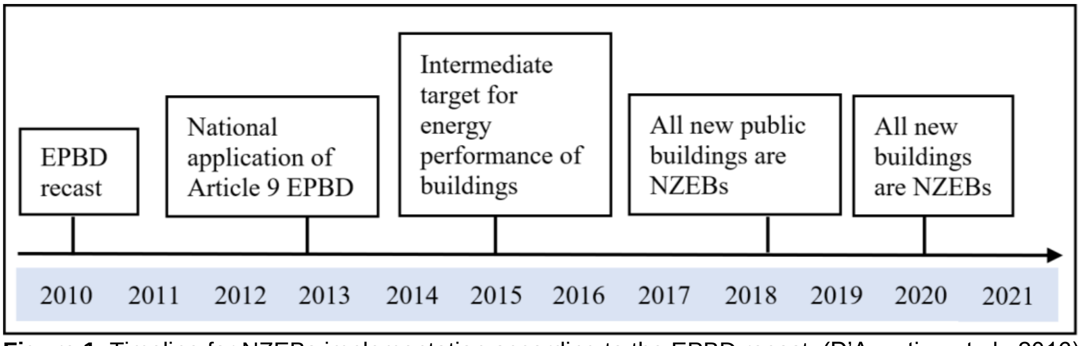

Figure 1. Timeline for NZEBs implementation according to the EPBD recast. (D'Agostino et al., 2016)

As highlighted in Figure 1, NZEBs has gained much attention in 2010 through EPBD recast. Based on the recast, European Union Parliament had agreed to set the targets for all members of Parliament to regulate a regulation in their counties that all new buildings are NZEBs in 2020. Thus, NZEBs has been seen as one of important concept that need to be address and implement in every country. Besides that, the NZEBs implementation in healthcare building is a key part of the needed transition towards sustainable energy efficiency as well as (CO_2) emissions control in its energy dimension. According to Morgenstern et al. (2016), the healthcare buildings have high electricity consumption as the result of the extensive use of various medical equipment, mechanical ventilation of building, lightings, and sterilization. Besides that, some of the countries have shown that healthcare buildings are considered as one of the most energy-intensive commercial-sector buildings. For instance, healthcare buildings in the United States are considered as one of the most intensive users of total energy among the building types where the high electricity consumptions are due to the continuity of services and the utilizations variety of complex applications in energy-intensive equipment such as medical imaging equipment (USEIA, 2016).

In the context of Malaysia's construction industry, there is an upward trend of demand in energy consumption in building sector as the energy consumption in the commercial and residential building had an increase of 2.7% and accounts for up to 14.4% in 2013 as compared to 11.7% in 1993 (Suruhanjaya Tenaga, 2015). Additionally, Saidur et al. (2011), analysed the energy use, energy savings and corresponding emission reductions for the energy using equipment in a public hospital in Malaysia. The authors reported that around 19,311 MWh of energy consumption for the hospital in 2008. Besides that, for the end-use energy breakdown for the hospital, the authors found that lighting uses a major fraction of total energy consumption comprises of 36% and followed by medical equipment which accounts for 34%. The findings also showed that the energy intensity of the hospital is 234 kWh/m.

In response, the aim of this paper is to present findings based on an exploratory study of architect's perspectives in terms of their knowledge of NZEBs and sustainable practices in Malaysia. An emphasis is given on the NZEBs benefits through NZEBs design strategies since the literature finds there are many possible ways of design strategies to achieved NZEBs. This paper intends to fill gaps in our understanding of benefits that can be gained from NZEBs concept in Malaysia healthcare buildings through NZEBs design strategies and necessary to recommend ways for constructions practitioners to design a healthcare building according to NZEBs concept while maintaining quality in the delivery of health services.

LITERATURE REVIEW

The objective of this section is to review the Malaysia healthcare projects developments as well as an overview of sustainability in construction. Moreover, the existing NZEBs definitions framework will be reviewed in this section as it is important to be clear and understand for this paper. Besides that, this section will also explain the principle of NZEBs and the NZEBs adaptation in healthcare building design.

MALAYSIA HEALTHCARE PROJECT DEVELOPMENT

In general, the total health expenditure for Malaysia's national spending shows an increasing trend from RM8,303 million in 1997 to RM44,748 million in 2013 (MoH, 2015). From this trend, it is indicating that the Malaysia's health expenditure is growing exponentially over sixteen years. Even though there is a breakdown of several types of expenditure, but most of this expenditure is spent for curative care whereby curative care services provided by hospitals include in-patient, out-patient, day-care services with minimal home care services in which consuming 61 percent of the total expenditure as illustrated in Figure 2.

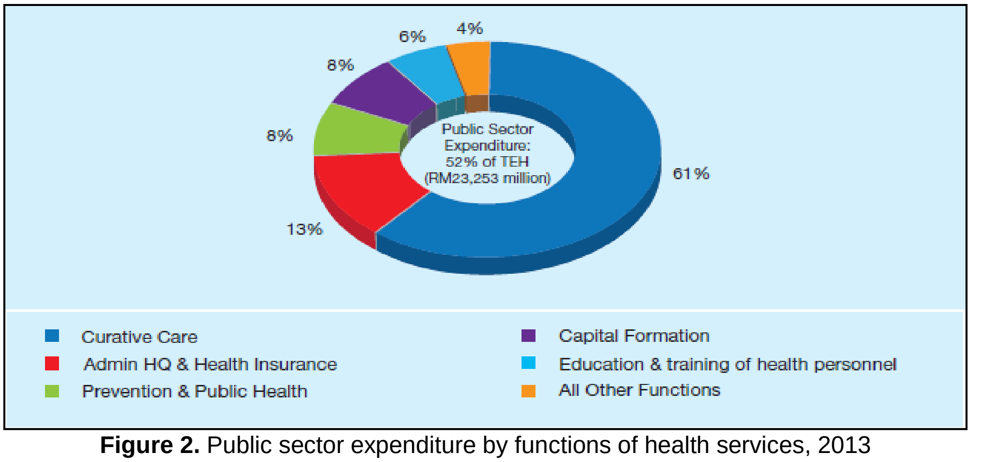

Figure 2. Public sector expenditure by functions of health services, 2013 (Malaysia Ministry of Health, 2015)

Furthermore, based on the Tenth Malaysia Plan (10MP – 2011-2015), there are four strategies that need to be implemented in order to support one of the trust in 10MP which is to achieve the quality of life of an advanced nation. The four strategies involved are: 1. Establish a comprehensive healthcare system & recreational infrastructure, 2. Encourage health awareness & healthy lifestyle activities, 3. Empower the community to plan or implement individual wellness programme (responsible for own health), 4. Transform the health sector to increase the efficiency and effectiveness of the delivery system to ensure universal access. Based on those strategies, three Key Result Areas (KRA) were identified which comprising health sector transformation towards a more efficient and effective health system in ensuring universal access to healthcare, health awareness and healthy lifestyle and empowerment of individual and community to be responsible for their health (MoH, 2012). Therefore, The Country Health Plan: 10th Malaysia Plan 2011-2015 has highlights several physical healthcare development projects that will need to be implemented in order to achieve KRA (MoH, 2012).

As a matter of facts, since 2007 until 2014, the development progress of healthcare buildings projects in Malaysia is consistent every year although, for the past four years, the allocation budget for the healthcare project development is decreasing in 2014 as compared to 2011. However, as compared to 2010, there is an increase of 2 new hospitals in 2014. Moreover, up to 2014, there are 133 government hospitals and 9 special medical institutions which comprise of 35,318 beds and 4,942 beds respectively (MoH, 2014). Overall Bed Occupancy Rate (BOR) for MoH hospitals and Institutions in 2014 was 71.79%. Besides that, in the Tenth Malaysia Plan (10MP – 2011-2015), the government has approved an allocation of RM21.98 billion in 2014 to be allocated for MoH which consists of RM20.49 billion for the Operating Budget and RM1.49 billion for the Development Budget.

The initial allocation for the development budget that has been approved by the government for MoH in 2014 was RM 1.662 billion which includes constructing new hospitals, upgrading existing hospitals, facilities and others (MoH, 2014). However, the allocation was reduced to RM1.495 billion due to warrants restrictions. MoH has spent 92.90% of the adjusted development budget allocation, or equivalent to RM1.389 billion which comprised of 71 new MoH programs. Based on 71 new MoH programs, it is involved in 67 physical projects and 4 non-physical projects (MoH, 2014). Moreover, for the physical projects, Public Works Department (JKR) will handle 53 projects while JKR Sabah and JKR Sarawak will handle 4 projects and 3 projects respectively. The remaining 4 projects will be handled by the Engineering Services Division (ESD) of MoH (MoH, 2014).

Therefore, it is important to consider the healthcare buildings sustainability in the long term. Some of the challenges including low building performance and high energy consumption for Malaysian healthcare buildings need to be address in order to ensure the sustainability of healthcare buildings can be performed and thus the KRA in which to support the health sector transformation towards a more efficient and effective health system in ensuring universal access to healthcare could be achieved.

OVERVIEW OF SUSTAINABILITY IN CONSTRUCTION

Recently, there is a growing attention given towards sustainability in construction or sustainable development. Generally, sustainability can be described as the use of natural resources whereby these resources are in such an equilibrium condition that do not attain deterioration, depletion and non-renewable point and these resources able to be used for the next generations (Yılmaz & Bakış, 2015). In addition, the term sustainable development can be defined as improving social, economic and environmental conditions by increasing the quality of life of people and allow them to live in a healthy environment (Ortiz et al., 2015). In other words, sustainability enables people to foresee a continuous development without take it for granted of the resources that were used in order to prevent resources depletion for the next generations.

In general, sustainability has three main components namely environmental, economic, and social as illustrated in Figure 3. For the sustainability of construction, it is directly involved in the environmental aspects of sustainability. Moreover, the NZEBs concept in constructions will enable to support the environmental aspect.

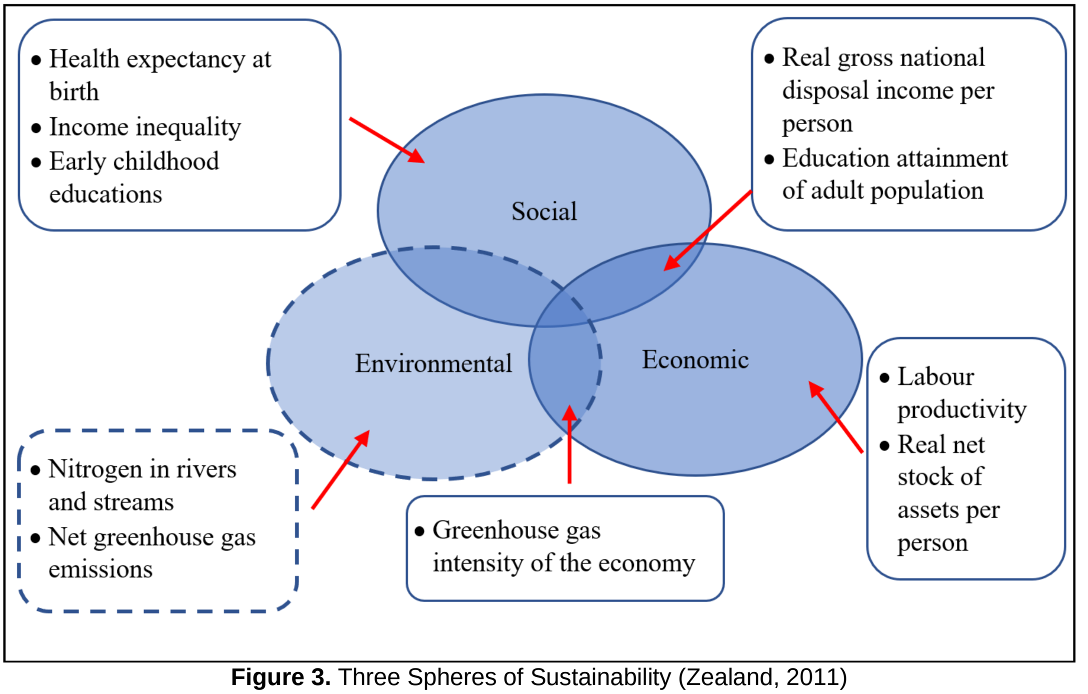

Figure 3. Three Spheres of Sustainability (Zealand, 2011)

The concept of environmental sustainability can be described as the transformational changes for the built environment, whereby the energy consumption levels are reducing and natural resource depletion problems are able to overcome (Wong & Zhou, 2015). Besides that, sustainable resources are dependent on their ability to renew themselves. Furthermore, it is essential to consider the rates of renewal of resources and the rates of resources for clearance of contaminants are not exceeding when determining usage level of natural resources (Patel & Chugan, 2013). Additionally, environmental sustainability involves the subjects of (Yılmaz & Bakış, 2015);

- Protection of aliveness and diversity on the earth,
- Conservation of life-support systems,
- Sustainable usage of renewable resources,
- Being saving in using unrenewable resources,
- Minimizing harm to the environment and living things, and
- Protection of cultural and historical environments.

Therefore, in order to overcome the increasing concern of today's resource depletion and to address environmental considerations in both developed and developing countries, NZEBs is beneficial to apply for healthcare buildings design since it able to improve sustainability in the construction industry.

UNDERSTANDING NZEBS DEFINITION

Initially, there are four main primary definitions of NZEBs for the earliest definition and classification of net-zero energy buildings (NZEBs) can be found in the study by Torcellini *et al.*, (2006), in which the authors had addressed the four main definitions as:

- **Net Zero Source Energy** - A source ZEB produces at least as much energy as it uses in a year when accounted for at the source. Source energy refers to the primary energy used to generate and deliver the energy to the site.
- **Net Zero Energy Costs** - In a cost NZEB, the amount of money the utility pays the building owner for the renewable energy the building exports to the grid is at least equal to the amount the owner pays the utility for the energy services and energy used over the year.
- **Net Zero Site Energy** - A site NZEB produces at least as much energy as it uses in a year when accounted for at the site.
- **Net Zero Emissions** - A net-zero emissions building produces at least as much emissions-free renewable energy as it uses from emissions-producing energy sources.

According to the four definitions as highlighted above, it showed that NZEBs is a concept that can represent buildings that is a grid-connected, where the buildings able to produces energy from various types of renewable sources and the building's energy consumption are equal with its energy production during the period of one year. This view is supported by Sartori et al. (2010) that highlighted NZEBs is theoretically a building with a lower of energy demand and are equal by the on-site generation of electricity, or other energy carriers, from renewable sources. In the same vein, another study by Kurnitzki et al. (2011) and Voss et al. (2011) emphasized that NZEBs are characteristically a grid-connected building with very high energy performance. Thus, it showed that there is a large volume of previous studies describing the role of renewable sources toward NZEBs and resulting a lower of energy demand in buildings.

Concerning that the pathway to a NZEB is given by the balance of energy efficiency and energy supply, hence the criteria on energy efficiency and energy supply are important to the definition framework (Sartori et al., 2012). As illustrated in Figure 4, the graph displayed that the NZEBs underlines the priority of energy efficiency as the path to success. Similarly, Buildings Performance Institute Europe (BPIE) in its reports entitled "Implementing Nearly Zero-Energy Buildings (NZEB) In Poland - Towards A Definition And Roadmap", BPIE specified that the government of Poland has to support the energy efficiency measure in order to successfully implementing NZEBs (BPIE, 2015). Hence, it shows that energy efficiency measure has a significant part in NZEBs and this measure should not be overlooked.

To conclude, based on the developed NZEBs definition framework, it is understood that NZEBs represent buildings that are connected to the electricity grid and the total usage of energy is equal to the total 'onsite' generation of energy from a renewable source over the course of a year. Besides that, NZEBs are buildings with high energy performance and are optimally efficient. Other than that, it is important to realise that energy efficiency plays an important role towards NZEBs implementation success.

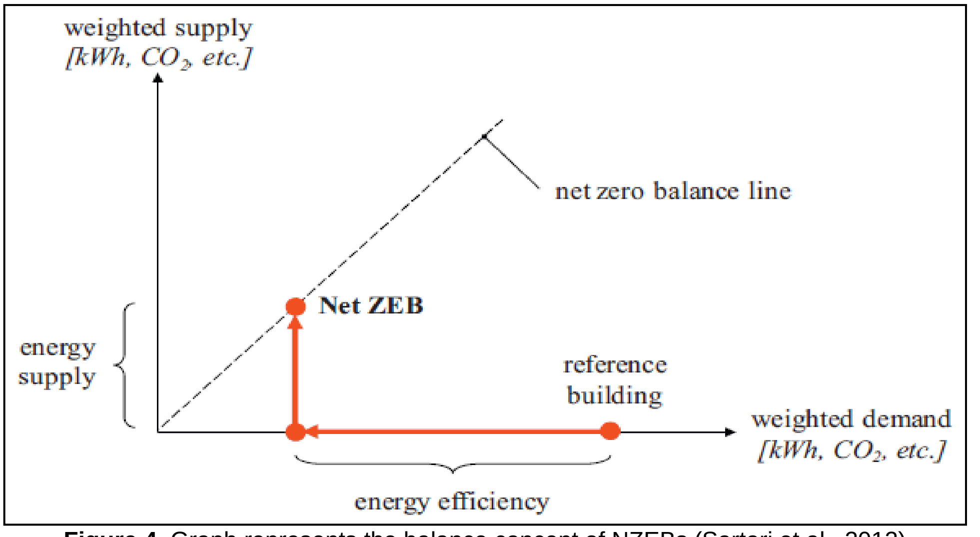

Figure 4. Graph represents the balance concept of NZEBs (Sartori et al., 2012)

EXISTING REVIEWS ON NZEBS CONCEPTS

Since 2008, the participation of several countries through an international collaborative research initiative IEA SHC Task 40 / ECBCS Annex 52 "Towards Net Zero Solar Energy Building" has been working to establish an internationally agreed understanding on NZEBs and international definitions framework (Marszal et al., 2011). Even though it is a challenging task to determine if a building is truly NZEB, but Buildings Performance Institute Europe (BPIE) (2011), in the study towards a successful implementation of nearly Zero-Energy Buildings introduced that there are three principles and approaches for implementing NZEBs as illustrated in Figure 5, which are the first principle of energy demand, second principle for renewable energy share and third principle for primary energy and CO_2 emissions.

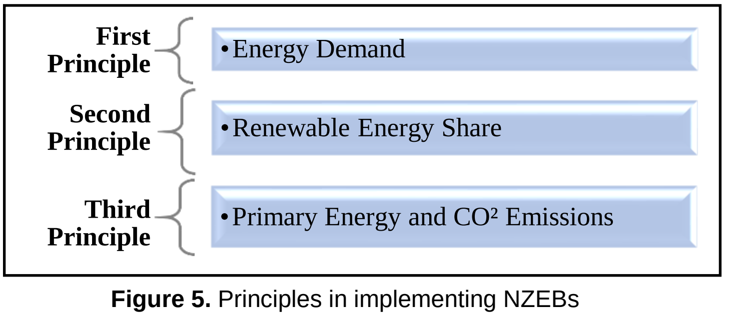

Figure 5. Principles in implementing NZEBs

For the first principle which is energy demand, it is essential to be realised that there are a variety of ways to approach energy demand reduction with much different of academic disciplines (Sorrell, 2015). Furthermore, socio-economic aspects mostly dominated the energy demand side (Schmidt & Weigt, 2013). Therefore, it is important that the boundary in the energy flow related to the operation of the building which consists of energy demand quality and method to assess corresponding value should be a clearly defined (BPIE, 2011). According to Schmidt & Weigt (2013), several aspects such as efficiency of markets, the performance of business and design of products, including choices, behaviors, practices, and lifestyles of consumers need to be considered in order to achieve a change on the demand side. Besides that, building's energy demand involving the sum of useful heat, cold and electricity needed for space cooling, space heating, domestic hot water, and lighting should also include the distribution and storage losses within the building (BPIE, 2011).

For the second principle which is renewable energy share, the application of renewable energy technologies has a lot of benefits and potentials in providing energy services with reducing the emissions of both air pollutants and greenhouse gases as well as protecting the environment. According to Panwar et al. (2010) in the study of renewable energy gadgets to meet out energy needs and mitigation potential of greenhouse gases, the authors highlighted that the development of renewable energy system potentially to resolve the presently most critical tasks such as improving the reliability of energy supply, solving problems that relates to the local energy and water supply, improving the standard of living as well as increasing the level of employment of the local population, and implementation of the obligations of the countries with regard to fulfilling the international mutually agreements relating to environmental protection.

Therefore, BPIE (2011) stated it is important that the boundary in the energy flow related to the operation of the building where the calculation and measurement of renewable energy share are clear and method to assess this share including the sum of energy needs and system losses should be clearly defined. For instance, the application of hybrid power generation system which involves the combination of two energy resources which are the wind and solar energy is an effective solution to generate energy and this hybrid power generation system has better efficiency for energy generation than the conventional energy resources (Ingole & Rakhonde, 2015).

For the third principle, that is primary energy and CO_2 emissions are crucial to being considered in the implementation of NZEBs. According to Zavadskas et al. (2017), primary energy can be generally defined as the total energy consumption in the form of natural resources to produce final energy in the form of electricity or district heating. Furthermore, the consideration of primary energy use is important, as it will reflect the NZEBs concept. For instance, 13 Member States in the European Union have included a numerical target of primary energy use to apply with NZEBs definition in practice (Boermans et al., 2014).

In addition, the role of primary energy as a performance indicator is significant as in the Directive 2010/31/EU on the energy performance of buildings (EPBD), it is stated that the energy performance of a building shall include an indicator based on primary energy factors (PEFs) (Recast, 2010). Moreover, Bruni & Sarto et al. (2013) pointed out that the use of primary energy as performance indicator able to provide a measure of CO_2 emissions besides energy consumption, and the authors added that the current Italian building regulations use primary energy as the indicator.

Therefore, it is important that the principles of NZEBs are clear and method to assess should be clearly defined. Besides that, clear national rules should be available on how to account for the net export if the generation of renewable energy is greater than the energy consumptions during a balance period.

RESEARCH METHODOLOGY

The objective of this section is to explain the methods that were used for this paper comprised of research design and interview data collection procedures. This paper reviewed an existing literature relating to NZEBs in order to gain an understanding as well as insight into previous studies. To ensure the quality of this paper. A literature review is important to provide insight into ways in which the researcher can limit the scope to a needed area of inquiry (Creswell, 2013). Besides that, the semi-structured interviews were conducted with architects in order to explore their understanding on NZEBs as well as to explore the sustainable practices in the context of Malaysian construction industry. The summary data gathering process is illustrated in Figure 5.

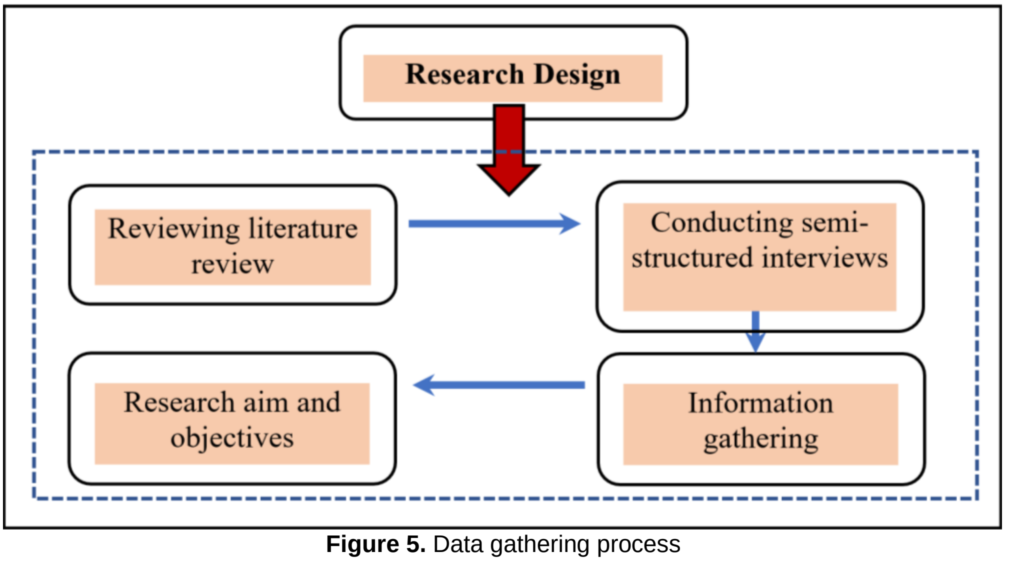

Figure 5. Data gathering process

INTERVIEW DATA COLLECTION PROCEDURE

A semi-structured interview was chosen for this study since there is a limited literature on the NZEBs practices or sustainable practices for designing healthcare buildings in Malaysia context. Thus, these interviews were conducted to focus on getting an understanding of this aspect. In addition, despite simply rely on studies from other countries, the interview sessions that have been conducted are important to ensure that all sustainable practices in Malaysia are captured. The interviews sessions are important since Malaysia's culture differs than the other countries. The difference in terms of locality aspect, cultural aspect or governance aspect should be taken into consideration. Hence, by directly asking the architects who are involved in Malaysian construction industry, sustainable practices that may only apply to Malaysia context can be identified.

Moreover, the "face-to-face" method of interviews will be undertaken for this study. The selections of places for interviews was decided by interviewees, and each of interview took approximately 30 to 60 minutes to complete. All sessions are recorded for the purpose of transcription as well to ensure that the data is accurate. Besides that, all information that has been obtained from the interview sessions are screened and compared with the literature review in order to find similarities and disparities.

Furthermore, the authors used the concept of saturation in order to determine the suitable sample size for the interviews. This decision was based upon recommendations by Mason (2010) to avoid ending up with repetitive and unessential information which does not contribute to the research. Besides that, the amounts of information obtainable from interviews do not necessarily increase just because more data was obtained (Mason, 2010) and the authors further claimed that researchers generally use saturation as a guiding principle during their data collection towards determining appropriate sample sizes. Therefore, the authors decided to choose 3 interviewees to be interviewed for this paper. The interviewees were assigned codes as R1, R2, and R3. Besides that, to ensure the quality of the interviews, all respondents required to be active in Malaysian construction industry and had direct involvement in the buildings projects located within Malaysia. The key profiles for the interviewees are shown in Table 1.

Table 1. Key profiles for interviewees

Item	R1	R2	R3
Position	Architect	Architect	Architect
Organization	Consultant	Consultant	Developer / Client
Experience	More than 5 years	More than 5 years	More than 5 years

As for the interviews, the objective of this paper will be involved to obtain the architect's opinions on design and practices of NZEBs based on the Malaysian construction industry context as well as to identify the benefits of NZEBs.

FINDINGS AND DISCUSSION

In order to identify the benefits of NZEBs, the practices that support this concept are crucial to being identified. Based on Leach et al. (2014) there is a growing interest in NZEBs as many NZEBs demonstration projects showed that the goal of this concept is achievable. However, the need to identify NZEBs approaches is crucial in order to enable broad replication of NZEBs, especially in a healthcare building.

Furthermore, all interviewees indicate that there are various of ways to achieve sustainability in construction since construction sustainability is too wide and different architect will have its own different of sustainable practices, but then all practices will be based on the same principles which are to increase the dependent on renewable sources as well as to protect the environment (R1, R2, R3). Similarly, according to Aelenei et al. (2013) in the study on close inspection of the relevant design strategies and relative performance indicators of the eight case studies, the authors stated that there is no standard approach for designing NZEBs. However, Habash et al. (2014) highlighted that it is important for architects or engineers to have specific design guidelines and strategies in order to popularize NZEBs.

Even though there are still no specific design strategies in achieving NZEBs design, but based on Lu et al. (2015) there are several common design elements and some consensus in designing NZEBs. The design elements that support NZEBs is important to be identified since the adaptation of NZEBs in buildings will ensuring the high performance in buildings can be achieved as well as to support the use of renewable energy for buildings.

Generally, according to Aelenei et al. (2012) and Lu et al. (2015), there are three main steps of a design approach that can be applied in designing NZEBs which are the first design approach is a passive approach that focused on reducing energy demand. The second design approach is the use energy efficiency system, and the third design approach is renewable energy system which is needed to generate renewable energy as well as to offset in large measure the energy demand. The combination of this three-design approach is also crucial to be taken in designing healthcare buildings since these approaches able to succeed in reaching the desired energy performance as well as NZEBs target.

Passive approaches are one of the key element when designing NZEBs as this approach will affect directly to the energy needs for the buildings mechanical and electrical systems, and also indirectly affect the renewable energy generation (Aleinei et al., 2013). Similarly, Scognamiglio et al. (2014) also highlighted that the passive approach plays a significant part in the NZEBs design. Besides that, the improvement of energy consumption of a building can be achieved through passive design strategies as buildings are constructed that react to the environment, hence it is possible for a building to achieve high environmental quality (Hootman, 2012).

Furthermore, Habash et al. (2014) stated that architects have an important role in controlling aspects of a passive design when designing NZEBs. Similarly, some of the interviewees have highlighted that an architect is responsible to ensure some elements of passive approach should be carried out in order to support the sustainability of construction. When designing a healthcare building, an architect has to follow the standard specification and requirements that have been set by the government. However, an architect can propose some sustainable materials to be applied when designing the healthcare buildings (R1). For design and build projects, an architect is encouraged to apply some sustainable elements into passive approaches such as materials selections and a number of ventilation for the buildings. (R2).

One of the most important aspects of passive design strategies is maximizing the amount of natural daylight into a building and reduce the amount of energy requires for artificial lighting in a building (Aksamija, 2015). This strategy is important for designing buildings, especially healthcare buildings since these types of buildings is operating continuously 24 hours a day and required a huge amount of artificial light to provide services to patients. Besides that, it is important to control passive design strategies in a building as this approach will influence the energy performance of a building such as natural lighting, heat gain, shading, and envelope conduction. (Habash et al., 2014; Omrany and Marsono, 2015). In addition, Aleinei et al. (2013) stated that the passive design strategies in buildings should be appropriately orientating building towards solar heating maximization when buildings are dealing with heating challenges.

Moreover, according to Thalfeldt et al. (2013) when designing NZEBs, façades have a strong impact on heating, cooling and artificial light in the building needs as well as on daylight. Thus, it is important to realise that the passive heating solutions are essential to be studied along with passive cooling solutions in order to prevent overheating in a building (Aleinei et al., 2013). Therefore, it is important to consider passive approach while designing NZEBs as it will enable maximisation of natural daylight and the amount need of artificial light in the building needs will be reduced, hence it will create a pleasant environment.

Besides that, the energy required for heating, cooling and lighting the healthcare buildings can be reduced through passive approach, thus this design approachable to support the principles of NZEBs and the benefits that can be gained from NZEBs is where the energy demand for buildings can be reduced.

In regards to second design approaches which is the utilisations of energy efficiency system, it is important to realise that the pathway towards NZEBs is reducing energy demand by means of energy efficiency measures (Sartori et al., 2010). Besides that, energy efficiency has a significant part to act as a parameter that indicates the reduction level of energy consumption in carrying out a related task (Parameshwaran et al., 2012). Generally, energy efficiency can be improved by several design strategies including airtightness to avoid infiltration and mechanical ventilation systems with heat recovery to provide air conditioning and indoor air quality (IAQ) (Barbolini et al., 2017). However, the selection of high-efficiency technologies is also crucial in the improvement of energy efficiency in a building. One of the interviewees has indicated the importance of energy efficiency system in healthcare buildings.

As an architect, we have emphasized the importance of energy efficiency measure for this healthcare building since this measure is much easier to be applied rather than the passive approaches and these measures do not relates to any hygienic condition requirements (R1). There are various energy efficiency technologies that can be applied in buildings. For instance, the use of low power lighting, energy efficient electrical equipment such as washing machines and dishwashers with a warm water connection are strategies in planning an equated energy balance (Musall & Voss, 2012). Similarly, based on Maassen (2017), healthcare building's energy demand can be significantly reduced by considering energy efficiency as a factor in buying medical equipment, besides the proper use of electricity.

Moreover, in relation to the availability of energy efficiency technologies in Malaysia, there are no problems to obtain the technologies since Malaysia is one of developing countries (R2, R3). However, the cost of this technologies is quite high and for that reason, most of the developers are not fully utilised the application of energy efficiency technologies in buildings (R3). Similarly, based on Vogel et al. (2015) which investigated barriers that prevent energy efficiency adoption in multifamily buildings in Sweden and authors proposed a novel categorization framework for barriers to energy efficiency in buildings.

The results showed that barriers related to contextual level are the most significant to prevent energy efficiency adoption. The contextual level is characterized by the rules and regulations that influence technological design and market development. In addition, as stated by interviewees R2, The local authorities will reject the proposal of purchasing some energy efficiency technologies form the architect if the cost is too high (R2). Thus, based on the findings and the arguments, energy efficiency measure in a building is vital towards designing a healthcare that achieved NZEBs since energy reduction, as well as energy savings in buildings, can be achieved. Moreover, by improving the energy efficiency of the various incorporated building systems, it will help to reduce building´s energy demand (Sartori et al., 2012). It shows that NZEBs can provide a reduction in both primary energy as well as CO_2 emissions.

For the third design approaches which are renewable energy system, the necessity of renewable energy systems is vital towards NZEBs since these systems are required to reduce and as well as to offset the thermal energy need in buildings. Moreover, designing NZEBs is not only achieved by reducing the energy consumption of the building with passive design methods or the application of energy efficient system in buildings, but NZEBs should also be designed with the balances energy requirements between active energy production techniques and renewable technologies in order to achieve the NZEBs objective (Kolokotsa et al., 2011).

For designing a healthcare building, it is important to consider renewable technologies as part of the building since it able to reduce the dependency of non-renewable sources especially past literature had shown that the energy consumption of healthcare buildings is high. Besides that, the utilization of renewable energy systems in buildings can be a critical need and solutions against global warming and environmental pollution (Koo et al., 2013).

Furthermore, most of the countries have already promoted the application of renewable energy systems in buildings through enforcement, regulation, target, incentive, and fund provided in the countries (Winkel et al., 2011). However, in the context of Malaysia, the interviewees agreed that there is a lack of enforcement and target set by the government even though there is some government efforts to promote renewable energy systems in Malaysia (R1, R2, R3). One of the interviewees has highlighted that, The government has provided some incentives to promote the green concept in construction such as tax exemption, however, the enforcement is not executed strictly and thus, the utilisation of renewable technologies is not widely used (R2).

With respect to renewable energy systems, there are a lot of benefits that can be gain from these systems and several studies have demonstrated the benefits of these systems. For instance, Phuangpornpitak & Tia (2013) studying the integration of renewable energy in a smart grid system and they concluded that renewable energy system is one of incomparable option to generate energy since these systems able to provide a clean energy resource. However, they suggested that some issues such as the design, sizing, and the suitability of the system in terms cost for energy generation need to be addressed in order to ensure that it can be used thoroughly and commercially.

As a result, since renewable energy system is crucial towards NZEBs and there is a lot of benefits that can be gained through these systems, this design approach should be applied appropriately when designing a healthcare building. Besides that, policies and guidelines that have been provided by the government should be along with enforcement in order to ensure that all government's efforts and targets can be achieved. Therefore, through the application of renewable energy technologies to support NZEBs, it indicates that this concept can reduce the dependency on non-renewable energy and thus, a greener environment can be achieved. The summary of NZEBs practices and benefits are shown in Table 2.

Table 2. Summary of NZEBs practices and benefits

NZEBs practices	Literature	Key points from interviewees	NZEBs benefits
Passive approaches	Ferrara et al. (2015); Ascione et al. (2016b); Becchio et al. (2015b); Rodriguez-Ubinas et al. (2014); Barbolini et al. (2017)	"We have taken into consideration on passive elements before preparing architecture plan." (R1) "…it is undeniable that passive design elements plays a significant part in architecture." (R3)	Reducing energy demand Reducing primary energy and CO_2 emissions

NZEBs practices	Literature	Key points from interviewees	NZEBs benefits
Energy efficiency technologies	Gaitani et al. (2015); Dalla et al. (2015); Buonomano et al. (2015); Ascione et al. (2016a)	"As an architect, we can propose to developer some energy efficiency system for buildings." (R1) "If we don't have any cost constraints, we can widely use energy efficiency in buildings to save electricity." (R2)	Reducing primary energy and CO_2 emissions
Renewable energy systems	Kurnitski et al. (2011); Becchio et al. (2015a); Tian et al. (2015): Mohamed et al. (2014); Ajla Aksamija (2015)	"…even though the movement is still slow, but we are still put our efforts to increase the utilisation of renewable energy technologies in buildings" (R2)	Increasing renewable energy share Reducing primary energy and CO_2 emissions

CONCLUSION AND RECOMMENDATION

The NZEBs concept is gaining significant since it is a realistic solution for the mitigation of CO_2 emissions. The adoption of this concept into Malaysian healthcare buildings is vital because of the energy consumption for this buildings is high and this concept able to support the government initiative in Construction Industry Transformation Programme (CITP) 2016-2020 which one of strategic thrust that seeks to transform the industry is environmental sustainability. However, NZEBs concept in Malaysian construction industry is still considered new and it needs to be explored. But, the previous studies on NZEBs able to give an insight into design and practice of this concept and thus, it gives the understanding to be explored based on Malaysia context. This paper is comprised of a comprehensive existing literature review and through semi-structured interviews with architects in Malaysia that aims to explore the architect's opinions on benefits of NZEBs concept through the design and practices of NZEBs in Malaysian construction industry context. The findings showed that by the adaptation of NZEBs concept in Malaysia healthcare buildings, benefits that can be gained through this concept are it helps to reduce energy demand for healthcare buildings, reduce primary energy and CO_2 emissions for healthcare buildings, and increase the renewable energy share for healthcare buildings. However, the authors recommend that by increasing the number of sample size, it will produce any significantly different findings. Besides that, it is also recommended that this paper need to further investigate in order to get more comprehensive findings as well as reflect more on the Malaysia context. Further, this paper provides some sustainable design and practices as basis for further investigation.

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A CRITICAL REVIEW OF MANAGEMENT OF BUILDING FIRE SAFETY IN MALAYSIAN PUBLIC UNIVERSITIES

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Abstract
Fire Safety Management (FSM) is essential in ensuring the safety of building occupants from the fire as well as property protection. Recently, there had been a reduction in annual budgetary allocations to public universities in Malaysia due to some economic tensions. This situation had left many institutions in question with the options of scaling down their expenses as well as sourcing for other means of meeting up with the shortfalls in allocated funds. Hence, it affects the sustainability of the building itself. This paper aims at examining the possibility of reducing incidents that could lead to expending unbudgeted resource to rehabilitating property destroyed by fire on campus, in addition to limiting risk to life and interruption of academic and business activities. Several types of research had been conducted on FSM, nevertheless very few consider Higher Education Institutions (HEI)s holistically. This study was carried out through a comprehensive review of the literature on FSM within the context of Malaysian public universities and sustainability from relevant databases. The paper reveals that FSM is critical to preventing fire disaster in buildings. It identifies the need for an investigation into the implementation of FSM to developing effective FSM framework for assisting facilities managers and other stakeholders to preserve the university buildings against fire risks. Implementation of effective FSM is beneficial in several ways including reduction of property insurance premiums, business continuity and promoting an efficient and sustainable work environment.

Keywords*: Fire safety; FSM; Malaysian public universities; Sustainability*

INTRODUCTION

Buildings are among the most important investments for any Higher Educational Institution (HEI) which serve as meeting point for students, teachers and the communities (Nadzim and Taib, 2014). Malaysian government and private organisations made huge investments running into billions of Ringgits in constructing these structures and the facilities within them. The number of HEIs including Universities and Colleges publicly and privately funded is evident for the expenditure (StudyMalaysia, 2015). For instance, located in Malaysia are over 500 tertiary institutions comprising 20 public universities, over 70 private universities, 33 polytechnics, 72 community colleges, 27 teachers' education institutes and about 403 colleges (StudyMalaysia, 2015; Ahmad, 2013). HEIs provide students and the entire institutions' community with an environment that is attractive, conducive to learning and academic success (Hassanain, 2008). It is therefore essential to adequately protect buildings from fire destruction, as well as the life of the users through appropriate Fire safety management (FSM) implementation (Nadzim and Taib, 2014). FSM encompasses the combination of or coordination of some activities or programs to avert destruction from fire (Nadzim and Taib, 2014). Such agendas include escape routes provisions and maintenance, fire prevention measures, staff training, and fire drill training, etc. Howard and Kara-Zaitri, (1999) described FSM as "the application by a manager of policy, standards, tools, information, and practices to the task of analysing, evaluating and controlling fire safety." Pickard (1994) asserted that fire safety strategy for a specific building requires management policies and procedures for the plan to efficiently function. Fire safety strategy should be a continuing process such that fire safety systems are regularly checked and maintained. Usually, fire takes place without warning allowing building occupants limited time to either react to escape or suppress the fire (Salleh and Ahmad, 2009).

Effective FSM requires identifying all the potential risk associated with the premises and efficiently undertake an assessment of the adequacy of the measures provided or needed to resist the threat (The Scottish Government's Police, Community Safety Directorate, Her Majesty Fire Service Inspectorate for Scotland, 2008). Also, the concept of sustainable development was firmly embedded within the environmental movement but still has social and economic values in sustainable use of natural resources. It implies that sustainability is a vast area covering topics from economics to social and the issues relating to ecological sustainability. While all business enterprises can make contributions towards its sustainability attainment, the ability to make a difference varies by sector and organisation size where the role of business in contributing to sustainable development remains unlimited (Mohamed and Wee, 2016; Mohamed et al., 2016). Given that, sustainable development is a decent business in itself; it creates chances for suppliers of 'green consumers,' as well as developers of environmentally safer materials and processes. Hence, firms that invest in 'green' are eco-efficiency and engage themselves in social well-being.

BACKGROUND OF STUDY

FSM is concerned with lessening the potential of harm to life and damage to properties resulting from the occurrence of fire in buildings (Canadian Wood Council, 2000; Sanni-Anibire and Hassanain, 2015). The importance of FSM in relation to modern large or complex structures has been identified for several years, though, the challenges are yet to be thoroughly addressed. The growing complexity of buildings, as well as sophistication of the fire safety systems, places more responsibility on management that has been traditionally essential (Porter, 1990). Protecting buildings from the fire can be tackled from two perspectives. Namely Building design, and Building operation /management (Ramli et al., 2013). Although fire safety priority is very low during the building design process, it is included to satisfy minimum requirements for building regulations and recommendation of the insurance company (Mydin, 2013). Therefore, if an efficient management team is put in place to operate a property, such property would be safe from the fire even if poorly designed. However, a well-designed building that is not run by efficient management team may likely have issues with fire safety during the building lifecycle. Many of the major disasters including fires which happened in recent decades were characterised by a failure of management, whether before or during those incidences. An example was the 1987 fire at King's Cross underground station, London. Consequently, considerable emphasis has been placed on the importance of management in safety as well as focusing on both corporate and individual liability in this respect (Porter, 1990). Apart from that, sustainable development is a process of change in which exploitation of resources, the investments direction, the orientation of technological developments and institutional are all in agreement and enhance the present and prospects of meeting human needs and aspirations. It is reflected on many Governments around the world when they take on sustainable development on the agenda for the development project (Masrom et al. 2017; Mohamed et al., 2017). Organisations around the globe that incorporate sustainable practices to strengthen their organisational goals can increase shareholders' values and build better international market share.

PROBLEM STATEMENT

Campus Firewatch (2011) stated that no fewer than 146 people perished in campus fire in the United States between 2000 and 2011. These comprised of both on-campus and off-campus fire incidences. Several injuries were sustained and property loss worth $9 million were reported (United States Fire Administration, 2011). The impact of fire fatality on HEIs campuses could be severe due to the homogenous nature of students' population concerning age and experience (Taub and Servaty-Seib, 2008). Furthermore, substantial fire loads such as books, papers, and another document in lecturer's offices in High Education Institutions (HEI)s could contribute significantly to fire severity (Kong, 2009).

Research had shown that some fire incidence occurs in tertiary institutions' buildings (Mowrer, 1999). The cost of the incidents to the HEIs from both financial and public image perspectives could be substantial (Bubka, Mary Ann, and Coderre, 2010). These incidents take place especially among students in hostels while they are attending classes or during siesta (Hassanain, 2008). Numerous fire incidents mainly of small-scale occur on campuses without being reported. Report from the United States Fire Administration (USFA) indicated that an average 1700 fires were recorded yearly in the United States of America (USA) (United States Fire Administration, 2001). The abovementioned figure increased by over 100% in the USFA's report between 2007 and 2009 as 3,800 fires were recounted in university houses in 2009. The campus fires usually occur during September and October when schools are starting a fresh academic session which leads to estimated fatalities of five people annually (United States Fire Administration, 2011).

In the United Kingdom, two campus fire incidents were recorded in 2001 (Times Higher Education, 2001a, 2001b). The first one occurred in the main campus of the City University London and destroyed College Building on 25 of May. The result of the fire investigation exposed that the fire started from an office of a member of staff. The affected building was grade II listed building accommodating five academic departments, the school of journalism and computing as well as the vice-chancellor's office. The second incident took place on November 2, 2001. The fire attacked a 100 years old Bower Building at the Glasgow University, Scotland destroying Ph.D. work of ten Botany students. It took two years to carry out restoration work on the damaged building at the cost 6.5 pound and GBP 3.5 million for the equipment. The building was opened for use in 2005 (University of Glasgow, 2017; Times Higher Education, 2001b). Furthermore, on September 12, 2014, the GlaxoSmithKline Carbon Neutral Laboratory for Sustainable Chemistry at the Nottingham University, UK was destroyed by fire attributed to an electrical fault. The GBP 20 million structure which was at the advanced stage of completion was razed entirely down (British Broadcasting Corporation (BBC), 2015; Nottinghamshire and City of Nottingham Fire and Rescue Authority Community Safety Committee, 2015). Other examples of a fire incident in HEIs include a clubhouse fire at Nelson Mandela Metropolitan University South Africa in October 2016, the University of Jos Nigeria Library fire on October 10, 2016. The hostel fire at International Islamic University of Malaysia (IIUM), 2014, Dewan Tunku Canselor (DTC), of Universiti Malaya (UM) among others are more examples (Ali, 2003; Nottinghamshire and City of Nottingham Fire and Rescue Authority Community Safety Committee, 2015; Sadiq, 2016;

Spies, 2016). The need for HEIs to protect students, employees, and physical facilities cannot be overstated, because if a disastrous loss ensues, media coverage may affect the institutions' reputation, posing a risk to future admissions, financial strengths, and endowments (Bubka, Mary Ann and Coderre, 2010).

Report from the Fire and Rescue Department of Malaysia (FRDM) indicated a steady increase in fire incidents from 2000 to 2015 (Sulaiman, 2008; Salleh and Ahmad, 2009; Rahim, 2015). For instance, in 2006 FRDM attended to 18,913 fire calls, and 20,225 in 2007. Fire occurrences in 2012 were 29,848 which was 11% higher than the relatively stable statistics from 2009 to 2012. From 2013 to 2015, fire incidents recorded were 33,640 for 2013, 54,517 for 2014 and 80,183 for 2015 (Salleh and Ahmad, 2009; Rahim, 2015). Rahim (2015) also conveyed that fire-related death doubled between 2011 to 2012 that is, from 72 deaths in 2011 to 152 in 2012 and then to 165 in 2013. The cost implication of damages from a fire in 2013 is about 20 billion Malaysia Ringgit (MYR 20 billion), which was an increase of MYR 874.31 million from 2012 (Department of Statistic Malaysia, 2013; Fire and Rescue Department Malaysia, 2014). The main causes of fire include arson, cooking, smoking, electrical sources, and other unknown sources, to this Rahim (2015) called for more investigative efforts to obtain useful information for developing precautionary strategies in Malaysia.

No doubt enormous investments made in the education especially in the higher education sector is in line with fulfilling a set target of creating an attractive environment, conducive for learning and academic excellence (Hassanain, 2008). It is, therefore, duty bound on all stakeholders to zealously guard various infrastructure against fire destruction. In accordance with building world-class higher education institutions, Malaysian government spent more on education compared to other sectors, for instance, Musa (2003) reported that about 27% of the National budget was spent on education. Such gesture allows for training of human resources that shall tackle future challenges in various sectors of the economy (Kaur et al., 2008). However, in recent time, the fund allocation to public universities is dwindling due to some challenging economic situation (Else, 2016; Malay Mail, 2016). Therefore, necessitating paying considerable attention to protecting buildings from all form of disasters including fire.

Figure 1 illustrates the aim and objectives of the research from which this review paper was derived.

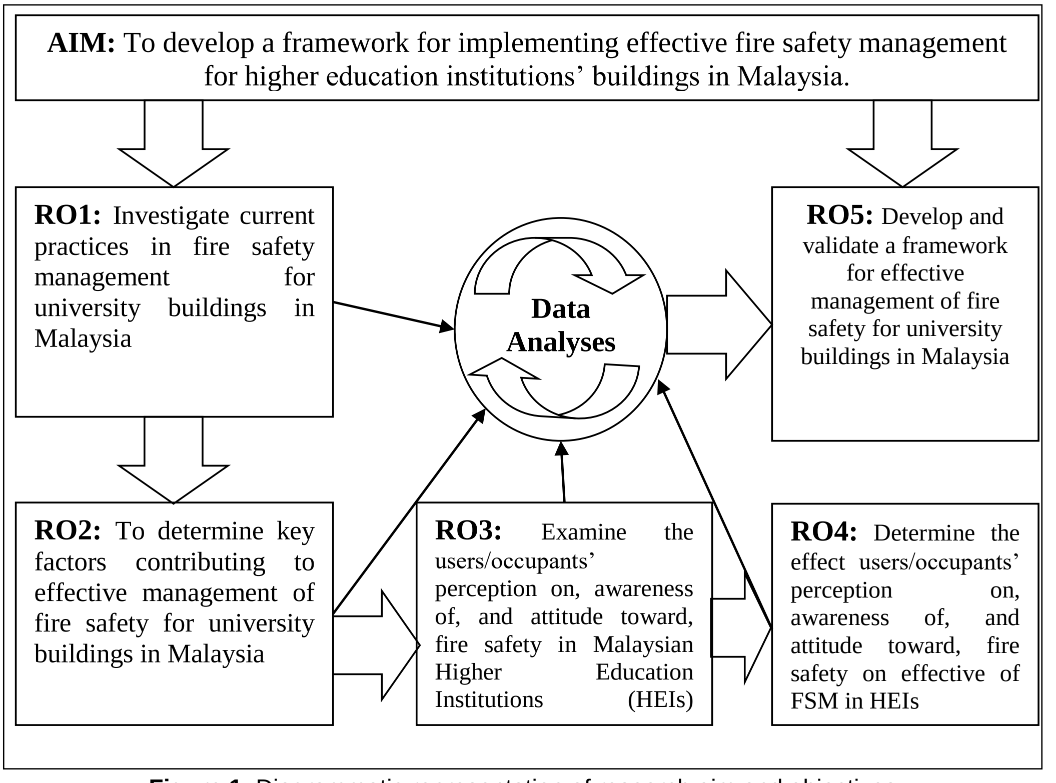

Figure 1. Diagrammatic representation of research aim and objectives

DISCUSSION

The substantial effort had also been made at internationalising Malaysian higher education with a resultant saving of about RM4 billion annually. RM2.5billion is the amount that could be saved if Malaysian students decide not to study abroad and take advantage of provisions made in Malaysian universities and other higher educations, whereas RM1.5 billion in net revenue realised from over 50000 students from various countries (Kaur et al. 2008). Malaysia had also set the target of becoming by 2020, the world's sixth-biggest education exporting country, and therefore making plans to accommodating about 200,000 international students (Hughes, 2015). This projection doubled the aspiration in the 9th Malaysia plan for attracting 100,000 students from overseas by 2010 (Kaur et al. 2008). Given these strides, research relating to the managing fire safety in Malaysian higher education's buildings becomes imperative especially as no structure has total immunity against fire occurrences (Ramachandran, 1999; Rahim, 2015). Previous research concerning management of fire safety in buildings had dealt with other types of building/structure such as karaoke establishment (Lui & Chow, 2000), oil and gas (Santos-reyes and Beard, 2001), non-residential high-rise buildings (Chow, 2002; Tharmarajan, 2007), residential high-rise buildings (Yatim, 2009), Nursing homes (Moore, 2012), Hotel buildings (Chen et al. 2012), Heritage buildings (Salleh and Ahmad, 2009), Passenger terminals Howard and Kara-Zaitri, 1999) and airport terminals (Chow and Ng, 2003), enclosed shopping centres (Freeman, 2010), Cruise Vessel Construction (Räisänen, 2014) and hospital (Ong and Suleiman, 2015). However, few of these studies had explicitly looked at tertiary education buildings holistically. Though Kong (2009) studied possible implementation of performance-based design for fire safety provisions in higher education institutes, He did not emphasize the strategies for fire safety management adopted. In addition, there are research covering students' housing (Griffin, 2011; Hassanain, 2008; Sanni-Anibire and Hassanain, 2015; Agyekum et al., 2016) and other publications dealing with cafeteria (Hassanain and Hafeez, 2005) and library (Khalid, 2013; Hassanain and Ashwal, 2005). This research seeks to investigate the implementation of fire safety management implementation in Malaysian Higher Educational Institutions (HEI) to developing a framework for effective fire safety management for all building types and occupancies in the institutions.

Statistical surveys in different parts of the world established that fires occur regularly in buildings with costly consequences (Chen et al. 2013; Sanni-Anibire and Hassanain, 2015). In the United Kingdom (UK), the direct cost of the fire was estimated at 8 billion GBP in 2003. Also, the Health And Safety Commission (HSC) approximated the annual value of health and safety failure as 18billion GBP (Furness and Muckett, 2007). The United States Fire Administration reported that about 118 deaths related to campus fire had been witnessed from the year 2000 to 2015 in the United States of America (USA). 80% of those incidents occurred in off-campus housing due to the following factors (U.S. Fire Administration, 2015):

- Careless disposal of smoking materials;
- Missing or disabled smoke alarms;
- Lack of automatic fire sprinklers; and
- Alcohol consumption.

According to the guidance jointly produced by the Scottish Government's Police and Community Safety Directorate, HM Fire Service Inspectorate for Scotland (2008), the obligation of an organisation's management to fire safety is essential to achieving suitable fire safety standards in buildings and in upholding staff's culture concerning fire safety. Dublin Fire Brigade (2012) consider FSM as a critical factor which must be present to forestall fire disaster in buildings such as large shopping malls. According to Dublin Fire Brigade (2012), and the British Standards Institution (2017) FSM structure should make provisions for the following:

- Clear lines of duty, authority, accountability, and resources;
- Replacements of absentees with specific tasks; and
- An emergency services liaison officers to call for supplying information to the fire and rescue services.

FSM encompasses the whole lifecycle of the building and includes the following constituents (Dublin Fire Brigade, 2012):

- Day to day operation of the building;
- Changes of building;
- Change of use; and
- Unit in disuse.

Similarly, Tsui and Chow (2004) outlined the objectives of FSM components to include: Maintenance of fire safety measures and fire prevention, staff training, emergency action plan, and an assessment on building alternative. FSM programmes consist of inspection, education, and training, fire suppression, emergency service, evaluation of fire probability, fire prevention, report and record keeping, as well as communication (Malhotra, 1993; Chow, 2001). The following benefit can be accomplished when an effective FSM is developed and implemented (Chow, 2001):

- Reduction of property insurance premiums;
- Continuity of business operation;
- Enhancement of public images and customer service;
- Promote an efficient work environment;
- Quality gain realisation; and
- Influence an organisation profitability.

For the above-stated benefits to be realised, the fire safety managers should come up with his responsibilities as indicated by the Dublin fire brigade, thus:

- Awareness of fire safety features and their functions;
- To monitor general maintenance and building/refurbishment work;
- To maintain register of fire certificates and compliance for all buildings/ refurbishment works;
- Fire safety risk assessment;
- To be present at the commencement of building occupation; and
- To liaise with fire authority and seek advice.

The person appointed as a fire safety manager is responsible for total control and daily building safety management. The appointed fire safety manager should be practically knowledgeable to direct firefighters to affected areas in buildings during emergency especially in complex structures (Chow, 2001). Hence, Fire safety practices and awareness is essential as adequate knowledge of fire, cause, prevention, suppression and the provision of proper firefighting equipment are important to be available and handled adequately by the appointed person in any building.

CONCLUSION

This research seeks to investigate the implementation of fire safety management strategies in Malaysian Higher Educational Institutions (HEI) to having sustainability aspects. The paper highlighted FSM as a critical factor in preventing fire incidents and destructive effects on human and properties. FSM identifies all probable risk associated with the universities built facilities and provide necessary measures to resist the threat. The work also creates a space for research into detail understanding of fire safety management practices, and current implementation strategies of FSM principles among Malaysian universities. Therefore, a framework for effective fire safety management for all building types and occupancies in the institutions shall be developed and validated on completion of the research.

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FEASIBILITY STUDY ON COMPOSITION AND MECHANICAL PROPERTIES OF PALM OIL FUEL ASH AND QUARRY DUST REPLACEMENT IN SOLIDIFICATION/ STABILIZATION MATRICES

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Abstract
This research was conducted to investigate the suitable waste materials to be used in Solidification/Stabilization (S/S) technique. For that reason, palm oil fuel ash and quarry dust have been selected to be used in S/S matrices. These wastes have national issues in recent years due to the increasing amount caused by population and industrialization growth in Malaysia. In this research, different percentages of Palm Oil Fuel Ash (POFA) and quarry dust (0%, 10%, 20%, 30% and 40%) are used to replace the Portland cement and sand respectively. The investigation included determination of heavy metal concentration and chemical composition of POFA and quarry dust. Other than that, the respective samples were also examined for its mechanical properties such as density and compressive strength. The results indicated that 10% of POFA and 20% of quarry dust are the best percentages to be incorporated in S/S matrices. The 10% replacement of POFA recorded a considerable value of density ranging from 1500 kg/m^3 to 1660 kg/m^3.Meanwhile the compressive strength 10% of POFA achieved the strength of 22.60 MPa. Besides that, 20% replacement of quarry dust in natural sand recorded a value of density ranging from 2080 kg/m^3 to 2147 kg/m^3 with the highest strength is 24.91MPa. The results show a possibility of POFA and quarry dust for cement and sand replacements in improving the S/S matrices as a medium to treat wastes.

Keywords: *Solidification/stabilization; Palm oil fuel ash; Quarry dust; Waste utilization*

INTRODUCTION

Solidification/Stabilization (S/S) is a quick and inexpensive technique in treating the waste by converting immobilizing contaminants into a less soluble or a less toxic form and encapsulating them by a creation of a durable matrix with high structural integrity (Navarro-Blasco et al., 2013). S/S is one of the techniques by mixing the wastes with some solid addictive materials like fly ash, cement, lime, cement kiln dust, sulfur, clay and some combinations of them (Wright and Noordhuis, 1991). Apart from that, S/S also undergoes in exchange the waste into easily handled solid with reduced hazards from volatilization, leaching or spillage. This technique used to prevent or minimize the release of hazardous compounds from the finished asphalt road base into the environment by producing a solid mixture, improve handling characteristics, decrease the surface area for contaminant transport and bonded the contaminate into a non-toxic form (Tuncan et al., 2000).

Among various types of binders, cement-based systems are the most widely used, due to its relatively low cost, wide availability, and versatility (Galetakis and Soultana, 2016). The S/S method also offers the possibility to reuse the solidified waste for construction purposes, such as building blocks and concretes (Johnson et al., 2016; Kadir et al., 2014a; Kadir et al., 2016). It also provides minimum energy consumption compared to thermal or other methods. Other advantages provided by the S/S method are the possibility to treat the contaminated waste in-situ rather than ex-situ especially for the massive quantities of petroleum sludge, thus eliminate the transportation cost. Also, only simple equipment and materials are needed for its execution.

The application of binders can increase absorption thus helps in improving their immobilization and prevent the detrimental effects on binder hydration such as combining the use of cement and active carbon (Caldwell et al., 2009). Lampris et al. (2009) found that the Portland cement with the addition of high carbon power plant fly ash (HCFA) significantly reduced the leaching of PHCs. Besides, an advantage of applying S/S techniques is that some hazardous heavy metals in wastes can be immobilized into the matrices.

This paper aim is to evaluate the composition and mechanical properties of palm oil fuel ash and quarry dust in S/S matrices. The present research has been undertaken to investigate the possibility of using palm oil fuel ash and quarry dust as a partial replacement material for cement and natural sand respectively. It is envisaged that the outcomes of this research will enhance the potential use of palm oil fuel ash and quarry dust as a viable alternative material in the production of S/S matrices.

OVERVIEW OF PALM OIL FUEL ASH AND QUARRY DUST

Palm Oil Fuel Ash (POFA) is a by-product of the palm oil industry produced in massive amounts approximately 4 million tons per year (Foo and Hameed, 2009). Recently, POFA is used as a partial replacement for ordinary Portland cement in conventional concrete and the purpose is to enhance strength and durability of concrete (Tangchirapat et al. 2007; Mazenan et al., 2017; Abdul Awal and Hussin 2011; Chindaprasirt et al., 2008, Ismail et al., 2011; Malhotra, 2002). Some works have utilized POFA as supplementary material in mixes with other alumino-silicate materials to make geo-polymer cement paste or mortars. The reaction product, formed according to a poly-condensation process, exhibits a network structure that could evoke that of organic thermoset polymers.

The beneficial effects of POFA as a cement replacement concerning to its mechanical properties and durability have been widely discussed. For example in a study by Ariffin (2013) has stated that low calcium fly ash blended with POFA was used to produce geo-polymer cement and so compressive strength of up to 28 MPa was obtained. Also, a geo-polymer binder was fabricated from a ternary mix of slag, palm oil fuel ash and rice husk ash; however, a small content of POFA was used in the binder (Karim, 2013). In other study presented by Mijarsh et al. (2013), treated POFA was activated to produce the geo-polymer binder. Supplementary materials such as silica fume, calcium hydroxide and alumina hydroxide in addition to the alkaline activator were also used to increase the efficiency of treated POFA in the production of geo-polymer products.

Apart of that, the combination of ultrafine palm oil fuel ash and ground blast furnace slag (GGBS) was used to investigate the compressive strength and microstructure of geo-polymer binder. Islam et al. (2014) also examined strength development for a geo-polymeric binder from ground granulated blast furnace slag and palm oil fuel ash. The study revealed that the binder with the binary mix of the low content of POFA and GGBS achieved the highest compressive strength.

A similar observation was made by Tangchirapat et al. (2007) and Chindaprasirt et al. (2008) who suggested the use of POFA up to 10% of binder mass due to its low pozzolanic reactivity. Hence, to enhance the pozzolanic reactivity POFA must be ground to produce smaller particle size. Chindaprasirt et al. (2008) also in his research found that the compressive strength of POFA-incorporated mortar having a median particle size of 7 µm (3% retained on a 45 µm sieve) to have higher values than control mortar due to the filler effect of finer particle size.

On the other hand, the use of adequate industrial or agricultural by-products such as quarry dust for partial cement replacement is an environmentally friendly method of disposal of large quantities of materials that would otherwise pollute land, water and air. The granite cutting industry produces large amounts of wastes which are solids (generated during extraction) and sludge (generated during transformation processes) (Ho et al. 2002). Granite fines are often referred to as quarry or rock dust, and this residue represents less than 1% of aggregate production or between 1% and 2% by mass of the total aggregate crushed in quarries, according to Abukerh and Fairfield (2011).

A study conducted by Raman et al. (2011) evaluate the suitability of quarry dust as a partial substitute for sand in high-strength concrete (HSC) containing rice husk ash (RHA). The findings of the research indicate that even though quarry dust inclusion as partial replacement of sand results in some minor impacts to the workability of fresh concrete, it can be compensated by a good mix design and by the use of super-plasticizer. The mixes containing 20% quarry dust were chosen as the optimum mix design for both grades of concrete. Incorporation of quarry dust into concrete mixes does improve some workability properties of the mixes.

Medina et al. (2017) assesses the viability of designing new blended cement in which 10% or 20% of the clinker is replaced with quarry waste by analyzing the effect of its inclusion on the chemical, rheological, mechanical and microstructure properties of the end product. The later age change in the pore structure of the new mortars induced by the slow pozzolanic kinetics of this supplementary cementitious material (SCM) translates into a narrow difference in the mechanical performance between these and conventional mortars.

On top of that, the quarry industry produces millions of tones of wastes in the form of quarry dust was increasing yearly (Bashar et al., 2016). These wastes are dumped in the factory yards and hence reuse of quarry dust might help in reducing the overuse of mining and quarrying. Hence, the use of these quarry dust could result in significant cost saving in regions where limestone filler is not available, and it is an efficient use of a material that would otherwise be a waste product (Schankoski et al., 2017).

MATERIALS AND METHODS

The experimental program presented in this paper was done at Universiti Tun Hussein Onn Malaysia and Kolej Kemahiran Tinggi Mara located in Batu Pahat, Johor, Malaysia. The details of the experimental works used are described in the following sections.

Materials

The primary raw materials used in this research were Portland cement type Cem II / B-M 32.5R according to Malaysian standard (MS-EN 197-1, 2014), palm oil fuel ash, quarry dust, river sand and regular tap water. Also, Portland cement in this research is all-general purpose use green label cement. It is manufactured by grinding calcium sulphate as a setting regulator with Portland cement clinker and other carefully selected secondary constituents under careful and stringent quality standards monitored by Lafarge Malaysia.

Palm oil fuel ash (POFA) was collected from Plantation Company located at Kluang, Johor, Malaysia (Figure 1). POFA for this research procured from industry is a waste of oil palm dry biomass which was burnt as a fuel at the temperature of 800 ºC to 1000 ºC. After procuring the ash, it was oven dried at a temperature of 105 ºC for about 24 hours to remove moisture and further, it was sieved by using 90 µm sieve to eliminate unburned fibers and to improve its fineness, as particle size plays a crucial role in pozzolanic reactivity. Hence, to enhance the pozzolanic reactivity POFA must be ground to produce lower particle size as suggested by Kroehong et al. (2011). The process involving palm oil fuel ash is summarized in Figure 2.

Figure 1. Palm oil fuel ash at plantation mill located in Kluang, Malaysia

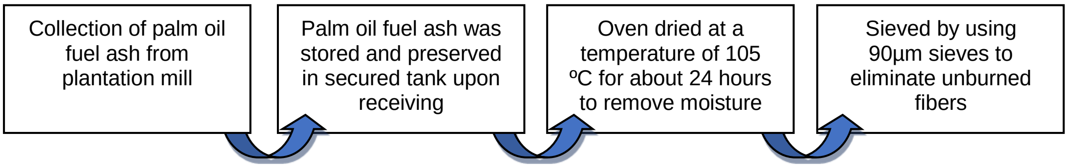

Figure 2. Palm oil fuel ash process before being used

Using quarry waste as a substitute of sand in construction materials would resolve the environmental problems caused by the large-scale depletion of the natural sources of river and mining sands. In addition, quarry waste can be a profitable alternative to the natural sands when the overall construction cost increases due to the transportation of sands from their sources. Quarry dust wastes were obtained from Bina Kuari Sdn Bhd located at Kedah, Malaysia. Upon testing, quarry dust was dry first to make sure it is in a dry condition (Figure 3). The quarry dust waste was dried and placed in the oven at 105ºC. After drying, quarry dust waste was grinded and crushed to make it easier in the sieve process as well as to eliminate impurities.

Figure 3. Quarry dust waste after grinding process

Mix proportion

Two types of S/S matrices were produced in this research, each of matrices were used to study both the effect of palm fuel ash replacement in cement and to investigate the effect of quarry dust replacement in sand. As such, the investigation begins with replacement of palm oil fuel ash (POFA) to cement with 10%, 20%, 30%, and 40% respectively. The S/S mortar was tested in comparison to the control mix that had a ratio of 1:2.75 of cement to fine aggregates. Meanwhile w/c ratio of 0.485 which is in accordance with the ASTM C109 standard for compressive strength testing of hydraulic cement mortars. Next, the same percentages will be used as a sand replacement by using quarry dust which is 10%, 20%, 30% and 40% respectively. Mix proportions are shown in Table 1 and Table 2.

Table 1. Details of mix proportion for palm oil fuel ash replacement

Samples	Percentage of POFA replacement (%)	Binder (kg)		Fine Aggregates (1:2.75 wt binder) (kg)		Water to binder ratio (%)
		Cement	POFA	Sand	Quarry dust	
A (Control)	0	7.893	0	21.706	0	0.485
B	10	7.104	0.789	21.706	0	0.485
C	20	6.314	1.579	21.706	0	0.485
D	30	5.525	2.367	21.706	0	0.485
E	40	4.736	3.157	21.706	0	0.485

Table 2. Details of mix proportion for quarry dust replacement

Samples	Percentage of quarry dust replacement (%)	Binder (kg)		Fine Aggregates (1:2.75 wt binder) (kg)		Water to binder ratio (%)
		Cement	POFA	Sand	Quarry dust	
A (Control)	0	7.893	0	21.706	0	0.485
F	10	7.893	0	19.535	2.171	0.485
G	20	7.893	0	17.365	4.341	0.485
H	30	7.893	0	15.194	6.512	0.485
I	40	7.893	0	13.024	8.682	0.485

During casting process, materials were weighed accordingly to their mix proportions. Firstly, the dry materials were mixed in a rotating pan mixer for 2 min to avoid segregation. Tap water was poured gradually to the respective mix samples. After about 5 min, the samples were transferred into moulds with size 100 × 100 × 100 mm³ for casting. Samples were compacted layer by layer into the moulds and the top of the surface were flattened evenly.

Test program

Characterization of main composition and heavy metals in raw materials such as palm oil fuel ash, quarry dust and cement is an important parameter to determine the components exist in the materials. Chemical composition and heavy metals characterization using X-Ray Fluorescence (XRF) has been widely used and this method is fast, accurate and non-destructive with a minimum of sample preparation. To get the result, sample preparation needs to be done by preparing the pressed pellet. The density of all experimental samples was conducted according to BS EN 12390-7:2009. Cube samples with size 100 × 100 × 100 mm³ that has been cured for 7, 14, and 28 days were used in this test. Meanwhile, compressive strength test in this research were in accordance with ASTM-C109 which is a standard test method for compressive strength of hydraulic cement mortars. The cube samples with an average of three samples for each proportion were tested using a compression testing machine of 2000 kN capacity with 0.75 kN/s of loading rates. Total maximum load indicated by the testing machine and the compressive strength results were calculated using Equation 1 as follows.

$$fm = \frac{p}{a} \tag{1}$$

where, fm = compressive strength in MPa; P = total maximum load in lbf or [N], and a = area of loaded surface in^2 or [mm^2].

RESULTS AND DISSCUSSION

Chemical Composition and Heavy Metal Characterization

The chemical compositions of the materials are shown in Table 3 and Figure 4. The results indicated the higher percentage of chemical composition for POFA, quarry dust and sand are silica (SiO_2) which is 46.31%, 53.00% and 51.80% respectively. Meanwhile calcium oxide is the highest composition of Portland cement with 53.30%. On the other hand, the lowest concentration of chemical composition for Portland cement and quarry dust are titanium oxide with 0.23% and 0.51% respectively. Besides f that, potassium oxide (K_2O) shows the lowest percentage of natural sand with is 0.3% meanwhile for POFA, concentration of magnesium oxide (MgO) with 0.2% shows the lowest value.

Table 3. Chemical composition of raw materials

Chemical composition	Formula	Concentration (%)			
		POFA	PORTLAND CEMENT	QUARRY DUST	SAND
Calcium oxide	CaO	1.09	53.30	2.35	0.58
Potassium oxide	K_2O	2.81	1.07	5.05	0.30
Titanium dioxide	TiO_2	0.66	0.23	0.51	-
Ferric oxide	Fe_2O_3	4.76	3.14	4.52	0.39
Sulfur trioxide	SO_3	-	3.26	-	-
Silicon dioxide	SiO_2	46.30	12.6	53.00	51.80
Magnesium oxide	MgO	0.20	1.16	0.80	-
Aluminum oxide	Al_2O_3	22.00	2.79	11.70	7.30
Sodium oxide	Na_2O	0.45	0.19	1.75	-
Highest		SiO_2 (46.30)	CaO (53.30)	SiO_2 53.00	SiO_2 (51.80)
Lowest		MgO (0.20)	TiO_2 (0.23)	TiO_2 0.51	K_2O (0.30)

From the results, it has clearly seen that the most dominant of concentration in raw materials is calcium, silica and alumina. These chemical compositions that also known as C-S-H gel are the main materials that could enhance performance of the S/S matrices due to its pozzolanic reaction. The abundance of silica in POFA will generate a viable combination to create a good performance of S/S matrices. In depth, utilization of POFA improves resistance to chloride ion penetration (Chindaprasirt et al., 2008; Awal and Hussin 1999) enhances resistance to acidic environment (Tay, 1990) and sulphate attack (Jaturapitakkul et al., 2007).

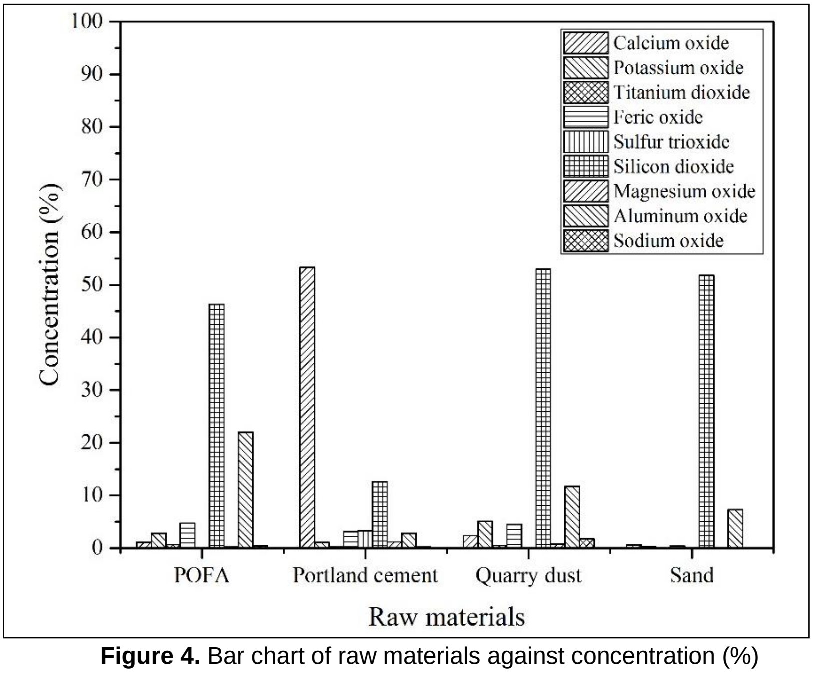

Figure 4. Bar chart of raw materials against concentration (%)

Based on Table 4, it shows that there were 11 major elements or heavy metals exist in materials such as POFA, Portland cement and quarry dust. The highest values for Fe are recorded highest element recorded in quarry dust (416 ppm) and Portland cement (296 ppm) respectively. Other than that, Ba recorded the second highest element in quarry dust and POFA with 389 ppm and 360 ppm respectively. Other than that, elements or heavy metals such as Cu (12 to 52 ppm), Pb (7 to 62 ppm), Zn (11 to 151 ppm), V (43 to 84 ppm), Ni (4 to 17 ppm), Cr (9 to 69 ppm) and As (6 to 37 ppm) are observed from the raw materials. Besides that, there were some heavy metals were not detected such Mn (POFA), Ba(Portland cement), Fe (POFA) and Cd (all raw materials).

Table 4. Heavy metals concentration of raw materials

Heavy metals concentration	Formula	Concentration (ppm)			
		POFA	PORTLAND CEMENT	QUARRY DUST	SAND
Copper	Cu	6	20	12	52
Lead	Pb	19	60	62	7
Zinc	Zn	34	132	151	11
Manganese	Mn	-	7	6	-
Vanadium	V	60	84	43	-
Nickel	Ni	8	17	14	4
Barium	Ba	360	-	389	24
Chromium	Cr	12	69	16	9
Ferum	Fe	-	296	416	-
Arsenic	As	16	37	6	6
Cadmium	Cd	-	-	-	-

Effect of density of palm oil fuel ash and quarry dust replacement in S/S matrices

The density values of palm oil fuel ash replacement and quarry dust replacement in S/S matrices are given in Figure 5 and Figure 6 respectively. The values for control samples are in the range of 2100 kg/m^3 to 2260 kg/m^3. From Figure 5, it shows that density of the different percentages of POFA with10%, 20%, 30% and 40% are much lower than the control samples. The lowest values of the density are recorded by 40% replacement of POFA ranging from 1380 kg/m^3 to 1430 kg/m^3. Meanwhile, the 10% replacement of POFA recorded a considerable value of density ranging from 1500 kg/m^3 to 1660 kg/m^3 respectively. From the observation, it shows that higher percentages of POFA replacements will decrease the density of the S/S samples. This hypothesis supported by Raut and Gomez (2016) that used oil palm fiber reinforced mortar utilizing palm oil fly ash as a complementary binder. It has been found that the mortar mix is slightly lower than the control mix due to POFA has lower density replacing the cement particles.

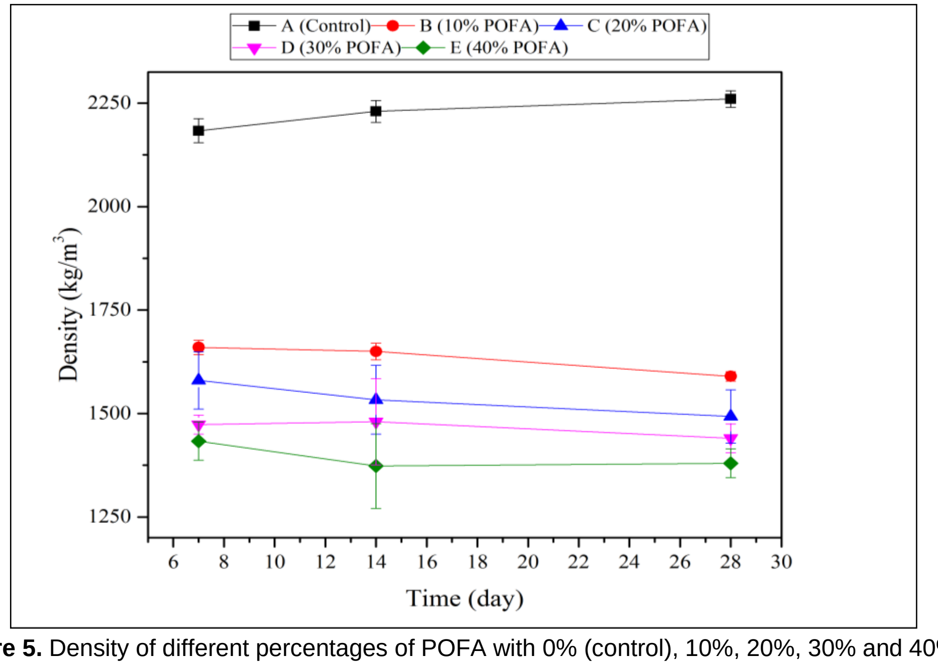

Figure 5. Density of different percentages of POFA with 0% (control), 10%, 20%, 30% and 40% at different ages (7, 14 and 28 days)

Similar trends of density are observed from quarry dust replacement in S/S matrices. It shows that density of the quarry dust samples recorded slightly lower than the control samples. 20% replacement of quarry dust in natural sand recorded a value of density ranging from 2080 kg/m^3 to 2147 kg/m^3. Meanwhile, the lowest was obtained from 40% of quarry dust replacement samples with average of 2073 kg/m^3 to 2087 kg/m^3.

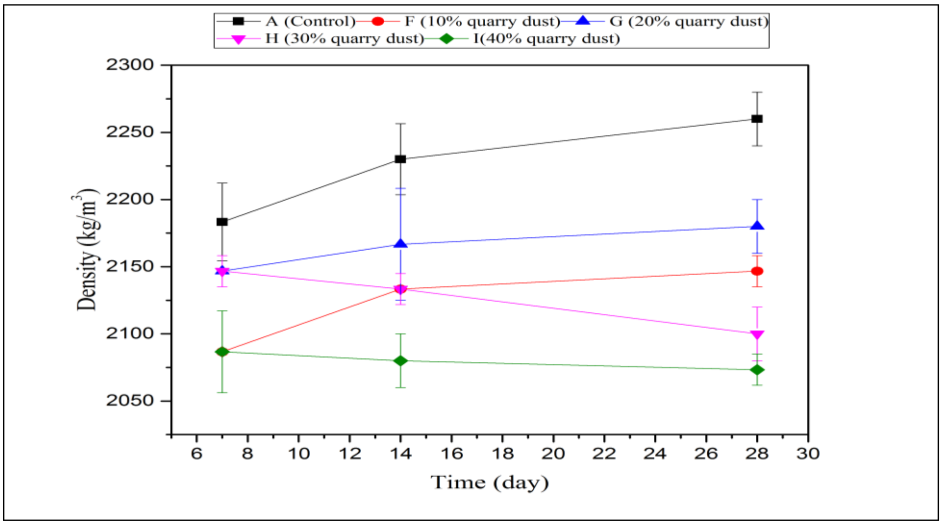

Figure 6. Density of different percentages of quarry dust with 0% (control), 10%, 20%, 30% and 40% at various ages (7, 14 and 28 days)

Effect of compressive strength of palm oil fuel ash and quarry dust replacement in S/S matrices

The results of compressive strength test of palm oil ash and quarry dust replacement at 7 days, 14 days and 28 days in triplicate samples are shown in Figure 7 and Figure 8. It can be observed that compressive strength develops over curing time.

By referring to Figure 7, the compressive strength of different percentages of palm oil fuel ash with 0% (control), 10%, 20%, 30% and 40% of various S/S matrices ages (7, 14 and 28 days) were presented. As expected, control sample without POFA and quarry dust replacement showed the highest value of compressive strength at each tested days. On 28 days, the highest compressive strength was achieved by control sample with 27.75 MPa followed by 10% of POFA (22.60 MPa), 20% of POFA (18.13MPa), 30% of POFA (11.62 MPa) and 40% of POFA (9.35 MPa).The development of compressive strength for POFA replacements was slightly slower than control samples. It can be seen from the difference of strength at 7 days to 28 days. In comparison to the control sample, the compressive strength of palm oil fuel ash replacement decreased 21.4%, 36.93%, 59.58% and 67.48% for 10%, 20%, 30% and 40% of POFA respectively. It indicated that higher value of POFA replacement influences the decreasing on the compressive strength compared to control samples. Addition of POFA produces tiny voids in between the particles that result in a wide spread of void size and thus lower the mechanical strength. Tay and Show (1995) suggested using oil palm ash sieved through a 150 µm sieve to be used as a replacement to cement and discovered that only a 10% replacement level showed a minor reduction in compressive strength than the control mix for compressive strength exposed to 1 year of curing indicating the presence of a pozzolanic reaction. Hence, to increase its pozzolanic reactivity, OPA must be ground to produce finer particle size material.

Other than that, all the samples were above the landfill disposal limit regulatory which is higher than 0.34 MPa at 28 days as suggested by Yin et al. (2008). However, only 10% of POFA replacement in S/S is above comparative mortar limit of 20 MPa on 28 days.

Figure 8 shows the compressive strength of different percentages of quarry dust as a partial sand material in S/S matrices. The strength at 28 days for quarry dust replacement is recorded at peak from 20% of replacement with 24.91 MPa followed by 10% of quarry dust (23.75 MPa), 30% of quarry dust (22.84 MPa) and lastly from 40% of quarry dust (13.80 MPa). It indicated that 20% of quarry dust replacement in sand is the most suitable to imitate control compressive strength. To be compared to the control sample, the compressive strength is decreased with by 10.23%, 14.44%, 17.69% and 50.28% for 20%, 10%, 30% and 40% of quarry dust respectively. These fluctuated of compressive strength are strong because fine particles may seal voids in the paste and act as precipitation sites for the hydration products, thus strengthening the paste. This phenomenon is known as filler effect. However, if large amounts of fines are added in concrete, the bigger aggregate grains are further separated by the smaller particles and the filler effect is reduced (Vogt, 2010).

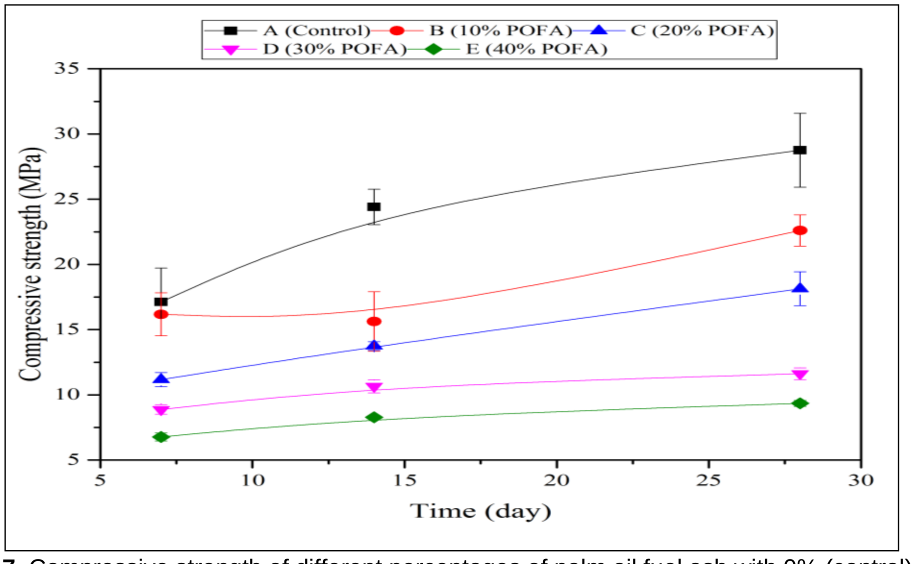

Figure 7. Compressive strength of different percentages of palm oil fuel ash with 0% (control), 10%, 20%, 30% and 40% at various ages (7, 14 and 28 days)

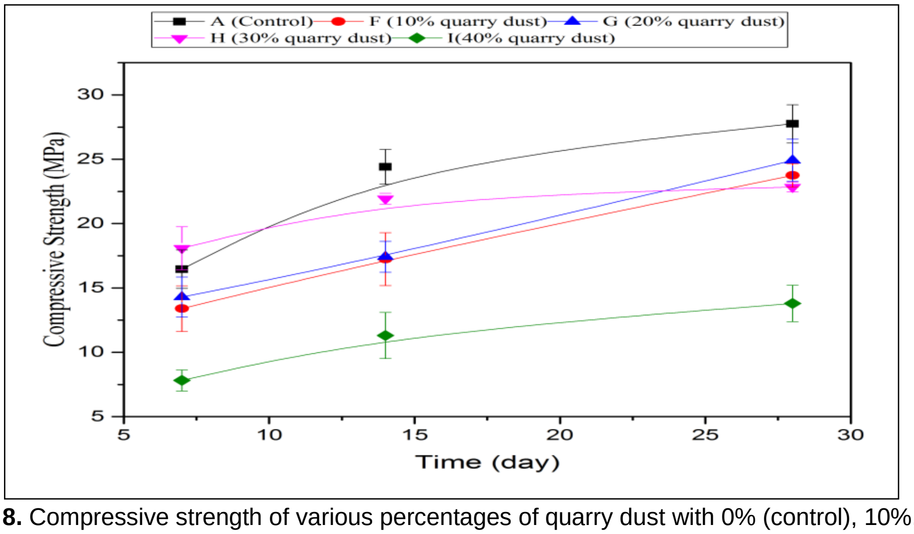

Figure 8. Compressive strength of various percentages of quarry dust with 0% (control), 10%, 20%, 30% and 40% at different ages (7, 14 and 28 days)

CONCLUSION

As a conclusion, this research is an attempt to study the feasibility of palm oil fuel ash and quarry dust replacement in S/S matrices. From the results, it shows that compared to the control samples, the best percentages of POFA and quarry dust replacement are 10% and 20% respectively. The results showed that, even though the strength was below than control samples, the replacement of POFA and quarry dust are still acceptable considering the replacement criteria such as cost reduction, waste utilization and waste immobilization in S/S matrices. These findings have been approved according to its composition, the comparable compressive strength during the curing period and also acceptable density. Replacement method could decrease the enormous quantities of palm oil and quarry dust disposal towards the environment. In addition to that, this study provides a sustainable solution by minimizing the treatment cost by utilizing palm oil ash to replace cement as well as quarry dust replacing sand providing low cost materials for S/S method that may lead to construction applications. Thus, a proper sustainable and safe way of reusing such waste materials as in this research is useful towards the environment and sustainability for development purposes.

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STRENGTH AND PHYSICAL PROPERTIES OF SUSTAINABLE CONCRETE MASONRY AT ELEVATED TEMPERATURE: A REVIEW

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Abstract
This paper is an overview of the performance of sustainable concrete masonry containing industrial waste as cement replacement which exposed to high temperature. These industrial waste or also known as pozzolanic materials have been widely used due to their characteristic which could act as the binder in concrete. By replacing cement content in concrete this could make concrete more sustainable due to less carbon content. Based on previous researches, the performance of these sustainable concrete have been investigated in terms of its residual compressive strength and physical properties at elevated temperature. These parameters are important factor that could contribute to the structural stability and valuation of serviceability state of structure. Large numbers of research on pozzolanic concrete at elevated temperature have been conducted. Among pozzolanic materials used were fly ash, palm oil fuel ash, volcanic ash, silica fume, blast slag furnace and etc. As general according to previous research, pozzolanic concrete have better performance than OPC concrete at temperature less than 500^{o}C. In terms of physical properties, surface cracks and discolouration of concrete has been observed. Studies show that, surface cracks on pozzolanic concrete were less than OPC concrete. Meanwhile, for discolouration impact, it was observed that, both OPC and pozzolanic concrete have similar changes in colour where the colour of concrete became lighter as the firing temperature increased. From this review it can be conclude that these industrial wastes are good in enhancing the sustainable environment as well as improving the fire resistance performance of concrete masonry.

Keywords: *Industrial Waste; Pozzolanic Materials; Concrete Masonry; Elevated Temperature*

INTRODUCTION

Sustainability is widely known as the most paramount factor in construction industries all over the world (Brizga et al., 2014). In UK for instance, the government are forecasting all the new domestic and commercial building to be zero carbon rated by 2016 and 2020 respectively (Alwan et al., 2015; Khalid et al., 2017). Meanwhile, in Malaysia, the implementation of sustainable construction and green building has been introduced in the Malaysia Construction Industry Master Plan (2005-2015).

One of the main concerns in construction industries is the large consumption of cement in the production of concrete. The usage of cement not only diminishes the natural resources such as lime stone yet it has contributed to the high emission of carbon dioxide and other greenhouse gases (GHGs) (Rehan and Nehdi, 2005; Ismail et al., 2016; Ramzi at al., 2016). According to Rahman et al. (2014), there is scientific report stated that about 7% of world's carbon dioxide emission was contributed by the cement production in industries. As well as stated by United Nation Environment Programme (UNEP) that one third or world carbon emission contributed by the building sector. Therefore, researchers have investigated the best alternative to overcome this problem.

Based on several research that have been conducted, researchers have found that large number of industrial waste such as fly ash (Babu et al., 2006), silica fume (Chen et al., 2010), bottom ash (Arenas et al., 2011) have good potential as cement replacement. This is because these industrial wastes have a pozzolanic characteristic which high in amorphous silica dioxide (SiO_2) and aluminium oxide (Al_2O_3). Pozzolanic characteristic own by this industrial waste enable these materials to act as the binder in the concrete.

It has been proved that about 10% to 20% replacement of cement by pozzolanic materials gave better strength than normal concrete (Awal and Hussin 2011; Lim et al., 2013; Rahman et al.,2014; Kroehong et al., 2011; Islam et al., 2016). Due to this, researchers have extended their research on fire resistance impact of pozzolanic concrete. This is because fire resistance is one of the vital parameter in determining the performance of building materials.

The effects of fire towards building materials such as concrete have been widely investigated. Ordinary Portland Cement (OPC) in the concrete has been proved to be one of best materials in concrete which could resist the impact of fire towards concrete (Phan, 1996). However, based on previous research it shows that there are changes in the physical and mechanical performance of concrete that exposed to elevated temperature. According to Ranjbar et al. (2014) these changes have been observed when this concrete was exposed to temperature more than 400ºC. This is because, at this stage, one of chemical composition of OPC which is calcium hydroxide will decompose into calcium oxide and water. The loss in strength is due to the dehydration of the calcium hydroxide and rehydration of calcium oxide. Due to this problem, researchers have extended their research on improvisation of the concrete or masonry unit performance at elevated temperature.

Large number of research on the effects of high temperature towards concrete containing pozzolanic materials has been conducted. Based on the findings, it shows that pozzolanic materials in concrete give a significant impact towards the physical and mechanical performance of concrete at elevated temperature. The following sections present a brief review of previous research on the pozzolanic concrete at elevated temperature.

EFFECTS ON RESIDUAL COMPRESSIVE STRENGTH

Residual compressive strength is defined as the strength of specimen after being exposed to heating and cooling process. This parameter has been investigated in order to determine the performance of specimens towards fire resistance (Ahn et al., 2016). Abundant of research have shown a significant effect on residual compressive strength of pozzolanic concrete at elevated temperature. Among pozzolanic materials that have widely used in the concrete are fly ash, silica fume, blast furnace slag, volcanic ash and palm oil fuel ash. Based on research conducted by Tanyildizi and Coskun (2008), fly ash has been used as the cement replacement in the concrete. The percentage of cement replacement was varied from 0% to 30% by weight. Meanwhile, the temperature exposed was 20ºC, 200ºC, 400ºC and 800ºC. From the findings, it shows that, at 200ºC the residual strength of concrete with 10% of fly ash is 98.34% of its original strength. However, concrete with 0% of fly ash has a residual compressive strength 97.95 of its original strength. This shows that, the existing of fly ash in concrete has helps in proving the residual compressive strength at elevated temperature. This is due to the formation of tobermorite which formed at high pressure and temperature.

A year before, similar research has been conducted by Aydin and Baradan (2007), where they have studied the effects of fly ash concrete when subjected to elevated temperature. However, in this research, the replacement of fly ash were higher which about 20% to 60%. As shown in Figure 1, it was observed that, all specimens gained strength when exposed to 300°C. Concurrently, the specimens show slightly decreased in strength when subjected to 600°C. Yet, the strength at this temperature was higher than strength at normal temperature. They have concluded that, the strength gained was due to strengthened hydrated cement paste during evaporation process which leads to greater Van der Waal's forces causes by cement gel layers which moving closer to each other. In a meantime, a significant fall in strength has been observed when specimens subjected to 900°C. Nonetheless, a positive effects shown by concrete containing 60% of fly ash where the residual compressive strength was increased. This clearly proved that fly ash could improve the fire resistance in the concrete.

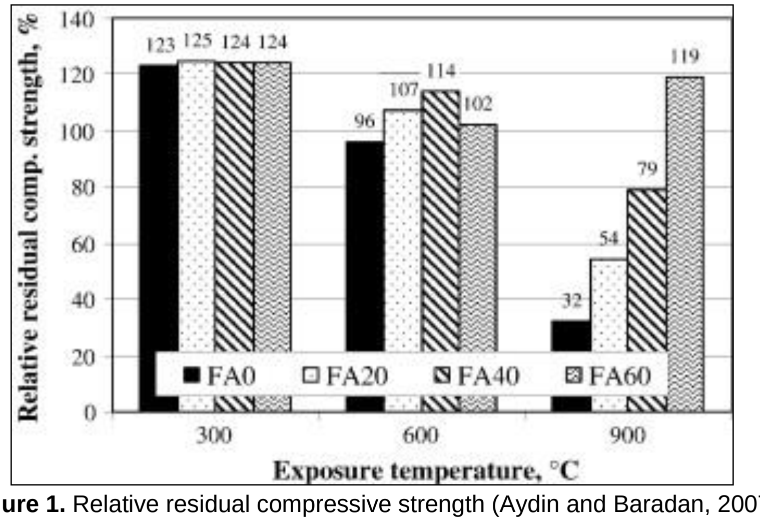

Figure 1. Relative residual compressive strength (Aydin and Baradan, 2007)

Earlier research on fly ash concrete was conducted by Xu et al. (2001). The replacement percentage was 25% and 50%. In this research, it was found that, the strength of fly ash concrete only increased when subjected to 250°C and the strength started to fall when subjected to temperature 450°C. This probably because the fly ash used was low-calcium (class F) fly ash. Therefore, less formation of C-S-H gel were formed.

Based on Awal and Shehu (2015), the performance of concrete containing high volume of palm oil fuel ash (POFA) exposed to elevated temperature was observed. In their research, the replacement of cement by POFA was 50%, 60% and 70%. At temperature of 200°C all POFA concrete shows significant increase in residual compressive strength. On the other hand, the OPC concrete loss its strength from 44.4Mpa to 42Mpa. Increase in strength probably due to the increase in the binding properties of calcium-silicate-hydrate (C-H-S) gel in the concrete. POFA concrete continue in gaining the strength when exposed 400°C this is due to the formation of extra C-S-H gel. However, there were significant fall in strength which exposed to temperature more than 600°C. The effect of temperature on residual compressive strength of concrete is illustrated in Figure 2.

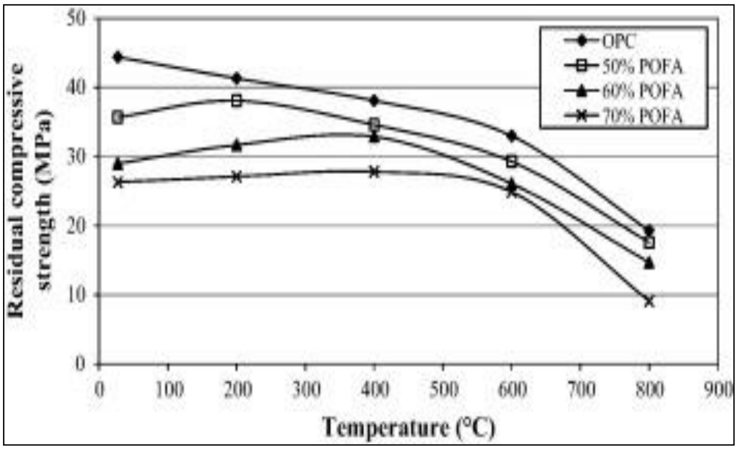

Figure 2. Effect of temperature on residual compressive strength of concrete (Awal and Shehu, 2015)

In research conducted by Ranjbar et al. (2014), POFA has been used as the replacement of conventional binder in geopolymer concrete which contains low calcium of fly ash. POFA was replaced the fly ash from 25% to 100%. All specimens were subjected to temperature up to 1000°C. According to the findings, it shows that all the residual strength of specimen was decreased as the percentage of POFA increase. However, despite the decreasing in strength due to high POFA content, it also shows that the residual strength was increased when all the specimens were subjected to temperature from 300°C to 500°C. Deterioration of specimen could be seen after it was subjected to 800°C and 1000°C.

Another research on POFA concrete at elevated temperature has been investigated by Ismail et al. (2010). Where in this research 20% of POFA has been replaced the OPC. The temperature range was 100°C to 800°C. At room temperature, the strength of OPC concrete was 45Mpa and POFA concrete was 41Mpa. Nevertheless, the strength of OPC concrete was decreased as the temperature increased. The strength of POFA concrete was slightly decreased at 100°C. However, the specimens keep gaining the strength when subjected to 300°C and 500°C of temperature. There were several factors that probably contributed in the strength gaining process which are the existence of change in thermal strain, fineness of POFA used and the hydration of un-hydrated cement. Finally, all the specimens were deteriorated when subjected to temperature greater than 800°C. In Canada, a research has shown a positive impact of volcanic ash in concrete at elevated temperature. In this research, the percentage of volcanic ash used was 5% to 20%. The exposed temperature was 25°C, 200°C, 400°C, 600°C and 800°C. It was observed that, volcanic ash concrete gained strength at 200°C and the highest strength gained was 20% volcanic ash concrete where the strength was increased about 14% from original strength. It was found that, the strength of concrete was decreased about 4% to 15% of original strength at 400°C. However, volcanic ash concrete has better performance than OPC concrete. The concrete strength was decreased as the temperature increased. All specimens were deteriorated at 800°C due to decomposition of C-S-H gel. Other than that, the loss in strength of concrete exposed to higher temperature probably due to the coarsening of the pore structure (Khandaker and Hossain, 2006).

For research conducted by Saridemir et al. (2016), two types of pozzolanic materials have been used in the production of concrete which are ground pumice (GP) and metakaolin (MK).

In this research, they have compared the performance of normal concrete with concrete containing GP and concrete containing GP and MK. In general, it was found that, all specimens were lost the strength as the temperature increase. At temperature of 200°C, there was no significant lost in strength in normal and pozzolanic concrete. In fact, there was slightly increase in strength for concrete containing 20% of GP. At this temperature, 20%GP concrete has highest formation of C-S-H gel. The strength continues to lose as the temperature increase to 500°C. However, it can be seen that, the strength of pozzolanic concrete at this stage was higher than normal concrete. Meanwhile, at 750°C all specimens show significant lost in strength due to the disintegration of C-S-H gel and increased in macro cracks.

According to previous research, majority of the research on pozzolanic concrete was focused on one type of pozzolanic materials. Different research has been conducted by Poon et al. (2001), where in this research the performance of high strength concrete at elevated temperature with different types of pozzolanic materials was conducted. Pozzolanic materials used were silica fume (SF), flay ash (FA) and blast furnace slag (BS). From the results as shown in Table 1, it shows that, fly ash and blast furnace slag concrete gained strength at 200°C due to the for C-S-H gel. However, strength of silica fume concrete was slightly decreased. Meanwhile, at 400°C, the strength of fly ash concrete and blast furnace slag were slightly decreased yet the strength was higher than normal concrete and silica fume concrete. It was found that, the structure of silica fume concrete was denser compared to other type of concrete. Denser structure causes splitting of concrete at high temperature. Therefore, this can be conclude that, silica fume is one of the pozzolanic materials that could not contribute in fire resistance of concrete.

Table 1. Residual compressive strength of concrete with different types of pozzolanic materials (Poon et al., 2001)

	Compressive Strength (MPa)				
Mix	**20**	**200**	**400**	**600**	**800**
HS-CC	91.3	88.0 (96%)	81.5 (89%)	53.0 (58%)	21.9 (24%)
HS-SF5	106.1	105.5 (99%)	98.7 (93%)	55.2 (52%)	22.3 (21%)
HS-SF10	119.9	117.7 (98%)	104.3 (87%)	52.8 (44%)	19.2 (16%)
HS-FA20	96.6	110.2 (114%)	92.9 (96%)	59.8 (62%)	27.0 (28%)
HS-FA30	102.8	124.6 (121%)	100.7 (98%)	68.9 (67%)	32.9 (32%)
HS-FA40	107.7	131.5 (122%)	112.2 (104%)	61.4 (57%)	32.3 (30%)
HS-SF+FA	123.9	135.3 (109%)	116.5 (94%)	63.2 (51%)	23.5 (19%)
HS-BS30	111.9	126.7 (113%)	108.5 (97%)	59.3 (53%)	30.2 (27%)
HS-BS40	115.5	133.3 (115%)	114.9 (100%)	70.5 (61%)	33.6 (29%)

EFFECTS ON PHYSICAL CHARACTERISTIC

Physical characteristic of concrete at elevated temperature could be observed by the formation of crack on the concrete surface and the changes in colour of concrete. Following section discuss the changes in physical properties of pozzolanic concrete after been exposed to elevated temperature.

Surface Cracks

Crack will be formed when concrete subjected to high temperature. When concrete subjected to high temperature, several transformations will be occurred such as moisture evaporation, internal vapour pressure, aggregates expansion, cement paste contraction and chemical decomposition (Shahidan et al., 2016). Usually, at the early stage, this transformation unable to cause any formation of crack except the rate of heating is extremely high (Peng et al., 2008; Shahidan et al., 2011). Research conducted by Ismail et al. (2010) has shown the impact of temperature rise on the formation of cracks on POFA concrete. 20% of POFA has been replaced the OPC content. From the findings, it shows that no formation of crack was observed on both OPC and POFA concrete at 100°C and 300°C. However, hair-line cracks were started to appeared at 500°C and became significant at 800°C. Hair-line crack at 500°C was due to development of adequate vapour pressure and the process of expansion and contraction which cause cracks in both OPC and POFA concrete. From this research, it was found that there was no significant different between OPC and POFA concrete. In other research by Tanyildizi and Coskun (2008), there was significant different in physical properties of OPC and fly ash concrete at elevated temperature. It was found that, the cracks started to appeared when subjected to 400°C. It also can be seen that, the formation of cracks decreased as the percentage of fly ash increased at the same time it was increased as the temperature increased. Similar observation was found by Ahn et al. (2016) where the addition of fly ash enables to reduce the formation of cracks. It can be conclude that fly ash could improve the physical properties of concrete when subjected to high temperature.

As well as fly ash, volcanic ash also has been proved as one of the pozzolanic materials that could improve the physical properties of concrete at high temperature. According to Khandaker and Hossain (2006), about 5% to 20% of volcanic ash has been replaced the OPC in concrete. From the observation, the cracks were started at 300°C. Width of cracks was increased as temperature increased in a meantime it was decreased as the percentage of volcanic ash increased. For OPC concrete, it was found that, there was one or two major crack in the middle of specimen. Meanwhile, for volcanic ash concrete, there were only minor cracks and no major cracks were observed. Poon et al. (2001) in their research has proved that silica fume is not good to be used as fire resistance materials. It was found that, at early stage firing, hairline cracks were formed at silica fume concrete surface. This is because the structure of silica fume was denser than other concrete which are fly ash and blast slag furnace concrete. Dense structure was results from vapour pressure formed by the evaporation of bound water. At final stage of firing, which at 800°C, OPC concrete shows major cracks however no splitting occurs. For fly ash and blast slag furnace concrete, only minor cracks were observed and for silica fume, the concrete was splitting. Figure 3 shows all the surface cracks patterns of different concretes at 800°C.

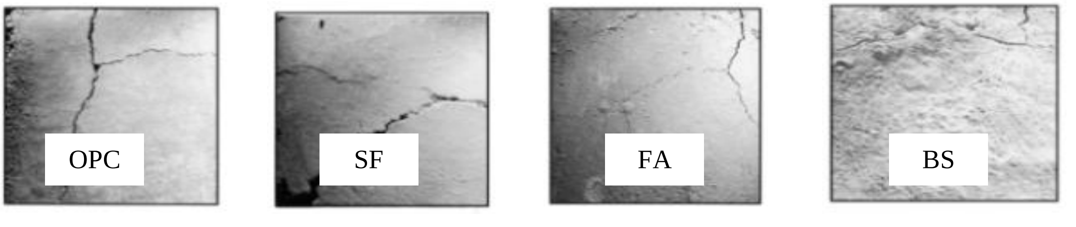

Figure 3. Surface cracking of concrete at 800°C (Poon et al.,2001)

However, different outcome has been determined by Saridemir et al. (2016), when concrete with ground pumice (GP) and metakaolin (MK) did not show any significant changes on the surface when it was exposed up to 500ºC temperature. The crack was observed when the specimens were exposed to 750ºC. In this research, it shows that, the existing of GP and MK did not contribute any impact towards formation of crack on concrete. It shows that, the formations of cracks increased as the temperature increased where similar observation has been made by Akçaözoglu (2013). The formation of cracks was the contributor of reduction in residual compressive strength. This explained the significant reduction in residual compressive strength of concrete when exposed to very high temperature.

Discolouration of Concrete

Heating process of concrete could affect the colour of the specimens. The colour of concrete changed as the temperature increased. The effect of high temperature towards discolouration of specimens was investigated extensively (Abdulkareem et al., 2014). According to Ismail et al. (2010), for OPC concrete, the colour was change from normal grey to whitish-grey as the specimens subjected to elevated temperature. At the same time, for POFA concrete, the concrete colour was changed from black (at room temperature) to light grey (at 800ºC). The dark colour was started to change when exposed to temperature of 300ºC. From this observation, it could benefits in the preliminary assessment of damage caused by fire hazards. Due to this, the intensity of fire can be figure out.

Another observation on POFA concrete has been made by Awal and Shehu (2014). For this research high volume of POFA has been used. It was noticed that, the colour of POFA concrete was change from black to light black at 400ºC. As the temperature increase, the colour keeps changing to light grey and eventually became brownish grey at 800ºC. Based on the observation, they have concluded that, the change in colour could be linked to the chemical transformation during heating process. Due to high content of Fe_2O_3 in POFA, this iron oxide has oxidized when subjected to temperature greater than 250ºC. Thus this process has contributed in colour changes.

Similar observation was identified by Ranjbar et al. (2014) where the colour of POFA was darker or black before exposed to high temperature. As the POFA content increased, darker colour was observed. In addition, it also shows that, colour of POFA concrete turned lighter when subjected to high temperature. The effect of elevated temperature towards discolouration of concrete is shown in Figure 4.

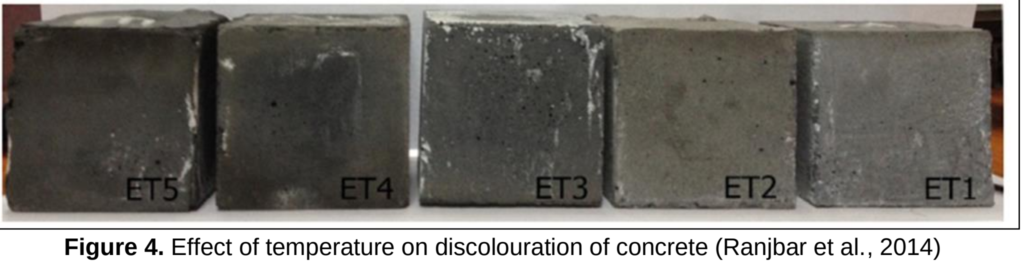

Figure 4. Effect of temperature on discolouration of concrete (Ranjbar et al., 2014)

For concrete with fly ash as the cement replacement, it was found that, the colour of concrete did not show any changes up to 200°C. The change in colour started to change at 400°C, where straw yellow was appeared on the surface. Meanwhile, at 800°C the colour turned to off white (Ismail et al., 2010). The findings of this research was similar to earlier research conducted by Li et al. (2004) where the stray yellow and off white were appeared on concrete at 400°C and 800°C respectively.

Parallel to observation by Saridemir et al. (2016), where discolouration of concrete was observed when specimens were exposed to elevated temperature. The specimens become lighter as the temperature higher. This is research, normal concrete and pozzolanic concrete show changes in colour when exposed to temperature 500°C and 750°C. The discolouration was due to the oxidized zones in the matrix.

CONCLUSION

This paper has reviewed several previous researches on concrete masonry containing pozzolanic materials at elevated temperature. Among parameters that have been focused are residual compressive strength and physical properties of concrete after subjected to elevated temperature. Following are conclusions that could be made based on the review.

Industrial waste has good potential as cement replacement due to their pozzolanic characteristic which could act as the binder in concrete. Therefore this could minimise the consumption of cement in production of concrete. It was found that, pozzolanic materials could improve the residual compressive strength of concrete when subjected to high temperature. However, it also shows that majority of pozzolanic concrete loss the strength when subjected to temperature greater than 500°C due to the disintegration of C-S-H gel.

Fly ash has been identified as one of the best pozzolanic materials that could improve fire resistance performance of concrete or increase the residual compressive strength at elevated temperature due to the formation of tobermorite. As the percentage of fly ash increased, the residual compressive strength increased. Meanwhile, silica fume was found to contribute in reduction of residual compressive strength of concrete due to its dense structure that could cause splitting of concrete at high temperature.

Pozzolanic also could reduce the formation of cracks on concrete surface. There were no major cracks were observed on pozzolanic concrete. Discolouration of concrete was observed when concrete were exposed to higher temperature. Pozzolanic concrete has darker colour than OPC concrete. It can be seen that, dark colour of pozzolanic concrete turned to lighter colour as the temperature was increased.

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THEME 2:

MATERIAL

SAND CEMENT BRICK CONTAINING COMPOSITE OF RCA AND PET AS SAND AGGREGATE REPLACEMENT

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Abstract
Brick has become one of the most important materials in the construction industry, which it is used for outer and inner walls of the buildings. As human population increases, the growth of the construction industry also increases to fulfil their living needs and it leads to the depletion of natural resources, environmental degradation and energy consumption. These problems pressure the industries to develop products and materials that are more environmental friendly as a solution. The aim of this study is to establish sustainable material properties for sand cement composite brick containing recycles concrete aggregate (RCA) and polyethylene terephthalate (PET). This study replaces the fine aggregates with 25%, 50% and 75% of recycled concrete aggregate (RCA) and 0.5%, 1.0% and 1.5% of waste PET in the production of brick. The best compressive strength was recorded at 24.9 MPa for brick contains 75% RCA and 0.5% PET. Meanwhile, an increment of RCA contents in brick increased the water absorption and initial rate of suction (IRS) percentages compared to a normal brick. The best mix ratio was the mix design of brick of replacement of 25% RCA and 1.5% PET. In a conclusion, the use of RCA and PET in the production of sand cement composite brick has provided better strength with tolerable results on water absorption and initial rate of suction compared to normal bricks.

Keywords: *Mortar brick, Concrete block, Concrete aggregate, Plastic fiber*

INTRODUCTION

The consumption of natural resources such as sand in brick production resulted in resource depletion, energy consumption and environmental degradation (Shakir et al., 2013; Sheikh Khalid et al., 2017; Zamir et al., 2017; Azmi et al., 2017). Currently, an increasing interest in environmental issues is pressuring industries to develop products and materials that are more environmentally friendly (Mazenan et al., 2017). Therefore, various attempts were made to use waste material in brick production. The use of waste polyethylene terephthalate (PET) and recycled concrete aggregates (RCA) from demolition wastes as a replacement of fine aggregates in the production of sand cement composite brick has been considered as one of the most sustainable materials to be added in the brick production (Shahidan et al., 2018). Therefore, the combination of both materials will promote new sand cement brick market opportunities, which are also favourable to the environment.

There are several researchers have studied the utilization of RCA wastes in concrete. According to Mirjana et al. (2010), a test was carried out on recycled aggregate concrete. For concrete age of 28 days, the authors claimed that the highest value of 100% RCA with water-cement ratio 0.5 was 45.66 MPa. According to Kou et al. (2012), a test was carried out to determine the properties of concrete with low strength of grade recycled aggregates. The results showed that the compressive strength of the concrete containing with low grade recycled aggregate were lower than that of the natural aggregate. This may be attributed to high water absorption of low-grade recycled aggregate incorporating with concrete.

Neno et al. (2014) investigated the performance of mortars by replacing natural sand with fine recycled concrete aggregates. The authors stated that the strength of mortar contains fine recycled concrete aggregate was higher than control. These results may be caused by some factors such as the sharp in its edge particles, more porous than sand and concrete particles have high specific surface. Therefore, the bond with the cement paste of the mix is better (Neno et al., 2014; Senin et al., 2016). Furthermore, RCA have greater cohesion between particles and strength due to non-hydrated cement as RCA will complete its hydraulic reactions and set when in contact with water during mixing.

Based on Ismail and Yaacob (2010), this research aims to discover the potential of recycled fine aggregate produced from demolition waste as a substitute for natural sand in brick. The result from the test concluded that the replacement of natural sand at levels of 50% and 75% by fine recycled aggregates showed good effects on compressive strength. However, the result is slightly lower in flexural strength as the volume of recycled aggregate exceeds 50% which is due to the high proportion of coarse aggregate in recycled aggregate that make the bricks more porous. Faisal et al. (2017) conducted an experiment by replacing natural sand aggregate with recycled concrete aggregate by weight for 55%, 65%, and 75% of production the sand cement brick. The brick with replacement by recycled concrete aggregate at 55% replacement provided the highest compressive and flexural strength as compared to other percentage replacements and control specimens. The authors claimed that, for replacement more than 55%, the strength of the brick was decreased for compressive and flexural strength, respectively.

For the experimental tests of concrete by Kou et al. (2012), the difference of RCA volume in size of less than 50mm was used as a replacement of natural aggregate. It showed that, the compressive strength of the concrete with 100% of RCA is 35.7% at 28 days which was lower than the natural aggregate concrete. However, the splitting tensile strengths of concrete made with 20% and 50% RCA were higher than natural aggregate. On the other hand, PET is a popular package for food and non-package products that commonly used to package soft drinks, water, juice, peanut butter, frozen foods and many other products (Leman et al., 2016; Saika and de Brito, 2012). Plastics consumption nowadays has become an integral part of our life. However, it is reported a world's annual consumption of PET drinks covers of approximately 10 million tons, which represents approximately 250 million bottles (Frigione et al., 2010; Irwan et al., 2013).

Therefore, some efforts have been made to explore the potential of waste in concrete besides it gives benefits to the environment. Different studies have been done on the usage of PET of different tensile strength, volume of fiber and water/cement ratios (Irwan et al., 2013; Khalid et al. 2017). Batayneh et al. (2007) studied with the ground plastics and glass to replace up to 20% of sand aggregates in concrete mixes. The author revealed that the ground plastics and glass waste materials could be reused successfully as partial substitutes for the sand or coarse aggregates in concrete mixtures. Albano et al. (2009) analysed the influence of content and particle size of waste PET bottles on the concrete behaviour at different water-cement ratio. Similar findings were found where the increased volume proportion and particle size of PET waste decrease the compressive strength, splitting tensile strength and modulus of elasticity of the concrete composite. The results signified the inclusion that PET reduce the structure strength and stiffness which give a promising feature where high strength is not necessary (Irwan et al., 2013).

The use of waste PET particles as the partial substitution of an equivalent 5% weight of natural sand in concrete was also investigated by Frigione (2010). The substitution however limited only to the fine aggregate fraction by 5% weight for particles in the fraction of 0.1 to 5mm which possessed similar grading curve to that substituted natural sand. Ramadevi and Manju (2012), also worked on the research of PET fiber as fine aggregate in concrete and result got revealed that an increase in compression and tensile strength. In the analysis of tests done, the compressive strength started to increase and it is been observed till 2% replacement of fine aggregate by PET bottle fibers and then the compressive strength gradually decreases. An experimental of addition of irregular PET fibre at 1% and 1.5% of volume fraction decrease the strength at about 5 to 6.4%, whereas 0.5% volume fraction IPET fibre exhibited an increment in concrete strength at 0.5%. (Irwan et al., 2014). The authors claimed that high content PET fibre increase the possibility of fibre balling during mixing and casting. Akçaözog˘lu et al. (2010) researched on the utilization of shredded waste PET bottle granules as a lightweight aggregate in the mortar. The authors reported that the compressive strength of the mixtures containing both sand and PET were higher than the mixtures containing PET without sand. However, porosity ratio of mixtures containing PET and sand aggregate together were higher than mixtures containing only PET. It also refers that the use of PET results in an increase in the shrinkage. Oliveira et al. (2011), carried out research on recycled PET fiber reinforced mortar, following the experiments of adding fraction of PET bottle in dry mix mortar about 0%, 0.5%, 1.0% and 1.5%. The authors found that it was significantly improving the flexural strength of mortars with a major improvement in mortar toughness. The maximum volume of PET fiber for a desired workability was 1.5%, which is the best performance of the mortar.

Reis et al. (2011) also reported there was a significant improvement in post-peak flexural behaviour of mortars using PET waste from beverage containers as partial replacement (by weight fractions of sand 5%, 10%, 15% and 20%) of sand in mortar. It is also observed that addition of shredded PET waste decreased the dry density of polymer mortars, fracture mechanics were altered by shredded PET and materials appeared to be more ductile. In this study, a series of physical, strength and durability tests were performed to determine the performance of brick containing RCA and PET as sand replacement materials. The specific gravity, density, compressive, water absorption and initial rate of suction (IRS) test have been conducted to determine the best mix proportion of RCA and PET contains.

EXPERIMENTAL

The experimental works are the sequence of operations that has been used for the study to be completed. This study investigated in terms of physical and mechanical through specific gravity, density, compressive, water absorption and initial rate of suction (IRS) tests.

Materials used

RCA was produced from the crushing of concrete cubes with a strength class of G30. RCA was sieved with a maximum size of 5mm after crushing it by a crushing machine. Waste

PET obtained from the PET plastics. The process started by collecting the mineral water bottles. Then, these bottles were processed in a granulator machine to obtain the standard size of fine aggregate. The irregular shape waste PET was sieve approximately or less than 5mm which made it is physically similar to size of fine aggregates. Figure 1 shows the irregular shape waste. Ordinary Portland Cement (OPC) was used in this study. Natural white sand was used as fine aggregates and this sand has passed the size sieve of 5mm.

Figure 1. The irregular shape waste PET

Mix proportions

The ratio of 1:3 was used as mix ratio of cement tosand which is in accordance to BS EN 998-2. The ratio was the most suitable mix proportion for mortar and it is stronger than it was necessary for most uses (Oliveira et al., 2011). The sand cement composite brick was designed according to the standard in order to exceed the require strength which is 7 MPa (MS 76:1972). The design mix was prepared according to BS 5628-3 design method. The optimum design mix used to produce sand cement composite brick with 25%, 50% and 75% of RCA and 0.5%, 1.0%, and 1.5% of PET waste as fine aggregate replacement. The details of mix proportion are given in Table 1.

Table 1. Mix proportion of sand cement composite brick

Mix Designation	Cement (kg)	Sand (kg)	RCA (%)	RCA (kg)	PET (%)	PET (kg)	w/c	Water (kg)
Normal		1.613	0	0	0	0	0.6	
RCA 25-PET 0.5		1.201	25		0.5	0.0017		
RCA 25-PET 1.0		1.193	25	0.354	1.0	0.0035	0.6	
RCA 25-PET 1.5		1.185	25		1.5	0.0052		
RCA 50-PET 0.5	0.463	0.798	50		0.5	0.0017		0.278
RCA 50-PET 1.0		0.790	50	0.709	1.0	0.0035	0.6	
RCA 50-PET 1.5		0.782	50		1.5	0.0052		
RCA 75-PET 0.5		0.395	75		0.5	0.0017		
RCA 75-PET 1.0		0.387	75	1.063	1.0	0.0035	0.6	
RCA 75-PET 1.5		0.379	75		1.5	0.0052		

The brick moulds were formed according to the scale from MS 76: 1972 and BS 3921: 1985 which stated 215mm length x 103mm width x 65mm height for 1 unit of normal brick as illustrated in Figure 2. The curing process was to produce better sand cement composite brick consist of RCA and PET waste and also to prevent the brick from hydrating. In this study, the method of curing was aired curing process as shown in Figure 3. This process was conducted to allow the bricks cure under atmospheric temperature. The duration of the curing process in this study was 7 and 28 days.

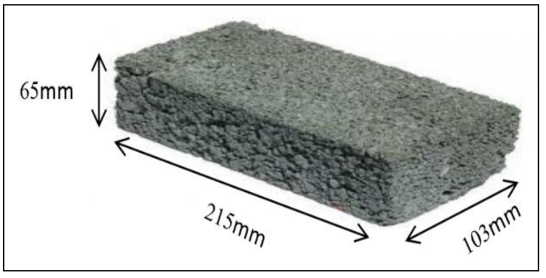

Figure 2. Size of unit sample

Figure 3. Air curing

Specific gravity test

Specific gravity test is the ratio of the weight of the unit volume of aggregate to the weight of an equal volume of distilled water at a stated temperature. The test was conducted on the RCA, PET and fine aggregate to determine its characteristics. A specific gravity test was conducted according to ASTM C128.

Compressive strength test

Compressive strength of the specimens was conducted in accordance to BS 6073-1:1981 and the test procedure was carried out using the compression testing machine. Three specimens of bricks were tested for each test. Control bricks and the brick containing recycled concrete aggregate and PET were tested for the compressive strength at the age of 7 and 28 days. The collapse load was recorded, which is the strength of the sample and the average compressive strength of the three specimens were recorded.

Water absorption test

Water absorption test was carried out to measure the ability of brick to allow water to pass through it. The standard used in this study follows the BS 3921:1985. Water absorption in percentage by mass after 24 hours immersed in clean water at temperature of 27±2ºC (Figure 4) can be calculated using following Equation 1.

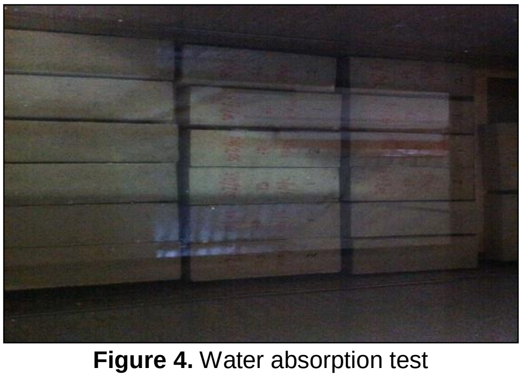

Figure 4. Water absorption test

$$W = \frac{M_2 - M_1}{M_1} \quad (1)$$

where:

W = Water absorption in %
M_1= weight of dry brick (g)
M_2= weight of wet brick (g)

Initial rate of suction (IRS)

IRS test was conducted by measuring the constant mass brick to heating at 105°C for not less than 48 hours (BS 3921). Then, brick was removed from the oven and cooled at room temperature. The brick was weighed and dry mass m_1 was recorded. After that, 2 steel bars or rod were placed at the bottom of the container approximately 100mm apart to form a platform for the brick. The water level inside the container should be about 4mm. The pre-weighted brick were placed on the bar and the water level was observed closely using measuring gauge to ensure the depth of immersion was maintained at 3±1mm through 1 minute as shown in Figure 5. IRS kg/m^2 can be calculated using the following Equation 2.

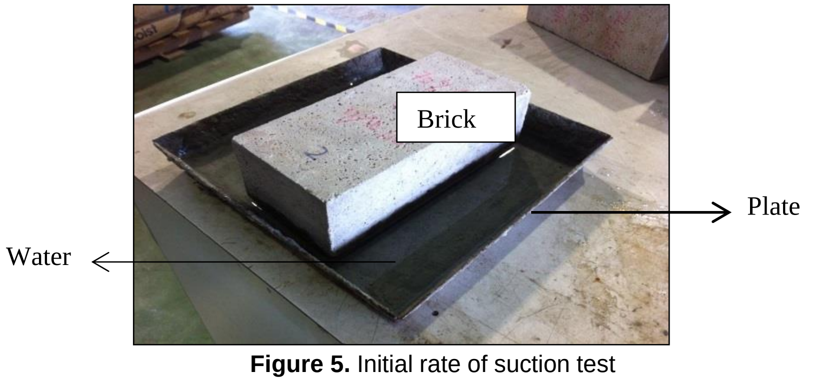

Figure 5. Initial rate of suction test

$$Initial\ Rate\ of\ Suction\ (IRS) = \frac{1000\ (M_2 - M_1)}{A} \quad (2)$$

where:

M_1= weight of dry brick (g)
M_2= weight of wet brick (g)
A= net area of contact surface of brick with water (mm^2)

RESULT AND DISCUSSION

The overall results of material testing of composite brick from the experimental works are presented, analysed and discussed in detail in this section. The material testing of bricks are focused on its physical and mechanical properties which are the specific gravity of materials, density of brick, compressive strength, water absorption and initial rate of suction (IRS) tests.

Specific gravity

Referring to Table 2, it shows that the specific gravity for PET was lower as compared to RCA and fine aggregate. The specific gravity of fine aggregate was 2.30 while for RCA and PET were 2.22 and 1.09 respectively. The lower value of specific gravity of PET shows that PET was lighter than that of natural fine aggregates and recycled concrete aggregate. From the Table 2, it also indicated that the water absorption materials for recycled concrete aggregate was higher than PET and fine aggregate. Similar findings on the performance of recycled aggregate concrete by Shi et al. (2015), the authors founded the water absorption for RCA was higher than natural fine aggregate. This was due to the large quantity of hydrated cement paste in the demolished concrete and the migration of water absorbed by the aggregate to the paste around particles of aggregates, which influence the volume of water and pores in the paste.

Table 2. Specific gravity and absorption of FA, RCA and PET

Material		Weight of aggregate (g)		Water absorption $\frac{A-B}{B}$ X 100 (%)	Average (%)	Specific gravity
		SSD (A)	Oven Dried (B)			
Fine Aggregate	1	500	490	2.04	2.15	2.30
	2	500	489	2.25		
RCA	1	500	485	3.09	3.20	2.22
	2	500	484	3.31		
PET	1	500	495	1.01	1.05	1.09
	2	500	496	0.81		

Bricks density

The density of brick specimens was calculated by dividing the weight with the volume. Density was calculated to determine the density of hardened composite bricks after 7 and 28 days. The results for all densities of the samples are shown in Table 3.

Table 3. Density data for bricks specimen

No. of Batch	RCA (%)	PET (%)	Weight (kg)		Volume (m^3)	Density (kg/m^3)	
			7 Days	28 Days		7 Days	28 Days
1	0	0	2.91	2.88	0.0014	2078.57	2057.14
2	25	0.5	2.88	2.83		2057.14	2021.43
3		1.0	2.83	2.83		2021.43	2028.57
4		1.5	2.86	2.86		2135.71	2085.71
5	50	0.5	2.96	2.92		2150.00	2064.29
6		1.0	2.95	2.94		2085.71	2100.00
7		1.5	2.92	2.88		2107.14	2064.29
8	75	0.5	2.99	2.92		2114.29	2042.86
9		1.0	2.93	2.90		2078.57	2071.43
10		1.5	2.93	2.86		2078.57	2042.86

From Table 3, it shows that a few samples reduced their weight at 28 days. For normal bricks, the weight differences are about 0.03kg between 7 days and 28 days. For RCA 25%, the reduction of weight can be seen when the PET volumes is 0.5% and 1.5%. While, the weight of 1.0% of PET increased about 0.01kg. The huge reduction of weight happened for the mixture of RCA 50% with PET 0.5% and PET 1.5%. For RCA 75%, the weight slightly reduced for 0.5%, 1.0% and 1.5% of PET. Each dimension of bricks were calculated and the average volume obtained is 0.0014m^3. Thereafter, the densities were calculated using the recorded weight for each specimen.

The value of densities is shown graphically in Figure 6. It shows that the density of normal bricks was 2078.57kg/m^3 for 7 days and 2057.14kg/m^3 for 28 days. The lowest reduction of density was the brick content with 25% of RCA and 1.5% of PET slightly reduced by 1.7% for 7 days and 2.1% for 28 days compared to a normal brick. Meanwhile, the mix designation brick of 75 % RCA and 0.5% PET indicates an increment of 2.9% (7 days) and 2.1% (28 days) compared to a normal brick. Figure 7 shows that increasing the PET contents as sand materials replacement reduced the density of brick. The brick contents 1.5%PET presented the lowest density among the mix designation of brick. The high volume of RCA with lower content of PET presents higher density compared to normal brick as shown in Figure 7. However, mix design content high volume of RCA and PET presents significant results to reduce the density of brick compared to normal brick.

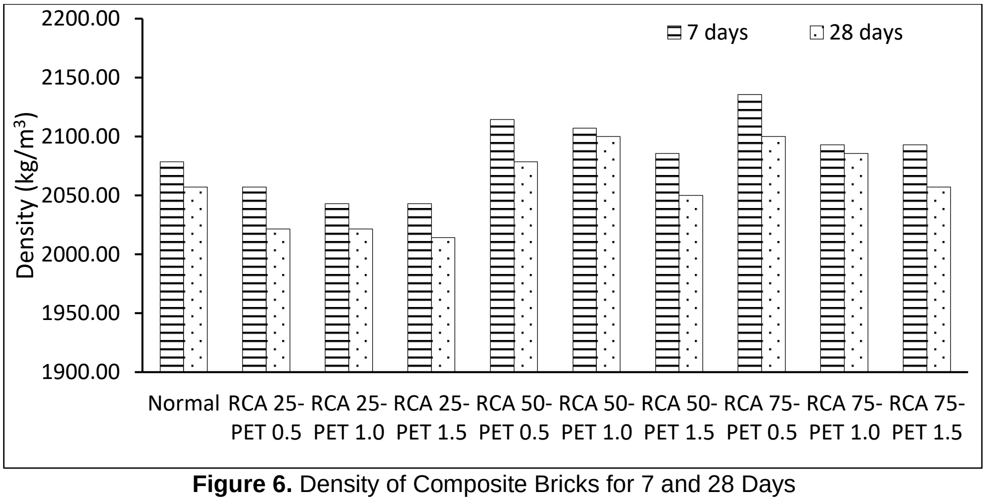

Figure 6. Density of Composite Bricks for 7 and 28 Days

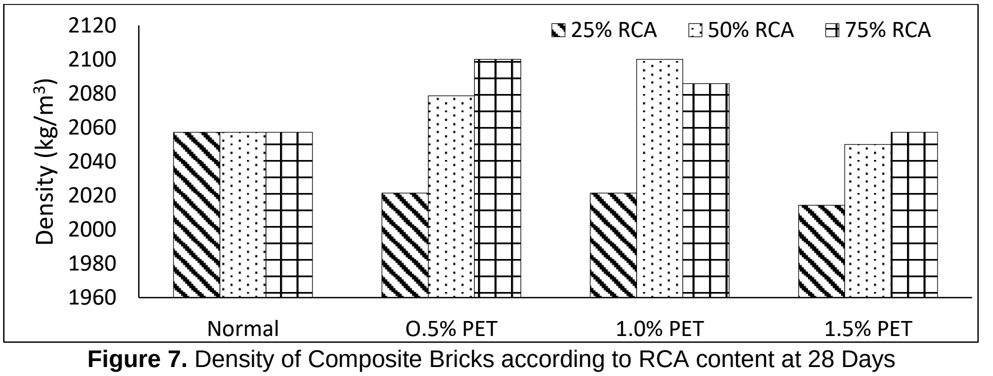

Figure 7. Density of Composite Bricks according to RCA content at 28 Days

According to Oliviera et al. (2011), density and moisture content are not significantly altered by incorporation of PET fibers in volume or size. The hardened mortar density results showed that the addition of fibers causes a small decrease in density. These means that partially substituting sand with PET and RCA may produce lighter bricks. However, the volume content of RCA incorporating in concrete needs to be determined (Lim, 2009).

Compressive strength

Compressive strength of bricks was tested at age 7 and 28 days. The investigation was done on the twelve (12) types of brick samples which contain 25%, 50% and 75% RCA with an addition of 0.5%, 1.0% and 1.5% of PET. Consequently, similar findings also agreeable. The average compressive strength for each percentage of the waste in the bricks is tabulated in Table 4.

Table 4. Compressive strength of brick specimens

RCA (%)	PET (%)	Compressive Strength (MPa)					
		7 Days			28 Days		
		Strength	Avg.	Standard Deviation	Strength	Avg.	Standard Deviation
0	0	13.9	13.10	2.16	15.4	14.10	1.30
		14.8			12.8		
		10.7			14.1		
25	0.5	7.8	7.17	1.54	10.9	8.70	1.89
		8.3			7.9		
		5.4			7.4		
	1.0	21.0	17.53	3.08	19.5	19.30	0.82
		16.3			20.0		
		15.3			18.4		
	1.5	21.3	23.50	2.62	19.8	24.20	3.47
		22.8			25.6		
		26.4			26.0		
50	0.5	21.0	20.60	0.85	24.6	22.20	3.76
		21.3			17.9		
		19.7			24.2		
	1.0	21.1	20.20	1.50	21.9	22.20	0.61
		18.5			21.8		
		21.1			22.9		
	1.5	21.8	21.60	0.32	22.1	22.20	1.25
		21.2			21.0		
		21.7			23.5		
75	0.5	22.8	22.20	0.55	22.0	24.90	2.57
		21.9			25.5		
		21.8			27.0		
	1.0	19.8	19.60	2.71	21.7	23.70	1.73
		16.8			24.6		
		22.2			24.8		
	1.5	19.3	19.50	0.67	23.9	23.60	1.87
		20.2			25.3		
		18.9			21.6		

From Table 4, a standard deviation was calculated to indicate the improvement of strength for the composite brick matrix. Also, it clearly can be seen that the value of strength started to decrease as the ratio of RCA and PET as sand replacement increased at 7 days of curing age. But, the strength of composite brick on 28 days shows that the strength starts to increase as the replacement of waste is increasing. For a better understanding, the result is shown graphically in Figure 8.

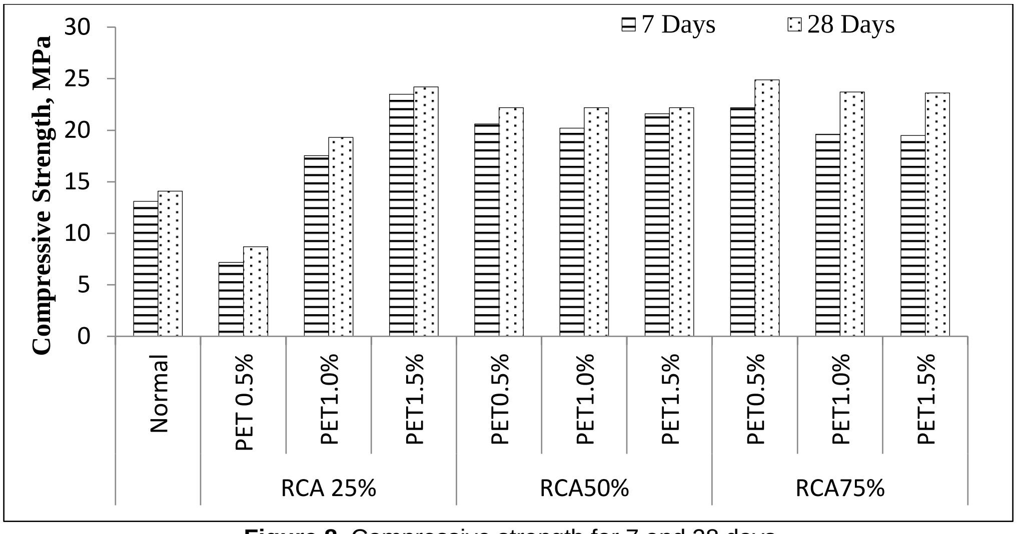

Figure 8. Compressive strength for 7 and 28 days

From Figure 8, all the samples achieved the strength more than the minimum strength for both 7 and 28 days in BS 6073-1:1981 which stated that the compressive strength of bricks should not be less than 7 N/mm². It obviously shows that some samples for both curing age achieved high strength from the normal brick. The sample with RCA 25% with PET 1.5% exhibit the second highest strength whereas RCA 25% with PET 0.5% achieve only 7.17 MPa which has lower strength rather than a normal brick. Nevertheless, the specimen with RCA of 25%, it shows the strength increased in line with the increment of PET volume for 0.5%, 1.0% and 1.5%. However, the strength for both curing age started to decrease for bricks containing 50% and the increasing volume of 0.5% and 1.0% of PET also reduced the strength of the composite brick. In contrast, the drastically reduced of strength was shown above similarly be drawn in the studies conducted by Poon et al. (2002) which conducted a study on RCA and reported that the replacement levels of 25% and 50% in bricks and blocks had little effect on the compressive strength of specimen. However, the authors mentioned that high level of replacements of these materials reduced the compressive strength significantly.

The increasing pattern of compressive strength is seen in samples containing RCA 75% for both curing ages of 7 and 28 days. The addition of 0.5% PET and 75% of RCA gave the highest strength among the other samples. The reduction of strength was shown when the volume of PET increased from 1.0% to 1.5% for RCA 75%. In addition, the highest percentages of increment strength for 7 days and 28 days among other samples were 17.30% and 17.37% respectively for 75% of RCA with a volume of PET for 1.0% and 1.5%. The overall result showed instability in compressive strength of bricks when RCA and PET were added. Similar studies have also been conducted by Ismail & Yaacob (2010) that the addition of recycled fine aggregate can successfully increase the compressive strength of bricks. Even with 100% of natural sand was replaced with recycled fine aggregate, the obtained results showed similar strength to bricks with natural sand. Dina et al. (2012) studied the compressive strength of solid cement bricks containing crushed brick aggregates where the results indicated that the strength gradually decreased by increasing the content of crushed bricks. The reduction of strength using recycled concrete as a substitution to the natural sand can be explained by the parameters of water-cement ratio, water absorption of RCA and ratio of replacement (Sheikh Khalid et al., 2017). Moreover, the results show in Figure 8 revealed that only a certain amount of PET fiber increased the strength with an addition of RCA of 25%, 50% and 75%. The trend of this study having the same pattern with the studied conducted by Irwan et al. (2013) which stated the good bonding between aggregate and binder as increase in PET content may result a decreasing in the strength of materials. The study also agreed with the conclusion made by Ismail et al. (2008) where the decreased trends attributed to the decrease in adhesive strength between the surface of the plastic waste and the cement paste.

Figure 9 illustrates the relative compressive strength of all types of samples for both curing age. The relative compressive strength, defined as the ratio of the compressive strength of the composite brick containing RCA and PET as sand replacement to that of the conventional sand cement brick. From Figure 9, percentage of average compressive strength for increment is in the range of 33% to 80% for 7 days and 36% to 77%. However, there is a reduction of ratio for samples with 25% RCA and 0.5% PET from a normal brick for both curing age.

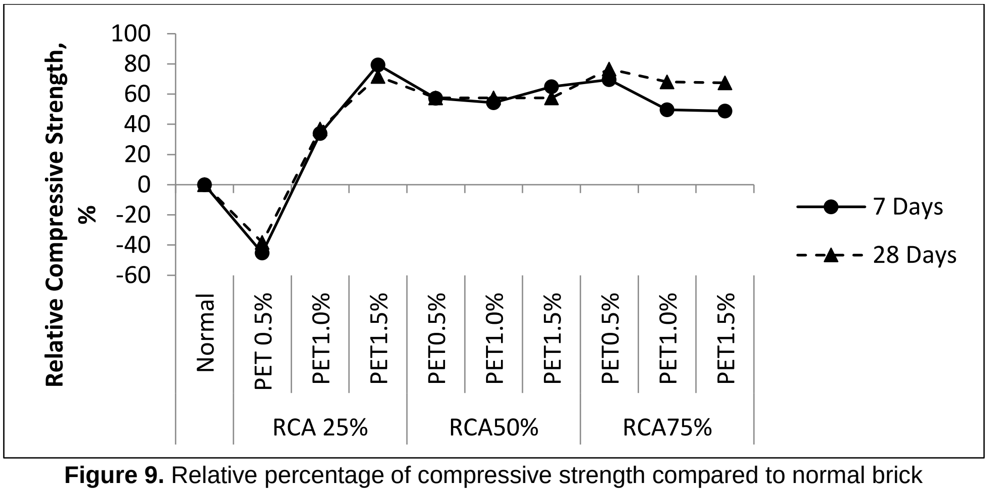

Figure 9. Relative percentage of compressive strength compared to normal brick

The highest percentage difference from normal brick was the RCA 25% with 1.5% PET of 79.39% at curing age of 7 days. The relative compressive strength for RCA 25% with PET 0.5% showed a reduction from normal a normal brick where the normal brick achieved a higher strength rather than the specimen. Based from the overall results obtained, the replacement of RCA and PET showed higher relative percentages of compressive strength compared to a normal brick similarly to that of Ismail & Yaacob (2010) which exhibited about 70% difference of brick containing waste of RCA with normal brick.

Water absorption

Figure 10 shows the evolution of water absorption from 7 to 28 days for all mixtures. The results illustrated that the water absorption increased with the replacement percentage of fine aggregate by the RCA and PET wastes for all mixtures. The results indicated that the sample with brick contents of RCA25% was less permeable when compared to brick containing 50% and 75% of RCA. The highest water absorption coefficients were obtained in sand cement composite brick with RCA75%PET1.5%. Samples with 75% of RCA absorbed more water than the samples containing 50% and 25% of RCA. This finding was in agreement with previous studies conducted by Bravo et al. (2015) and Oliveira et al. (2011) where the addition of RCA in mixtures increased the percentage of water absorption. The increased in water absorption was due to the increment in the porosity of the bricks containing RCA and PET and the poor bonding between cement paste and polymer pellets. The presence of cracks in the bricks also contributed to the increment in water absorption. It can be summarized that the water absorption of sand cement composite bricks with recycled fine aggregate and PET significantly affects the increased water absorption characteristic of bricks. From the results, only bricks with 25% RCA and 1.5% PET showed less permeable compared to control brick. Thus, one would expect the density to decrease and the water absorption to increase as the RCA and PET content is increased.

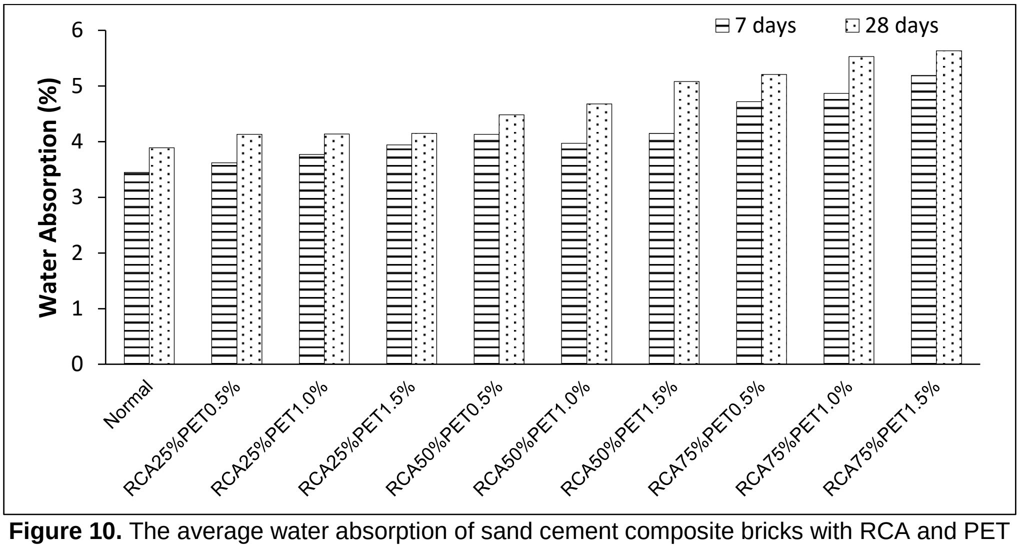

Figure 10. The average water absorption of sand cement composite bricks with RCA and PET

Initial rate of suction (IRS)

According to the results in Figure 11, brick content of RCA 25% PET 0.5% gives the lowest value of initial rate with 0.41 kg/m^2/min at 7 days. A similar trend was observed for the bricks containing RCA 25%PET 0.5% at 28 days with 0.45 kg/m^2/min. Samples produced using 75% of RCA showed greater initial surface absorption than did samples containing 50% of RCA content. Thus, sample with RCA75% were more porous than RCA 50%, therefore more water penetrated in the sample from their pores, thereby increasing the initial surface absorption rate. Nevertheless, the initial rate values of RCA and PET mixtures were found to be satisfactory. As illustrated in Figure 11, the conclusion that was drawn from the test showed that the initial surface absorption of brick increased with an increment in replacement contents. As explained previously, this can be attributed by the higher water absorption rate of RCA compared to the fine aggregate. A higher concentration of RCA results in greater initial surface absorption. These findings were in agreement with the studies conducted by Fan et al. (2016). In addition, further study needs to be conducted as the IRS does not have specific standard for the study of potential fine aggregate to be replaced with RCA and PET in bricks.

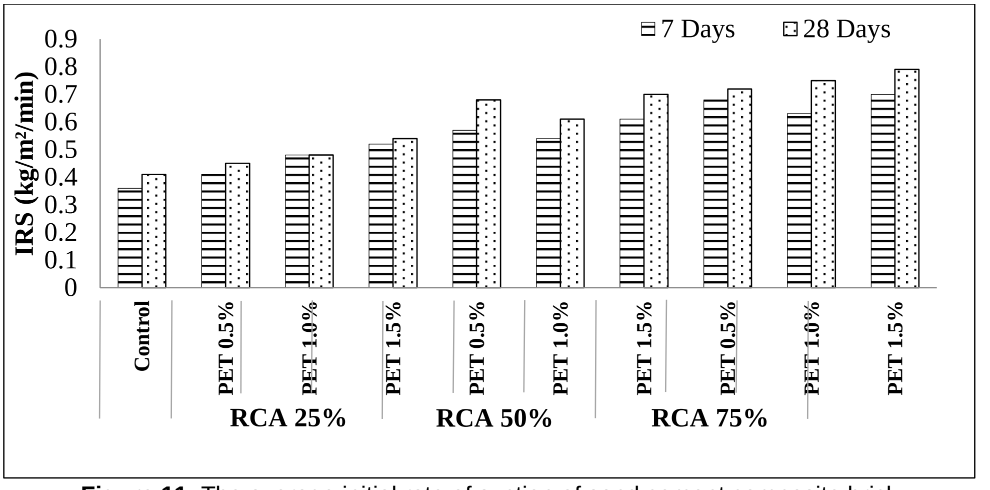

Figure 11. The average initial rate of suction of sand cement composite brick

CONCLUSION

From the results obtained on the physical properties testing, it is clearly shown that the RCA and PET are suitable to be used as sand replacement in composite brick. It was found that the specific gravity for sand is much higher than RCA and PET with a value of 2.865 that can be explained that there is an absence of mortar in sand.

Composite brick shows that there are some samples that increase and reduced their size which may be influenced by materials content and density of the constituent materials. However, overall average shows all specimens had a uniform size and surfaces are similar with natural sand cement brick. Other than that, the density of composite brick exhibits the higher density than normal brick for both curing age of 7 and 28 days as the RCA contents increase. It can be discussed that the high percentage of RCA (75%) can increase the strength of the bricks rather than 25% and 50% of RCA. However, the lower volume fraction of PET is seen to be suitably added when the volume of RCA is high but whenever the high volume of PET is used.

The results illustrated that the water absorption and IRS increased with the increases in replacement percentage of fine aggregate by the RCA and PET content for all mixtures. This was due to the increase in the porosity of the bricks containing RCA and PET and the poor bonding between cement paste and polymer. From the results, only brick containing 25% RCA show less permeable compared to a normal brick. The best mix ratio was the mix design brick of replacement of 25% RCA and 1.5% PET.

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THE EFFECT OF USING DIFFERENT SUBSTITUTES OF RUBBER ASH ON COMPRESSIVE STRENGTH OF CEMENT MORTAR

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Abstract

The utilization of industrial waste products in cement paste has attracted attention both with the energy crisis in the 70s and the rise of environmental consciousness. Most cement paste properties can be improved by incorporating different kinds of industrial wastes. In this study, the changes of the properties for rubber ash cement mortar were investigated with respect to different content of rubber ash as cement and as sand substitution and filler. Rubber ash was produced from the pyrolysis process. The cement paste is postulated to be a potential material especially for construction applications which are subjected to impact such as crash barrier, bridges and roads. Cubic specimens were produced by adding 10% volume rations of rubber as fillers, sand replacement and cement replacement into M30 quality cement mortar. The physical and mechanical tests were conducted at the end of day 7, 14 and 28. Tire rubber ash is more suitable to be used as sand replacement. It also works as an alternative to cement replacement and fillers. The density of the mortar specimen decreased with the presence of rubber ash. The highest reduction in density occurred when tire rubber ash was used as sand replacement. The compressive strength of the mortar specimen reduced when rubber ash was added in the cement mortar. At the early stages, tire rubber ash used as fillers had better compressive strength however at 28 days the use of tire rubber ash as cement replacement had the strongest compressive strength.

Keywords: *Rubber ash; Pyrolysis; Mortar; Cement; Density, Compressive strength.*

INTRODUCTION

Many countries face major environmental issues related to solid waste management around the world. Previous studies have indicated that waste tires constitute a significant portion of non-hazardous solid waste materials (Al-Akhras and Smadi, 2004). All over the world, the amount of waste rubber has gradually increased over recent years because of the increasing use of rubber products (Topçu and Sarıdemir, 2008). A large number of automobile, truck and off-road tires are discarded all over the world every year, and only a small proportion is either retreaded or burned to produce energy. In most countries, the majority of discarded tires are either shredded and then landfilled or stockpiled (Fattuhi and Clark, 1996). An estimated 1000 million tires reach the end of their useful lives every year.

The increasing amount of waste tires worldwide make the disposition of tires a relevant problem to be solved. One of the most crucial environmental issues all around the world is the disposal of the waste materials. Accumulations of discarded waste tires have been a major concern because the waste rubber is not easily biodegradable even after a long-period landfill treatment. However, material and energy are alternatives to disposal of the waste rubber.

Other researcher propose to use it as fuel material or as raw materials of rubber goods (Sukontasukkul and Tiamlom, 2012; Sheikh Khalid et al., 2017). Recycle of used rubber conserves valuable natural resources and reduces the amount of rubber entering landfill (Youssf et al., 2015).

Application of recycled rubber particles as alternative aggregates in concrete have received significant attention over the last two decades this is due to the massive quantities of scrap vehicle tires that have already accumulated and the increased annual production estimated to be one billion tires produced worldwide in 2011 (Hall and Najim, 2014). At present enormous quantities of tires are already stockpiled (whole tire) or landfilled (shredded tire), 3000 million inside EU and 1000 million in the US (Pacheco-Torgal et al., 2012). These stockpiled tires are dangerous because they pose a potential environmental concern such as fire hazards, and provide breeding grounds for mosquitoes (Gesoğlu and Güneyisi, 2011; Xue and Shinozuka, 2013; Meddah et al., 2014). The implementation of the Landfill Directive 1999/31/EC and the End of Live Vehicle Directive 2000/53/EC banned the land fill disposal of waste tires creating the driving force behind the recycling of these wastes (Shahidan et al., 2011). Tire rubber wastes are already used for paving purposes. However, it can only recycle a part of these wastes (Vieira et al., 2010). Some research has already been conducted on the use of waste tires as aggregate replacement in concrete showing that concrete with enhanced toughness and sound insulation properties can be achieved (Mazenan et al., 2017). The additional of fibrous rubber to concrete improve shock wave abrasion, reduced heat conductivity and noise level and increase resistance to acid rain (Noor et al., 2015; Khalid et al., 2017). Rubber aggregates are obtained from waste tires using two different technologies: mechanical grinding at ambient temperature or cryogenic grinding at a temperature below the glass transition temperature. The first method generates chipped rubber to replace coarse aggregates. As for the second method, crumb rubber is usually produced to replace fine aggregates (Pacheco-Torgal et al., 2012; Md Nor et al., 2011; Shahidan et al., 2017).

Rubber is used as an additive in concrete to form a composite. When the aggregates are replaced by crumb rubber, the compressive strength of concrete is reduced significantly (Moustafa and ElGawady, 2015; Son et al., 2011; Gupta et al., 2015; Xue and Shinozuka, 2013; Atahan and Yücel, 2012; Khalil et al., 2015; Liu et al., 2012; Noor et al., 2016). However this rubberized concrete have the ability to absorb a large amount of energy under compressive and tensile loads (Topçu and Avcular, 1997; Li et al., 2004). It is believed that the increase in impact resistance was derived from the enhanced ability of the material to absorb energy (Zheng et al., 2008; Topçu, 1995).

In line with these characteristics, rubberized concrete was recommended to be used in circumstances where vibration damping was required, such as in the highway construction as a shock absorber or sound barriers and in buildings as an earthquake shock-wave absorber (Topçu and Avcular, 1997; Fattuhi and Clark, 1996). Adding shredded rubber to concrete softens the concrete, yielding greater plastic deformation on impact and smaller deceleration forces(Atahan and Sevim, 2008). Chou et al. (2010) suggested that cement-based mortar and concrete products, modified with tire rubber as a partial replacement for sand, could be used in applications where mechanical properties are not of prime importance but high resistance to chloride ion penetration is required This effect could possibly save lives of vehicle occupants, while simultaneously preventing rubber from going to landfills or stockpiles.

However, the main disadvantages of utilizing waste tire rubber in concrete is its reduce the compressive strength and elastic modulus (Khorami and Ganjian, 2011; Thomas and Chandra Gupta, 2016; Youssf et al., 2016; Hilal, 2017). Eldin and Senouci were first to study concrete tire aggregate in 1993. Their research consisted on replacing fine (1 mm) and coarse (6 mm, 19 mm, 25 mm and 38 mm) aggregate. They found that concrete tire aggregate had lower compressive and tensile strength. They also observed that the loss of compressive strength increased with the size of the tire aggregate (Bravo and de Brito, 2012). The larger size of rubber particles added into the concrete the lower concrete strength, however the smaller size rubber particle the higher concrete strength (Liu et al., 2012; Li et al., 2014). By referring to the previous research, most of them are basically used waste tire as fine and coarse aggregate with size 1 mm to 40 mm. There are relative lacks of research area on the waste tire with size less than 1 mm. Therefore, this research mainly focusing on the uses of fines rubber crumb and rubber ash (less than 1 mm) as partially replacement fine aggregate in concrete.

Recent study investigated methods for increasing the damping capacity of concrete by replacing up to 20% of the fine aggregate with shredded rubber. Generally, some researchers reported an increase in damping and decrease in compressive strength was reported (Moustafa and ElGawady, 2015). The study on utilizing the waste tire in concrete has been carried by Bravo et al. (2012), Ismail et al. (2016), Leman (2016) and Shahiron et al. (2016). The utilizing waste tire rubber in concrete postulate to enhance the concrete characteristic. However, rubber tire aggregate was not applied in the construction industry due to lack of supportive data. Therefore, this research was carried out to investigate the opportunity of utilizing the waste tire in concrete as construction material in construction industry in order to minimize the waste product in the same time preserve the natural aggregate. This research was emphasized on the effect of different substitutions of tire rubber ash in cement mortar as cement replacement, sand replacement and fillers.

Rubber Ash

Rubber ash also known as carbon black is end product of tire went through the pyrolysis process. The pyrolysis involves subjecting tire to high temperature of 400 to 500°C, in absence of oxygen. Figure 1 illustrates the pyrolysis process from the waste tires through the pyrolysis plant to form the rubber ash. From the pyrolysis process the tires are broken down into their three-basic component which are fuel oil/gas, wire steel and carbon black. From a single tire, approximately 15% of the weight is steel, 30% becomes carbon black, 45% oil and final 10% gas as shown in Figure 2.

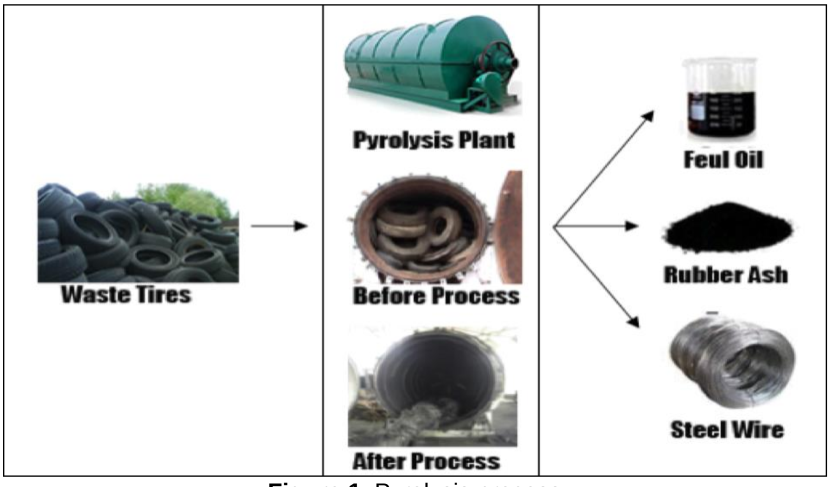

Figure 1. Pyrolysis process

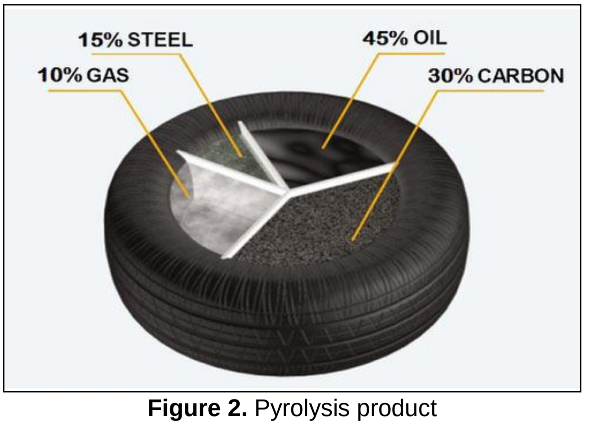

Figure 2. Pyrolysis product

EXPERIMENTAL WORK

Materials

Ordinary Portland Cement type 1 was used in the study. Only one type of cement used in this study, there is no other cement was used to avoid any different properties of cement. The cement was directly used from the original packaging without any alteration. The physical properties and chemical analysis of the cement are presented in

Table 1 and

Table 2 respectively. Figure 3 shows the scanning electron microscopy (SEM) of the cement used in this study at 100 times magnification. The median grain size for the cement was 18µm. Cement has fines grain size compared to the sand and rubber ash.

River sand was used as fine aggregate which has a maximum size of 5mm. The sand was dried using air-dried condition. Figure 4 shows the SEM of the sand used in this study at 100 times magnification. Sand has bigger grain size compared to the rubber ash and cement. The shape of the sand was angular. Rubber ash was obtained from the pyrolysis industry located at Gopeng, Perak, Malaysia. The rubber ash has a maximum size of 1mm. Figure 5 shows the

SEM of the rubber ash used in this study at 100 times magnification. Rubber ash has round in shape. This rubber ash directly used without any alteration.

Table 1. Physical properties of Type I cement used in this study

Property	Unit	Type I cement
Specific gravity	-	3.15
Passing grain size	%	78
Median grain size	μm	18
Blaine specific surface	m^2/kg	300
Initial setting time	Min	145
Final setting time	Min	270

Table 2. Chemical analysis of Type I cements used in this study

Component (%)	Type I cement
SiO_2	21.2
Al_2O_3	5.5
Fe_2O_3	3.1
CaO	63.7
MgO	1.5
SO_3	2.63
Na_2O	0.18
K_2O	0.71
TiO_2	-
Cl^-	-
Zn	-
Loss on ignition	0.96

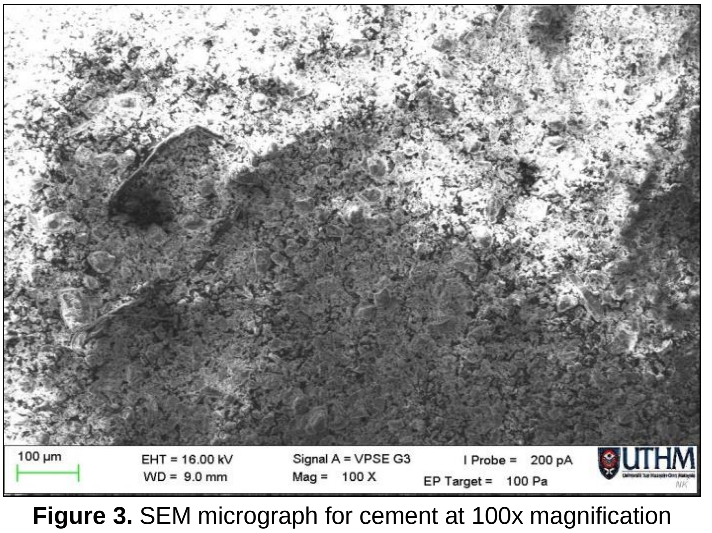

Figure 3. SEM micrograph for cement at 100x magnification

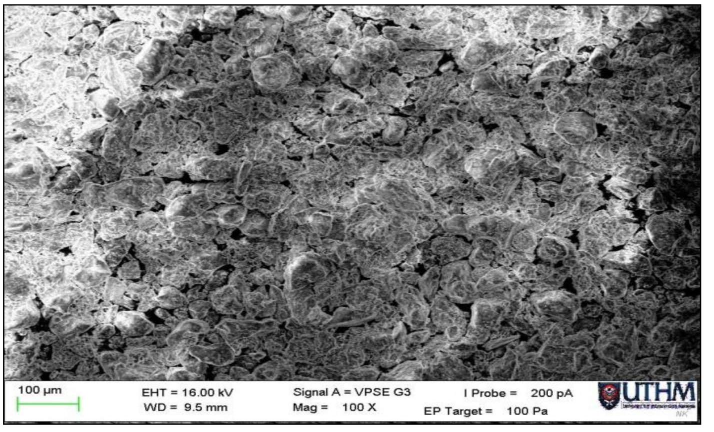

Figure 4. SEM micrograph for sand at 100x magnification

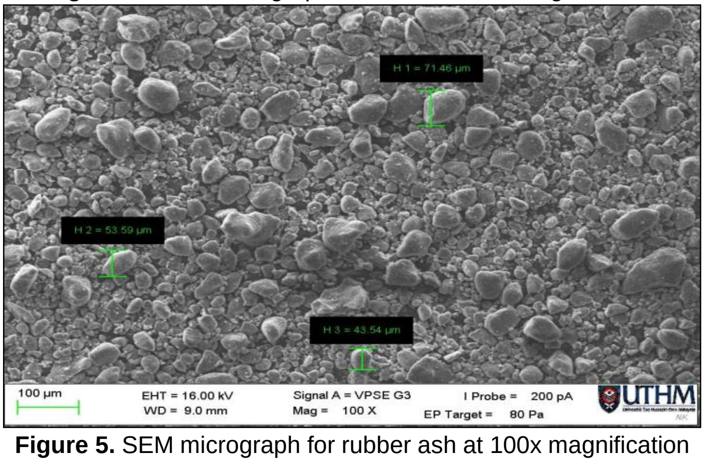

Figure 5. SEM micrograph for rubber ash at 100x magnification

Preparation of Mortar Specimen

The cement mortar was manually mixed by hand. First the cement, sand and rubber were dry mixed for 3 minutes. After that water was poured into the mix for another 3 minutes. The mortar mixture proportions were 1:2:0.55 by volume of the cement, sand and water, respectively. The rubber ash was added to the mortar mix as 0% for control (CT), 10% of the total volume as fillers (FIL), 10% of the volume of sand as sand replacement (SR) and 10% of the volume of cement as cement replacement (CR).

Table 3. Mortar mixture proportions used in the study

Mix	Cement (cm^3)	Sand (cm^3)	Rubber Ash (cm^3)	Water (cm^3)
1	6000	12000	0	3300
2	5400	12000	600	3300
3	6000	10800	1200	3300
4	6000	12000	1800	3300

Table 3 shows the mortar mixture proportions used in this study. The size of the specimens for compressive test were 100mm cube. Three mortar specimens were prepared and tested to obtain an average value for each test condition. Each specimen was cast in three layers and compacted. After casting, all specimens were left for 24 hours before being demolded and cured in a curing tank at 23±2 °C until the time of testing (Ramzi Hannan et al., 2016; Senin et al., 2016).

Test Procedures

The compressive strength test and density test for hardened mortar mixture was conducted according to ASTM C109-02 and ASTM C948-81. The compressive strength specimens were tested using a compression machine. The rate of compressive loading was 7kN/s. The rate of loading remained constant throughout the testing program.

RESULT AND DISCUSSION

Density

The addition of rubber ash reduced the density of the mortar specimens. Figure **6** shows the density of the mortar specimens for CT, CM, SM and FIL at 7, 14 and 28 days. The CT specimens had the highest density compared to others and SM had the lowest density compared to others. The density of the mortar specimens had increased over time. The reduction of density for the mortar specimens compared to CT at 7 days were 2.70%, 4.93% and 3.82% for CM, SM and FIL respectively. The reduction of density for the mortar specimens compared to CT at 14 days were 3.00%, 4.89% and 3.47% for CM, SM and FIL respectively. The reduction of density of the mortar specimens compared to CT at 28 days were 3.24%, 5.56% and 5.09% for CM, SM and FIL respectively. From the data, it has shown that the density for the rubber ash lighter than sand.

Nadal Gisbert et al. (2014) explained the increase of the rubber contain in the concrete reduce the density of the concrete due to the gaps surround the rubber. Gesoğlu et al. (2014a & 2014b) discovered the lighter weight concrete was produced by adding rubber in to concrete. The rubberized concrete densities were 2-11% lighter than control concrete specimens. Gesoğlu et al. (2014b) prepared the lowers density of 1900 kg/m^3 while the highest density is 2240 kg/m^3 and found that the fine particles of the rubber the higher the density of the rubberized concrete. This is due to the fine rubber played the role of fine aggregate and filled the small gaps among the concrete particles which improve the density and not effected more the permeability coefficient “K”.

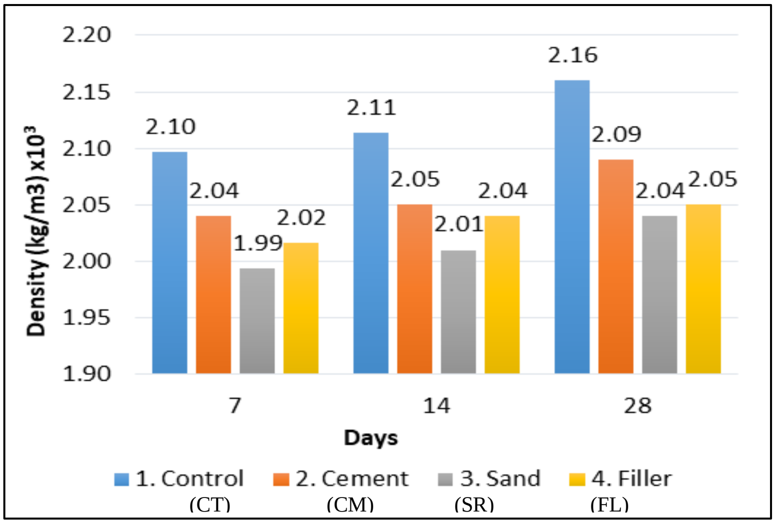

Figure 6. Graph Density of Cement Mortar Against Time

Compressive Strength

Figure 7 shows the development of compressive strength with curing period (7, 14 and 28 days) for hardened mortar containing 10% rubber for cement replacement (CR), sand replacement (SR) and filler (FIL). The rate of compressive strength developed was relatively high between 7 and 14 days, followed by a slower rate between 14 and 28 days. The rapid development of compressive strength of mortar containing different rubber replacements during the early stages of 7 and 14 days indicates rapid hydration during this period. The development of strength continues even at later stages. The compressive strength depends on the physical and mechanical properties of the constituent materials. If part of the materials is replaced by rubber, reduction in strength will occur. The compressive strength of mortar specimens for FIL indicated a higher value for the 7 days compared to CR and SR. However, the compressive strength of the cement mortar decreased when the rubber ash was added to the cement mortar because CT had the highest value among others. The percentage increase in compressive strength at 14 days was 17.77%, 17.95%, 28.57% and 26.50% for CT, CR, SR and FILL respectively. The percent increase in compressive strength of mortar specimens at 28 days were 2.31%, 20.03%, 14.07% and 3.16% for CT, CR, SR and FIL respectively. The percentage decrease in compressive strength of the mortar specimens at 28 days compared to control specimens were 16.79%, 17.12%, 22.60% for CR, SR and FIL respectively. From the finding it was in line with the previous study which the rubberized concrete slightly decreased compared to normal concrete (Chou et al., 2010; Moustafa and ElGawady, 2015). Gupta et al. (2015) explain the compressive strength of waste rubber tire fibre concrete decreases with the increase in the replacement level of fine aggregates by rubber fibre however the strength increases on partial replacement of cement by silica fume.

Al-Akhras and Smadi (2004) observed the increase in compressive strength when the tire rubber ash was replaced for fine aggregates up to 10%. The increase in compressive strength of mortar specimens at 90 days were 14%, 21%, 29%, and 45% at tire rubber ash content of 2.5%, 5%, 7.5% and 10% respectively. Rubber ash contains 14.6% zinc oxide that enhances the mechanical and physical properties of the cement mortar. Other than that rubber ash particle can consider as a nano material because the particle is smaller than 100nm that can fill all the void in the cement mortar (Senin et al., 2016).

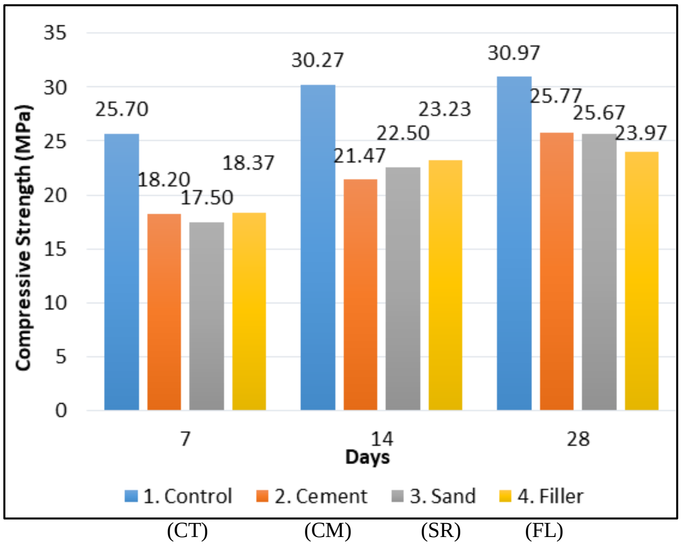

Figure 7. Compressive Strength of Cement Mortar against Time

CONCLUSIONS

This study presents the results of different substitution of tire rubber ash cement mortar amended with. Based on the results of the study the following conclusions are drawn:

- Tire rubber ash is more suitable to be used as sand replacement. It also works as an alternative to cement replacement and fillers.
- The density of the mortar specimen decreased with the presence of rubber ash. The highest reduction in density occurred when tire rubber ash was used as sand replacement.
- The compressive strength of the mortar specimen reduced when rubber ash was added in the cement mortar. At the early stages, tire rubber ash used as fillers had better compressive strength however at 28 days the use of tire rubber ash as cement replacement had the strongest compressive strength.

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UTILISATION OF WASTE GLASS POWDER AS A PARTIAL REPLACEMENT OF CEMENT IN SUSTAINABLE MORTAR PRACTICE

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Abstract
Millions of tonnes of waste glass are being generated annually worldwide. Waste glass, which is often disposed in landfills, causes unsustainability because it does not decompose in the environment. Glass is principally composed of silica. This work investigates the performance of cement mortar containing glass powder (GP) as a partial substitution for cement. Portland cement was partially replaced with 10%, 20% and 30% GP. The unit weight and compressive strength of the cement mortar after a curing period of 7 and 28 days were measured. Although the compressive strength of the cement mortar decreased, its unit weight had no significant change and remained equal to those of the control samples. Therefore, GP can be considered as a viable replacement for cement and is thus an economical construction material.

Keywords: *Cement mortar; Compressive strength; Replacement of cement; Sustainable mortar; Waste glass powder*

INTRODUCTION

Concrete, which is a mixture of Portland cement, aggregates (including fine and coarse aggregates) and water with or without admixtures, is the most commonly used construction material. Replacement of Portland cement with waste materials such as fly ash, bagasse ash, silica fume and marble powder has been the focus of many studies. Waste glass powder (GP) can also be used as a binder with the partial replacement of cement and as a filler material. When ground to a very fine powder, waste glass shows pozzolanic properties because of its high SiO_2 content. Therefore, waste GP, to some extent, can replace cement in concrete and contribute to strength development.

The production of Portland cement, an essential constituent of concrete, leads to the release of a significant amount of CO_2, a greenhouse gas. One tonne of Portland cement clinker production generates approximately 1 tonne of CO_2 and other greenhouse gases.

As environmental issues are increasingly becoming relevant because of the increasing awareness regarding the sustainable development of the cement and concrete industry (Naik and Moriconi, 2005; Shahidan et al., 2011), the incorporation of recycled materials in construction materials has started to attract considerable attention (Irwan et al., 2013; Ismail et al., 2016). Owing to population growth, the amount and type of waste materials have increased accordingly. Many of the nondecaying waste materials remain in the environment for hundreds or perhaps thousands of years (Leman et al., 2016; Khalid et al., 2017). The accumulation of waste, particularly nondecaying waste, is a global problem. Most of these materials are left as stockpiles and landfill materials or illegally dumped. Thus, in order to minimise the accumulation of waste materials and the depletion of virgin raw materials, several methods for recycling waste materials have been introduced (Sheikh Khalid et al., 2017). For example, some waste materials are currently used in concrete mixtures (Wang et al., 2004; Senin et al., 2016; Shahidan et al., 2016).

The use of waste materials such as waste glass in the construction industry has significantly increased in the past few years because they do not have any adverse effects on the properties of concrete. This provides an effective platform for the disposal of waste materials in permanent concrete structures.

The amount and composition of solid waste generated vary among countries and residential areas. Moreover, the amount of household waste differs among households which may also be correlated with population, national state income and person's level. The average typical composition (% by weight) of solid waste consists of organic matter (67.0%), paper (8%), textiles (2%), plastics (7%), glass (2%), metals (2%), sand (10%) and other materials (2%) (Abu El Qomboz and Busch, 2010; Shahidan et al., 2018).

In Gaza, solid waste accumulation has become a serious problem because of excessive waste dumps in the region and insufficient efforts by the authorities in solid waste management. In 2010, the Ministry of Planning estimated that 1300 tonnes of waste materials were generated by the Gaza Strip per day. This amount is estimated to reach 2350 tonnes/day after 20 years.

LITERATURE REVIEW

Millions of tonnes of waste glass are generated annually worldwide. The disposal of glass in landfills is not sustainable because it is principally composed of silica (Aarthi, 2016) which does not decompose in the environment.

In 2005, approximately 12.8 million tonnes of waste glass were generated in the US alone, and only 20% of this amount was recycled (Enviromental Protection Agency, 2006). Thus, recycling and reusing waste materials have become increasingly relevant owing to inadequate landfill sites in urban areas and the increasing cost of land filling.

Glass is an amorphous material with high-silica content, making it potentially pozzolanic when the sizes of its particles measure less than 75 μm. Glass is considered a pozzolanic material because of its ground silica content and its ability to exhibit properties similar to other pozzolanic materials. Powdered waste glass can be used as a partial replacement for cement in concrete. However, when used as aggregates in Portland cement concrete, crushed glass is susceptible to expansion and cracking due to alkali silica reaction. In the present study, we prepared concrete mixtures with different proportions of GP ranging from 10% to 30% with an increment of 10%. The concrete specimens were later tested for compressive strength after 7 and 28 days of curing (Shekhawat and Aggarwal, 2014; Sharif et al., 2017; Shahidan et al., 2017).

The main chemical component of waste liquid crystal glass is SiO_2. When liquid crystal glass replaces cement in concrete, the working ability of concrete tends to decrease with the increase of liquid crystal glass used whereas the compressive strength decreases as the proportion of the substitute increases (Pereira-de-Oliveira et al. 2012). Due to water repellence, waste glass can be used in concrete for the reduction of water absorption and shrinkage (Kumarappan, 2013).

Waste glass is a material generated during glass manufacturing which is possibly beneficial for resource and energy savings (Lam et al. 2007). In many countries, cullet is placed into a smelting zinc furnace for the production of substitute materials for silica.

The main aim of this study is to investigate the performance of cement mortar containing GP as a partial substitution for cement. Portland cement was partially replaced with 10%, 20%, and 30% of GP. The unit weight and the compressive strength of the specimens were measured after 7 and 28 days of curing in water.

The significance of this study is to determine the performance of mortar made of GP as a partial replacement of cement. The importance of this work lies in the establishment of the performance of mortar containing GP and to compare it with ordinary plain concrete. The study is expected to provide to following:

i. The optimum percentage of GP used to partially replace cement content in mortar that will directly influence the construction industry economically,
ii. Environmentally friendly usage of waste glass in concrete structures, and
iii. Contribution of waste glass in the strength development and durability of concrete.

EXPERIMENTAL WORK

In this work, 0%, 10%, 20% and 30% of ordinary Portland cement was replaced with GP in a mortar mix (Leman et al., 2017, Lam et al., 2007). Twenty-four 50 mm × 50 mm × 50 mm specimens were cast for different GP proportions and their properties were compared with those of a mortar prepared without GP (control mix). The unit weight and compressive stress tests were performed on the concrete specimens after 7 and 28 days of curing.

Materials

Portland cement (type I), natural sand, water and GP were used in this study.

Cement

Ordinary Portland cement (Grade 42.5R) with a specific gravity of 3.15 was used in the experiment. The degree of fineness of cement referred to the mean size of the grains. The rate of hydration and hydrolysis as well as the consequent development of strength depend on the fineness of the cement. To obtain the same rate of hardening in different brands of cement, the cement fineness was standardised. Fine cement exhibits quick action with water and gains early strength as its ultimate strength remains unaffected. However, there would be an increase in the shrinkage and cracking of cement as the fineness of cement increases (ASTM C204, 2004) as shown in Equation 1 and 2.

$$S = Ss * \frac{\sqrt{T}}{\sqrt{Ts}} \tag{1}$$

where S is the specific surface of the test sample (cm^2/g), Ss is the specific surface of standard cement (cm^2/g), T is the time interval of the manometer drop for the test sample (s) and Ts is the time interval of the manometer drop for standard cement (s).

$$S = 486 * \sqrt{54} = 3571.4 \text{ cm}^2/\text{g} \tag{2}$$

Water

Tap water, potable without any salts or chemicals, was used in the study. The water source was the soil and material laboratory in the Islamic University of Gaza.

Natural fine aggregates

As defined in ASTM C33-03, the grades of the fine aggregates were determined according to sieve grading. Natural fine aggregates were tested for their physical properties. The absorption was 2.26%, the dry unit weight was 1610.01 kg/m^3, the specific gravity of fine aggregate was 2.50 and the fineness modulus of the fine aggregate was 2.90.

Glass powder

Waste glass was obtained from the recycling glass industry in Portugal, where waste glass (such as car windscreens) is crushed and sold to the bottle industry. The waste glass was grounded until it exhibited the same fineness as that of cement. The specific gravity of GP was 2.45. The glass was ground until it passed a sieve of 75 µm.

The waste glass was prepared in two stages as follows:

Stage I: Glass samples were ground using a Los Angeles machine for 6000 revolutions as shown in Figure 1.

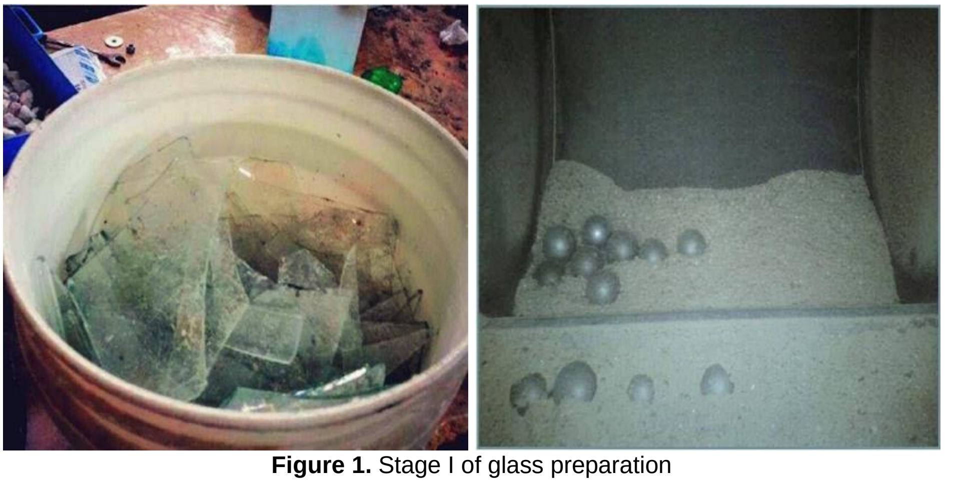

Figure 1. Stage I of glass preparation

Stage II is passed the sample through a sieve #200 as shown in Figure 2.

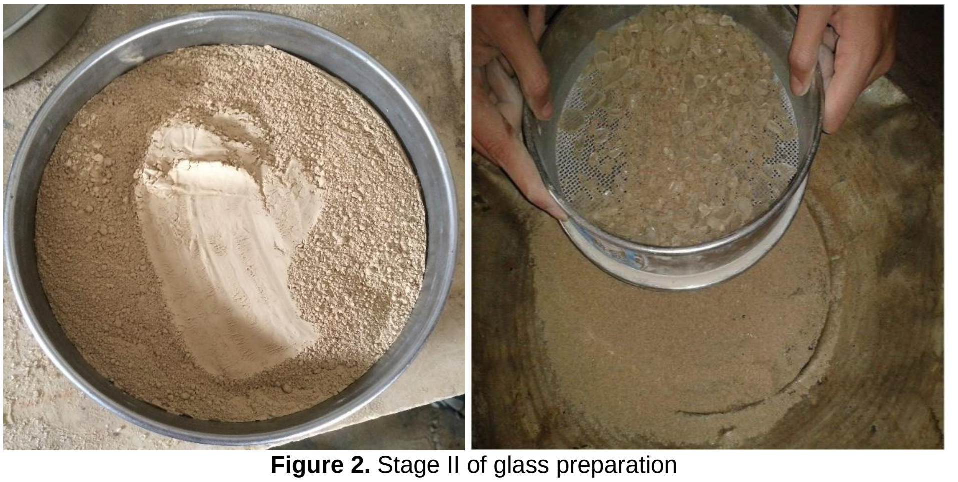

Figure 2. Stage II of glass preparation

The fineness of GP was measured using an air permeability test, according to Equation 1.

Mix Proportions

In this study, 0%, 10%, 20% and 30% of ordinary Portland cement was replaced with GP in a mortar mix. The components of the mortar had the following proportions (Table 1): cement, 2.75; sand, 0.64 and water as a reference mix. Figure 3 shows the preparation of mortar cubes and the compression test equipment (ASTM C109-88).

Table 1. Proportions of mix design results

Mix Number	Mix 1	Mix 2	Mix 3	Mix 4
	glass%			
	0%	**10%**	**20%**	**30%**
Material	weight(kg)			
cement	500	450	400	350
sand	1375	1375	1375	1375
GP	0	50	100	150
water	312	312	312	312
air content	0	0	0	0
total	2187	2187	2187	2187

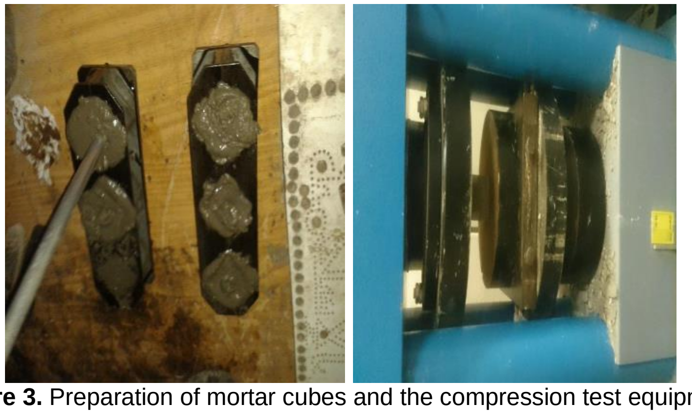

Figure 3. Preparation of mortar cubes and the compression test equipment

Results and analysis for GP cement mortars

Properties of mortar density

Density at 7 days

Table 2 shows the densities of mortar cube samples containing 0%, 10%, 20% or 30% GP. The overall density of the control samples cured for 7 days was 2.2 tonnes/m^3, which was nearly equal to that of the samples containing 10%, 20% or 30% GP. The test results indicated no significant reduction in dry density. This result is in agreement with previous studies by Taha and Nounu (2008, 2009).

Density of GP at 28 days

Table 3 shows the densities of mortar cube samples containing 0%, 10%, 20% or 30% GP. The overall density of the control samples cured for 28 days was 2.2 tonnes/m^3, which was nearly equal to that of the samples containing 10%, 20% or 30% GP. The test results indicated no significant reduction in dry density. This result is in agreement with previous studies by Taha and Nounu (2008) and Kumarappan (2013).

Compressive strength of cement mortars

Compressive strength at 7 days

Table 2 shows the average failure load of 23 kN for the control samples at the age of 7 days. The average failure loads of samples containing 10%, 20% and 30% GP were 20.3, 20 and 14 kN, respectively. The GP concrete samples cured for 7 days displayed lower compressive strength than the control samples. Their compressive strengths tended to decrease as the proportions of GP increased.

The reduction in compressive strength was 11.74%, 13.04% and 39.1% for samples containing 10%, 20% and 30% GP, respectively. This inclination may be due to the decrease in adhesive strength between the surface of the waste GP and the cement paste. In this study, the particle size of GP was larger than the particle size of cement. Porosity was the main factor affecting the compressive strength of mortar. This result is in agreement with studies by Shahidan et al (2017), Sadiqul et al. (2016), Irshad (2015), Idir and Tagnit-Hamou (2011) and Jain and Narayanan (2010).

Compressive strength at 28 days

Table 3 shows the average failure load of 37.3 kN for the control samples at the age of 28 days. The average failure loads of the samples containing 10%, 20% and 30% GP were 33.9, 31.7 and 25.4 kN, respectively. GP concrete samples at the age of 28 days displayed lower compressive strength than the control samples. The compressive strength tends to decrease as the proportions of GP increase.

The reduction in compressive strength was 9.1%, 15.0% and 32.9% for samples containing 10%, 20% and 30% GP, respectively. This inclination may be due to the decrease in adhesive strength between the surface of the waste GP and the cement paste. In this study, the particle size of GP was larger than the particle size of cement. Porosity was the main factor affecting the mortar’s compressive strength.

The compressive strength obtained in the present study is in agreement with the results obtained in previous studies which reported that the replacement of cement with glass results in a decrease in compressive strength. The decreased amount of calcium–silicate–hydrate (C–S–H) gel was due to the reaction between calcium hydroxide and pozzolanic mineral additions in the blended cement compositions (Habulat and Saman, 2013; Tayeh et al., 2013; Zeyad et al., 2016; and Zeyad et al., 2017).

Table 2. Density and compressive strength after a curing period of 7 days

Glass percent	Sample no	Dimension (mm)			Weight (g)	Density g/m^3	Failure Load (KN)	Average (KN)
		L	W	H				
0%	1	50	49	50	265	2.2	22	
0%	2	50	50	50	270	2.2	24	23
0%	3	51	50	50	270	2.1	22	
10%	1	50	50	50	265	2.1	18	
10%	2	51	51	50	275	2.1	22	20.3
10%	3	50	48	50	270	2.3	21	
20%	1	50	50	50	275	2.2	21	
20%	2	50	50	50	270	2.2	19	20
20%	3	50	50	50	270	2.2	21	
30%	1	50	50	50	270	2.2	14	
30%	2	50	50	50	270	2.2	14	14
30%	3	50	50	50	270	2.2	13	

Table 3. Density and compressive strength after a curing period of 28 days

Glass percent	Sample no	Dimension (mm)			Weight (g)	Density	Failure Load (KN)	Average (KN)
		L	W	H		g/m^3		
0%	1	50	50	50	273	2.2	37.7	37.3
0%	2	50	50	50	270	2.2	39.6	
0%	3	50	50	50	269	2.2	34.7	
10%	1	50	50	50	280	2.2	32.047	33.9
10%	2	50	50	50	275	2.2	35.243	
10%	3	50	50	50	270	2.2	34.617	
20%	1	50	50	50	270	2.2	29.252	31.7
20%	2	50	50	50	270	2.2	34.065	
20%	3	50	50	50	265	2.1	21.963	
30%	1	50	50	50	270	2.2	28.8	25.4
30%	2	50	50	50	270	2.2	26.1	
30%	3	50	50	50	275	2.2	21.3	

CONCLUSION

The effects of GP on the properties of cement mortar are as follows:

- The overall density of the 7-day control samples was 2.2 tonnes/m^3, which was nearly equal to that of the samples with 10%, 20% and 30% GP.
- The compressive strength of the samples containing GP tended to decrease with the increase in GP proportions. At the test age of 7 days, the compressive strength reduction was 11.74%, 13.04% and 39.1% for samples with 10%, 20% and 30% GP respectively.
- The overall density of the control samples cured for 28 days was 2.2 tonnes/m^3, which was nearly equal to that of samples with 10%, 20% and 30% GP.
- The compressive strength of the samples containing GP tended to decrease with the increase in GP proportions. At the test age of 28 days, the compressive strength reduction was 9.1%, 15.0% and 32.9% for samples with 10%, 20% and 30% GP respectively. This inclination may be due to the decrease in adhesive strength between the surface of the waste GP and cement paste.
- The production of GP cement mortar may potentially result in the reduction of emissions produced by cement manufacturing industries. Thus, GP is a potential substitute for cement. From the environmental perspective, GP is an eco-friendly and economical material. However, the durability of mortar containing different replacement ratios of GP requires further investigation.

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INFLUENCE OF GROUND COAL BOTTOM ASH WITH DIFFERENT GRINDING TIME AS CEMENT REPLACEMENT MATERIAL ON THE STRENGTH OF CONCRETE

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Abstract

This study was conducted to determine the strength of concrete containing coal bottom ash (CBA) as cement replacement material. The original CBA were sieved passing 150 um sieve size then ground for 20, 30, and 40 hours using ball mill machine. The normal concrete were designed for grade 30 based on Department of Environmental method (DOE). The concrete were produced by replacement level of cement 10%, 20% and 30%. The fresh concrete were tested using slump cone to determine the workability of concrete. It is found that slump height was decrease with increasing the replacement level of CBA. The concrete cubical with size 100 × 100 × 100 mm were prepared and tested to determine the compressive strength. It is shown that, the 20% replacement level of cement with 30 hours grinding time contributed high compressive strength compared to others. The presence of CBA in cement as a binder was improves the strength of concrete. It can be concluding that, CBA can be used as cement replacement material by appropriate replacement level.

Keywords: *Coal bottom ash; Compressive strength; Workability; Replacement material*

INTRODUCTION

In developing country like Malaysia, the coal generation power plant is gaining favour due to the growing need for energy and the call for a clean technology. In the electricity industry, gas remains the main fuel source for the generation industry, but coal is still become favourable because it is only viable fuel option in terms of cost and supply (Abubakar & Baharudin 2012). Coal bottom ash (CBA) is a waste material that disposed after process of electricity. According to Cheriaf et al. (1999) mention that, CBA qualifies the requirements of European standard EN 450 for use in concrete. A study carried out by Ghafoori and Bucholc (1997) revealed that durable concrete could be made with high-calcium CBA as fine aggregate. CBA is formed when pulverized coal is burned in a dry bottom boiler. The type of CBA produced depends on the type of furnace and also the origins of coal. From the burning process of coal, about 80% of the product were captured and recovered as Fly Ash (FA) meanwhile, the remaining 20% classified as bottom ash (BA).

CBA collected in water-filled hoppers at the bottom of the furnace is physically coarse, dark grey, granular and porous. At the wet bottom boiler, the CBA is kept in a molten state and collected when it falls into the ash hopper. Hence, high-pressure water jets immediately fractures the molten material into crystallized pellets, and it is referred as Boiler Slag. The other remaining combustion products go out along the flue gas chimney. Figure 1 shows the combustion of coal to generate bottom ash in a thermal power plant.

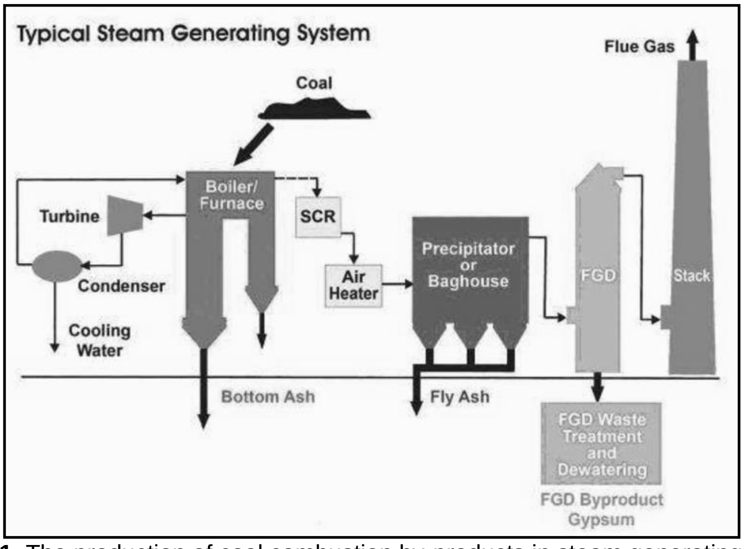

Figure 1. The production of coal combustion by-products in steam generating system (NETL, 2006)

The application of waste materials in concrete replacement has become popular an interesting topic in many research works. Concrete comprising three major substitutes which are cement, fine aggregate and coarse aggregate. Cement plays a vital role in concrete production. Its production is extremely energy-intensive and leads to excessive pollution, including SO_2 and CO_2 emissions (Azmi et al. 2017). The current cement production rate is approximately 1.2 billion tons/year and expected to grow about 3.5 billion tons/year by 2015 (Adesanya and Raheem, 2009). It is also the heaviest pollution industry and accountable for the 5–6% releases of CO_2, which causes about 4% of global warming (Rodrigues and Joekes, 2011). Such an enormous utilization of concrete calls for higher use of natural aggregates and cement, thus taking a toll on the environment. At least three-quarters of the total volume of concrete consist of coarse and fine aggregates (Rafieizonooz et al. 2016). Obviously, natural resources such as river sand are getting depleted (Aghabaglou, Tuyan and Ramyar, 2015). The prohibition on mining in some areas and the growing need for natural environment conservation further exacerbate the problem of river sand availability. Although aggregates are available at relatively low costs in most locales around the globe, availability is not universal. Some areas may be devoid of good quality sand and gravel, and a few others lack sources of crushed stone or rocks that can be mined easily and economically (Ramzi Hannan et al. 2016; Khalid et al. 2017). Furthermore, aggregates available locally may be unsuitable for use in construction due to poor quality, potential for chemical reaction and low strength.

Therefore, one of the most promising materials for replacing cement and sand material is coal bottom ash (CBA). CBA is one of the mineral by-product obtained from the combustion of coal used for power-generation purposes. CBA collected from electrical power generation is a dark grey, granular, porous and predominantly sand size material. These porous surfaces and very rough particle structure of CBA makes this material less durable (Bajare et al., 2013). Many research done use CBA as fine aggregate replacement as reported by Abidin et al.

(2017) and Torkittikul et al. (2017). Hence, with adequate grinding, CBA with pozzolanic activity can be used as a replacement material in Portland cement production (Cheriaf et al., 1999). Although grinding of CBA adds many good properties to concrete, it should be kept in mind that grinding is an expensive and time consuming process (Őzkan et al. 2007). Besides that, the quantity of coal bottom ash (CBA) to replace the cement for typical application is limited to 15-20% by mass of the total cementitious material. The small percentage is beneficial in optimizing the workability and low cost but it may not improve the durability (Suhendro, 2014).

The objective of this study is to determine the optimum replacement level of ground CBA in normal concrete. This study covers the physical properties of ground CBA and compressive strength of concrete incorporation with ground CBA as cement replacement material. The finding of these study are for further investigation on the use of ground CBA in different mix design grade.

MATERIAL AND METHOD

Ordinary Portland Cement (OPC) was used in this study supplied by Tasek Corporation Berhad and were certified by SIRIM compliance to MS EN 197-1:2014. The cement was stored in watertight drums to prevent unwanted solidification and humidity. The fine aggregates were used in this study is uncrushed river sand with 5 mm sieve passing. Meanwhile the coarse aggregate were sieve passing through 10 mm sieve size. The CBA that used in this study is a by-product from coal combustion process obtained from Kapar Energy Ventures power plant located at Kapar, Selangor. The original CBA physically dark grey and gravelly but after ground, the CBA become finer and the distribution size range is between 0.3 um and 40um.

Mix Proportion Design

The mixed design of concrete is very important to ensure the concrete grade design is successful. In this study, the mixed proportion were design according to British method of concrete mix design also called DOE method. The mixtures are specified by the weights of the different materials contained in a given volume of fully compacted concrete. It is assumed that the volume of freshly mixed concrete is an equal to the sum of the air content and of the absolute volumes of its constituent materials. Therefore, the DOE method requires that the absolute densities of the materials be known in order that their absolute volumes may be calculated. In DOE method, it is assumed that the strength of a concrete mix depends on free water/cement ratio, coarse aggregate type and also cement properties. The mix proportion design for this study was tabulated at Table 1 below. The water cement ratio 0.5 was designed for this mixture. The specimen were prepared for four different volume fraction (0, 10, 20 and 30 %) of ground CBA with three different grinding time which is 20, 30 and 40 hours. The concrete cube size 100 × 100 × 100 mm were prepared and cured in wet condition for 7 and 28 days. The workability of fresh concrete were tested using slump cone and the slump value were recorded.

Table 1. Mix proportion design by British DOE method

Cement (kg/m^3)	Water (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)
460	230	958	722

Particle Size Distribution

The particle size of ground CBA were conducted using Fritsch Analysette 22 after grinding process. The analysis method is laser light scattering technique where measuring range between 0.01 to 2100 µm. The wet method analysis were carried out where the ground CBA was dispersed in distilled water on the machine and take 5 to 10 second for measuring time. The testing has been conducted for all 3 different grinding time which is 20, 30 and 40 hours to identify the change of size after grinding process.

Specific Surface Area

These results of surface area were obtain from testing particle size analyser using Fritsch Analysette 22. The testing is important to analyse different fineness of ground CBA with different grinding time. Small amount of ground CBA was placed onto machine and directly measured.

Compression Test

The concrete cube specimen size (100 ×100 ×100 mm) were tested using 3000 kN hydraulic compression machine. The testing were conducted according to BS EN 12390-3: 2009. The loading rate during testing was constant within range 0.6 ± 0.2 MPa/s. The testing were carried out for 7 and 28 days curing period. The maximum load were record in N and compressive strength were expresse by Equation 1 below.

$$f_c = \frac{F}{A_c} \tag{1}$$

where:

f_c compressive strength in MPa (N/mm^2)
F maximum failure load in N
A_c cross sectional area of specimen in mm^2

RESULT AND DISCUSSION

Particle Size Distribution

Particle size distribution (PSD) analysis is important when particles have high aspect ratios. PSD analysis are susceptible to large uncertainties due to measurement the smallest. In this study, the ground CBA was graded using PSD analyzer. The finer particle will increase the specific surface area of ground CBA then increased the pozzolanic activity in concrete (Basirun et al.,2017; Hannan et al. 2017).

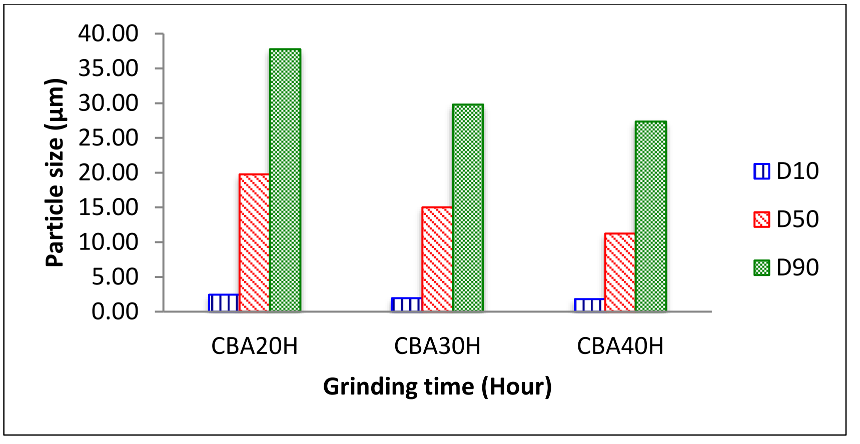

Figure 2. PSD analysis result for various grinding time of grinded CBA

Figure 2 shows, the particle size was decreased when the grinding time increases. The smallest particle size is at 40 hours grinding time, which is 1.819 µm, 11.254 µm and 27.356 µm at D10, D50 and D90 respectively compared to the others. While the narrowest particle is at 30 hours and coarsest particle are at 20 hours grinding time. The narrowest PSD value for D10, D50 and D90 is 1.948 µm , 15.036 µm and 29.807 µm respectively. Meanwhile, the coarsest PSD value for D10, D50 and D90 is 2.467 µm, 19.774 µm, and 37.749 µm respectively.

In general, pozzolanic reactivity of coal ash is directly related to its fineness. The filler effect of these fine pozzolanic materials is partly responsible for the increasing strength of concrete. As reported by Cheriaf et al. (1999), the pozzolanic activity of the CBA can be improved with adequate grinding. Thus, the strength will increase with decreasing particle size (Walker and Pavía, 2010). Hence, the finest of CBA having a greater contribution towards the strength of concrete.

Specific Surface Area

Specific surface areas of ground CBA directly obtained from Fritsch Analysette 22 machine and the results are shown in Table 2. The results shows the specific surface area of ground CBA become larger when grinding time increased to 30 hours and 40 hours. The specific surface area of 20 hours, 30 hours and 40 hours grinding time is 9627.76 cm^2/cm^3 , 12921.92 cm^2/cm^3 and 13528.18 cm^2/cm^3 respectively. The specific surface area ground CBA for 30 hours increased about 1.34 times while 1.40 times for 40 hours compared to 20 hours. This finding is very important to identify and understand the physical characteristic of ground CBA after grinding process. This outcome is similar with Khalid et al. (2016) on their report when they grinded POFA.

Table 2. Specific surface area of ground CBA

Parameter	20 hours	30 hours	40 hours
Specific surface area	9627.76 cm^2/cm^3	12921.92 cm^2/cm^3	13528.18cm^2/cm^3

Workability of Concrete

Slump test of fresh concrete was performed to ensure its consistency in that specific batch. The result for slump were shown in Figure 3. The workability of concrete was decrease with increasing the replacement level of grinded CBA. Increasing the replacement of grinded CBA will increase the specific surface area thus required more water demand to become workable. Kim and Lee (2011) also proved that black spots appeared on the surface of the fresh concrete with bottom ash due to suction of the water and cement paste by bottom ash. This factor also contributed towards lowering in slump value.

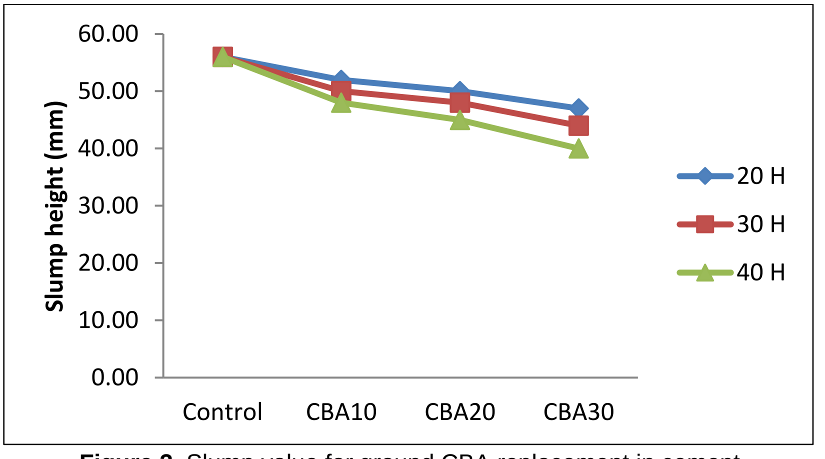

Figure 3. Slump value for ground CBA replacement in cement

Moreover, the workability of the fresh concrete for 40 hour grinding time is the lowest among the others. The slump decrease about 33% compared with control sample. As the grinding time increase, the slump value also linearly decreasing. It indicated that, the fine particle of the ground CBA are also one of the factor that contributes to decreasing the slump value. Finer particles require more water to wet their larger specific surface thus resulting in lowering the slump value. A fixed water cement ratio, the hydration process will result porosity in concrete occurs due to insufficient water demands. As reported by Abidin et al. (2014), the decrement of slump is because of the porosity of coal bottom ash, which absorbed more water with higher content of CBA replacement. Thus, increasing the water cement ratio, it will drop the strength of the concrete. The water cement ratio give major influence on designing concrete with CBA (Hamzah et al., 2016).

Compressive Strength

The compressive strength of concrete containing ground CBA as cement replacement material for 20 hours grinding time are shown in Figure 4. The concrete with 10% replacement level shows the highest compressive strength (37.90 MPa) compared among others replacements. Compressive strength start decrease at 20% and 30% replacements level about 1.5% and 4% respectively compared with control sample.

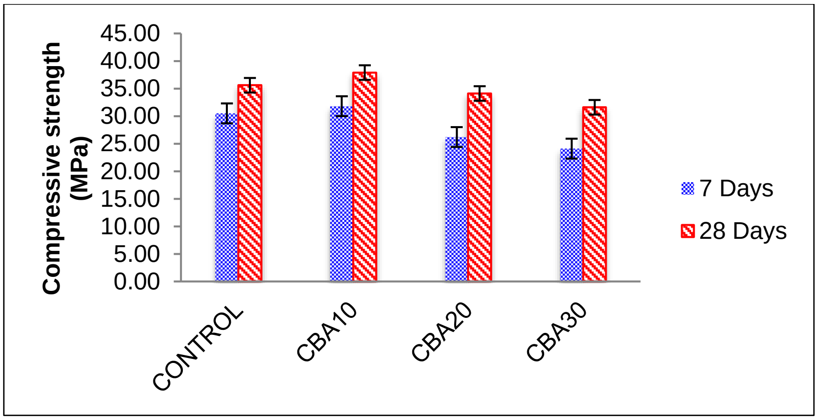

Figure 4. Compressive strength of concrete containing various CBA percentage for 20 hours grinding time

The reduction in strength is mainly considered due to the increasing replacement level of bottom ash that producing more porous concrete with more pores distributed around the bottom ash aggregate surface (Abidin et al. 2014),. The result were similar as reported by Ghafoori and Bucholc, (1996), that due to high water absorption rate, angular shape and very porous surface of the bottom ash, the strength of the concrete were decreased as the amount of replacement CBA increase, thus higher water content is required to achieve the degree of lubrication needed for a workable mixtures. Hence, it can be concluded that by 10% of 20 hours grinding time of CBA will present the higher strength value with small amount of CBA replacement.

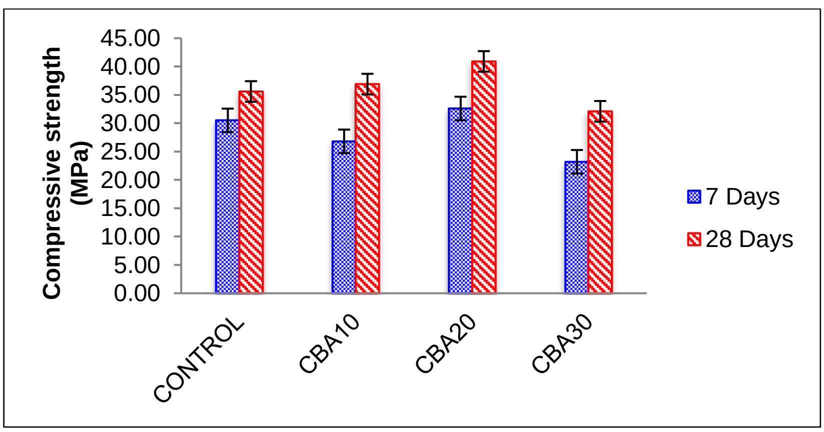

Figure 5. Compressive strength of concrete containing various CBA percentage for 30 hours grinding time

Figure 5 shows, the compressive strength of concrete containing various ground CBA percentage for 30 hours grinding time. The compressive strength value was rise up from 36.9 MPa at 10% replacement level to 40.9 MPa at 20% replacement level then dropped to 32.1 MPa in 30% CBA replacement level. It is noted that the highest strength were recorded at 20% CBA replacements with 15% increment compared with control sample. This result was in line with research by Jaturapitakkul et al. (2003) found, the compressive strength increase by replacing 20% CBA in production of concrete.

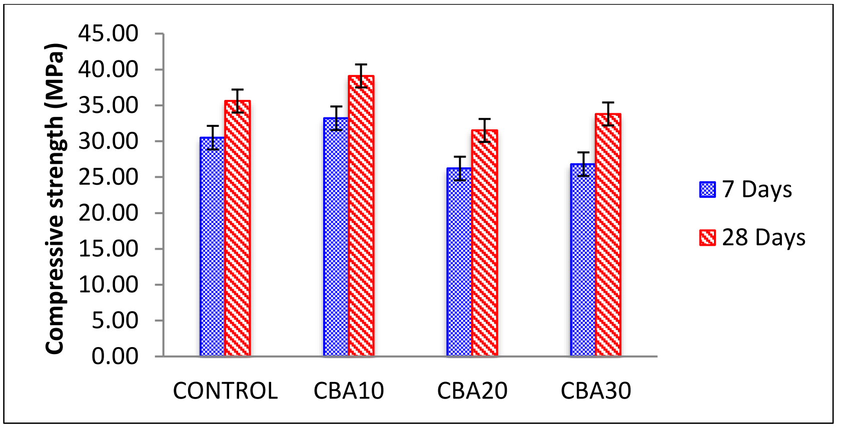

Figure 6. Compressive strength of concrete containing various CBA percentage for 40 hours grinding time

Figure 6 shows, the compressive strength of concrete containing ground CBA for 40 hours grinding time. The result shows, highest compressive strength obtained at 10% replacement level compared with control. Then, the strength was dropped at 20% replacement level and turn rise up at 30% replacement level. The 10% CBA replacement is the optimum CBA replacement as expected as it obtained the higher strength value for 40 hours grinding time. Based on PSD analysis conducted, the sample of CBA subjected to 40 hour grinding time is the finest particles among the others. The fineness of CBA is important to ensure a dense microstructure when hardened (Basirun et al., 2017). Chengzhi et al. (1996) have concluded, the addition of pozzolanic material influences the packing state and decreases the amount of filling water. This role depends on the fineness of pozzolanic materials.

In addition, finest particles of ground CBA will result high water demand. It can be seen when the slump height was decrease with increase of ground CBA replacement. This situation have proven that the CBA absorbed water faster during fresh state. Based on the discussion, 20% CBA of 30 hour grinding time is the selected replacement level as optimum CBA replacement in cement content. It is due to the high strength concrete grade value obtained besides minimal grinding time requirements. As mentioned by Pan et al. (2008), annual cement production in Taiwan is 20 million tons. Hence, by utilizing up to 20% CBA replacement in their cement production, the amount of cement production will be reduce by almost 4 million tons per year. Even though the contribution seems small, it is still considered as sustainability approach in reducing the amount of pollution occurred effect from cement production industry.

CONCLUSIONS

This paper has been discussed about the influence of fine coal bottom ash as cement replacement material in normal concrete. There are some conclusions that have been made:

(1) The different grinding time period were changed the physical characterictis of granular CBA to fineness particle. The size of particle become finness after grinding process and increase the specific surface area of ground CBA.
(2) The ground CBA have affecting the workability of concrete at fresh state. Workability of concerete were decreased linearly with increament of grinding time period. It

related to change of granular CBA to finness particle which specific surface area was increase and required more water to become workable.

(3) The normal concrete incorporation with ground CBA about 20% replacement level of cement at 30 hours grinding time shows the highest compressive strength compared to others. It is because of finness particle of ground CBA can reduce pore structure in concrete and produce dence packing structure.

(4) Ground CBA is cementitious material that can be used as supplementary cement material in concrete. The additional finness particle of CBA in concrete will increasing packing factor which helps minimise the voids and continues pozzolanic activity in cement matrix.

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THE EFFECT OF ASPHALT DUST WASTE ON THE RHEOLOGICAL PROPERTIES OF SELF COMPACTING CONCRETE

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Abstract
Development of Self Compacting Concrete (SCC) is significantly important in the construction industry while it greatly involved in recycled of powder waste material in improving rheological properties. Utilizing powder waste produced from asphalt batching plant so called Asphalt Dust Waste (ADW) was an alternative solution to reduce this waste material that cause pollution to surrounding dumping area. The aims of this study were to evaluate the optimum percentage of ADW as filler in coarse aggregate and the effect of ADW as cement replacement in develops SCC. The investigation focus on rheological performance at fresh state that was subjected to slump flow diameter, J Ring test, L Box and V Funnel test. Meanwhile, the hardened test was subjected to compressive strength and tensile strength at 7 and 28 days only. Difference percentage of ADW are 0%, 10%, 20%, 30%, 40% and 50%, has been incorporated into course aggregate by mass as a filler. The results shows 30% ADW as filler give the best performance on fresh state SCC. Base on 30% ADW filler, the potential of ADW as cement replacement at 0%, 10%, 20%, 30%, 40% and 50% were studied. According to the results obtained, utilizing ADW as filler and cement replacement in develop SCC was benefit for high workability, stability and segregation resistance in fresh state conditions. The results shows that 30% ADW as filler to aggregate content having better fresh performance, 10% ADW as cement replacement having 63 MPa and 50% cement replacement achieves minimum target of compressive strength which is 30 MPa. As conclusion, the utilization of ADW for the production of self compacting concrete is perfectly possible and also potentially reducing amount of cement in concrete mixture with high workability.

Keywords: *Self compacting concrete; Powder filler; Cement replacement; Asphalt dust waste*

INTRODUCTION

Concrete is the most commonly used in construction materials. Normal vibrated concrete is the common materials used in infra structure and building constructions. However, normal vibrated concrete create vibration impact to the workers during placing the concrete in long term effect. This given the importance reason to the industry in search for more efficient technologies and optimized resources (Mashitah et al., 2009; Ramli and Dawood, 2010; Martín et al,. 2013; Ibrahim et al., 2015; Ramzi et al., 2016; Senin et al., 2016; Bunnori and Jamil, 2016; Shahidan et al., 2016; Khalid et al., 2017). Self compacting concrete (SCC) was the latest technology introduce in 1988 in Japan to achieve durable concrete due to the shortage of skill workers in placing the concrete at construction site. It is not only provide durable concrete but also improve rheology properties in term of workability, stability and segregation resistance (Okamura and Ouchi 2003; EFNARC, 2002; Razak et al., 2008; Nuruddin et al., 2013; Ernida et al., 2017). The used of SCC have been extended around the world due the advantages in fresh state and high performance of hardened concrete. In fresh state condition, this concrete having high workability and stability to flow into the formwork by it own weight without segregation even in highly reinforce concrete structures. Beside that it give freedom of design work, reduce construction time and minimize workers on site that indirectly contribute to benefit overall cost (Silva et al., 2016). This type of concrete allows the used of waste materials from industrial or construction as a binder or filler. This is a part of contribution to reduce waste on the environment (Ismail et al., 2016; Tennich et al., 2017; Da et al., 2017).

There are many type of alternative waste powder in development of SCC. The most common addition materials are Fly Ash (FA), Silica Fume (SF), ground granular blast material (GBFS), Lime Stone powder (LS) and Rice Ash (Aruntas, 2014; Rahman et al., 2014; Saleh Ahari et al., 2015; Zhu et al., 2016; Benjeddou et al., 2017; Mohammed et al., 2017; Da et al., 2017). Until today, the revolution of utilizing waste powder as filler material in SCC constantly evolving. Researchers are looking for potential and economic waste materials that can perform in fresh state and hardened SCC.

Waste production from asphalt premix batching plant especially in sedimentation pond or wet site disposal is a major issue towards the environmental nowadays. This mineral ash wastes are come from surface aggregate that have been trapped using water sprinkler during production of bituminous or asphalt mixture and end up in sediment pond in premix batching plant area. It is namely call as Asphalt Dust Waste (ADW) as an alternative powder mineral for further research in development sustainable self compacting concrete. As conclusion, this mineral powder is not related with a fly ash that was produce from coal burning process in generator power plant (Acar and Atalay, 2016; Shahidan et al., 2017; Ismail and Shahidan, 2017). In Spain, the similar waste ends up in landfills in large amount. It is estimated about 4% of the asphalt mixture production produce ADW at asphalt mixture batching plant. It is abundant material that cannot be used for anything. Therefore, the potential of using ADW were explored and investigated for fresh state and hardened state of SCC by a group of researchers from Spain. It was found that, the utilization of this mineral powder as a filler was successful but depending on mix design ratio. Further more, this mineral waste are mostly a limestone characteristic according X-ray analysis and it is proven utilizing mineral waste with high incrustations of calcite will present satisfactory results compare with others commercial filler (Martín et al., 2013). In line with this study, another researchers from United Kingdom and Cyprus also using similar powder to produce normal vibrated concrete and not for self compacting concrete. In 2010, it is estimated the amount of powder waste in Cyprus alone was 230,000 tonnes. It is also being disposed in landfills and ends up create problem to the environment. Base on X-ray diffraction this dust having characteristic of limestone powder whish was Calcium Oxide (CaO). This research focuses on potential using recycled lime stone powder (RCL) as cement replacement ranging from 5% to 20% in mortar mixture. The increase in cement replacement has reduced the average compressive strength of each mix. Moreover, the 5% and 8% cement replacement in concrete mixture also shows reduction in compressive strength (Kanellopoulos et al., 2014). Therefore, the exploration for development of SCC might be useful to fully utilized amount of ADW to reduce the environmental impact and produce economic sustainable SCC for better future.

This review give a detailed information on previous research that using almost similar by product powder waste material generated from process of asphalt premix production in production of normal vibrated concrete (NVC) and self compacting concrete (SCC). However, there are very limited researches available on utilizing ADW in self compacting concrete especially utilizing local ADW in Malaysia. Therefore, the potential of recycled waste in production SCC such as ADW are need to be explored for the benefit of economic and environmental issue. The variety of mix design for SCC due to various constituent materials ratio also one of the reason to continue with this research. Furthermore, the effect of ADW in fresh properties are also important to be understand in order to set the water binder ratio in each mixture to avoid bleeding to fresh SCC (Su et al., 2001; Domone, 2006; Alyamaç & Ince, 2009; Md Nor et al., 2011; Parra et al., 2011; Khaleel & Abdul Razak, 2014; Thanh et al., 2015; Leman et al., 2016)

Therefore, the aim in this study is to simultaneously explore the potential of using local ADW in SCC as a filler material and cement replacement to archive minimum 600 mm slump flow. In details, the rheology properties in fresh state have been evaluated according to flow ability (slump flow diameter, t500), blocking resistance (J Ring test, L Box test) and resistant to segregation (V Funnel) through out 12 of SCC mixtures. Furthermore, the effect on the hardened properties focus on compressive strength and tensile strength was also investigated. As a result, the goal of this study is not only contributes to conforming the combination in optimization of ADW as filler and cement replacement in SCC but also introduce the efficiency to indicate the water binder ratio by utilizing ADW powder.

MATERIALS

This study was focus on obtaining a self compacting concrete by utilizing ADW as a filler material and potential as cement replacement. Local Ordinary Portland cement type 1 according to MS EN 197-1, CEM I 42.5 N was used in all mixtures. The granular materials are 4 mm passing fine aggregate (natural river sand) and maximum 12 mm course aggregate as shown in Figure 1. The ADW and 12 mm aggregate are from similar premix batching plant. The initial water cement ratio was 0.3 and using superplasticizer Sika ViscoCrete 2044 at 2% by weight from cement content.

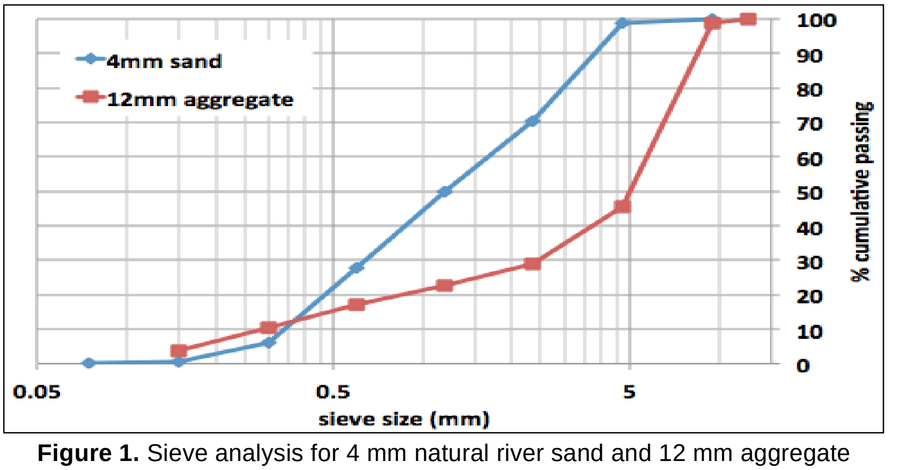

Figure 1. Sieve analysis for 4 mm natural river sand and 12 mm aggregate

In Figure 2, SEM image show the morphology of ADW. Obviously, the particles size of ADW is less than 10 µm and the physical properties almost similar with course aggregate material that is angular and irregular shape. Theoretically, powder less than 0.125 mm is suitable for improving packing factor and also influence in flow ability during fresh state of SCC (Felekoğlu et al., 2006; Corinaldesi and Moriconi, 2011; Singh et al., 2016; Mohammed et al., 2017). Therefore, the performance of ADW in fresh state and hardened state are important to be investigated.

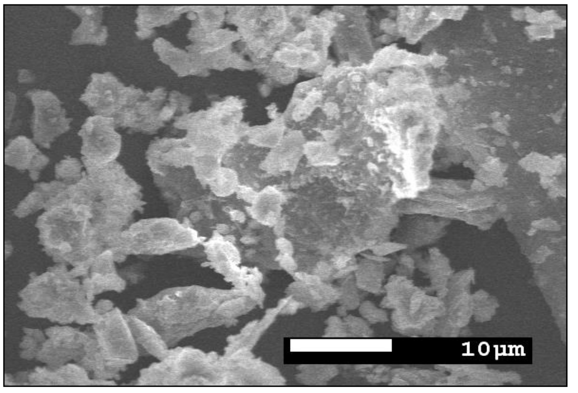

Figure 2. SEM image of ADW

The characteristics and chemical properties of constituent materials are important to be measured and identified in development of SCC. This information will lead to the expected outcome of fresh state condition and related to hardened performance. Therefore, the chemical properties of constituent material are measured using X-ray fluorescence (XRF) as shown in Table 1. It is clearly shown that ADW was not part of pozzolanic material compare to OPC cement and limestone powder that was obtain from previous researchers from Spain and Cyprus (Martín et al., 2013; Kanellopoulos et al., 2014). Eventhough, the powder was obtained from similar concept by product waste but the characteristics of the powder are totally depending on the type of the aggregate used in premix batching plant. The local powder (ADW) is having similar chemical characteristic with 12 mm aggregate used in this research. This similarity confirmed that the source of ADW was surface dust that was blown out from the aggregate during production of premix mixture and trapped in retention pond. As conclusion, this local powder was nothing but it is the dust coated on the aggregate surface after fractured process during production of aggregate through blasting and crushing at stone quarry. Since, this local powder was not pozzolanic material, therefore it is part of challenge in this study to fully utilized this powder as filler and also make it as a paste for cement replacement.

Table 1. The chemical properties for constituent materials

Properties	OPC cement	ADW	Aggregate	River sand
Calcium Oxide, (CaO)	55.60 %	2.98 %	1.85	-
Silicon Dioxide, (SiO2)	14.20 %	44.00	43.70	30.10 %
Aluminum Oxide, (Al2O3)	3.68 %	9.22 %	8.50	2.13 %
Iron (III) Oxide, (Fe2O3)	3.49 %	4.24 %	3.38	0.30 %
Sulfur Trioxide, (SO3)	3.20 %	-	-	-
Magnesium Oxide, (MgO)	1.48 %	0.62 %	0.49	-
Carbon, (C)	1.00 %	1.00 %	1.00	1.00 %
Potassium Oxide, (K2O)	0.48 %	-	-	0.19 %
Zirconium Dioxide, (ZrO2)	-	-	-	0.47 %
Titanium Dioxide, (TiO2)	-	0.46 %	0.39	0.32 %

METHODS

The main objective of this study was to investigate the potential of ADW as filler material and cement replacement in SCC. The ADW was collected from dumping area that was taken out from retention pond at local premix batching plant. The ADW have been dried using conventional method, which refer to dry under the sun for one day to remove moisture content. This is very important process to make sure the ADW totally became powder material with is suitable for development of SCC. Since, there is no special treatment applied in this process, it make ADW powder are economy and sustainable to be recycled in concrete material. Besides that, the powder also less than 0.125 mm and make it suitable for paste material in SCC.

The characteristic for fresh state SCC was refer to (EFNARC, 2002) and cluster according to (EPG, 2005) as shown in Table 1.

Table 1. Requirement for fresh properties

Fresh test	**Acceptance criteria**	
	EFNARC	EPG
Slump flow	650 – 800 mm	520 – 700 mm / class SF1 640 – 800 mm / class SF2 740 – 900 mm / class SF3
T 500mm slump flow	2 – 5 sec	≤ 2 sec / VS1 > 2 sec / VS2
J Ring	0 – 10 mm	-
V Funnel	8 – 12 sec	≤ 8 sec / class VF1 9 – 25 sec / class VF2
V Funnel T 5min	11 – 15 sec	-
L Box	H2/H1 = 0.8 to 1.0	H2/H1 ≥ 0.75 / class PA1 H2/H1 < 0.75 / class PA2

The dosage of constituent materials was studied from a method suggested by previous researchers with modifications base on similarity of powder materials used (Okamura and Ouchi, 2003; Dinakar et al., 2008; Alyamaç and Ince, 2009; Gesoğlu et al., 2012; Singh et al., 2016). Local Ordinary Portland cement type 1 according to MS EN 197-1, CEM I 42.5 N was used in all mixtures. The granular materials used are 4 mm passing fine aggregate (natural river sand) and maximum 12 mm course aggregate. The initial water cement ratio was at 0.3 and using superplasticizer Sika ViscoCrete 2044 at 2% fix from cement content by weight.

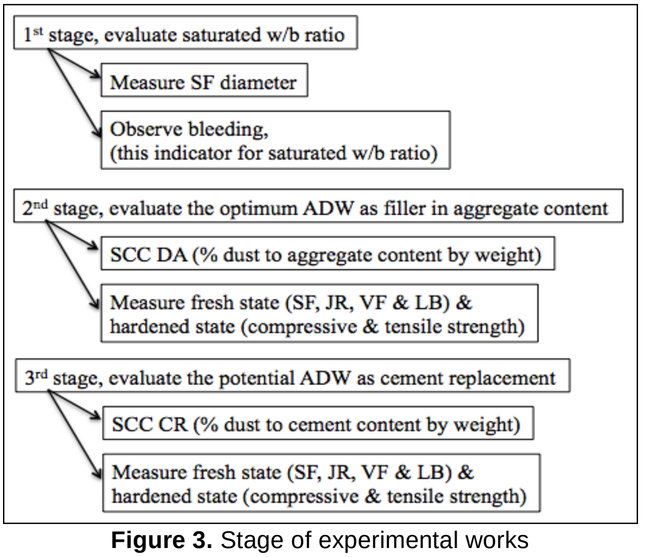

Figure 3. Stage of experimental works

This experimental work was carried out in three stages as shown in Figure 3. The first stage was conducted to evaluate the saturated water binder ratio on paste material by measuring the slump flow diameter and observe bleeding effect. The paste material is referring to amount of cement and ADW in mixture. The bleeding effect was the indicator for saturated water binder ratio. This result will be useful to estimate the amount of water demand in each SCC mixture. The amount of cement was fix to 10 kg in each mixture, 2% of SP by weight of cement and increase amount of ADW by accumulative 1 kg to the next mixture as shown in Table 2. The weight ratio (WR%) for cement and ADW will be used to estimate the water demand and the potential for cement replacement in stage 2 and stage 3.

The second stage consisted of the evaluation of ADW as filler material in course aggregate. The designation of the mixtures were SCC DA% at weight percentage 0%, 10%, 20%, 30%, 40% and 50% to aggregate content as shown in Table 3. The proportion of ADW was calculated base on weight percentage of course aggregate in the mixtures. This is to evaluate the influence of ADW to bond and carry the aggregate and sand in conjunction with flowability, stability and segregation resistance at fresh state SCC.

Table 2. Mix proportions for paste

Mixtures	Cement (kg)	ADW (kg)	Cement WR%	ADW WR%	2% SP (liters)
P0	10	0	1.0	0	0.2
P10	10	1	0.91	0.09	0.2
P20	10	2	0.83	0.17	0.2
P30	10	3	0.77	0.23	0.2
P40	10	4	0.71	0.29	0.2
P50	10	5	0.67	0.33	0.2

The third stage was continuity from stage two that was using the optimum 30% ADW as filler in SCC. This intent, 30% ADW in each mixtures as filler to achieve SCC slump flow base on evaluation from fresh and hardened performances at stage two. The designation of the mixtures were SCC DA30/CR% at weight percentage 0%, 10%, 20%, 30%, 40% and 50% as shown in Table 4. In this study, the ADW-CR was assumed to be part of cementitious material. Therefore, the amount of water was consistence even though the amount of cement was reducing in each mixture. However, the amount of SP is calculating base on amount of cement only. All mixtures are produced using vertical concrete mixer.

Table 3. Mix proportions for ADW as filler material (SCC DA)

Mixtures	Cement (kg)	Course Aggregate (kg)	Sand (kg)	ADW as filler (kg)	Water (kg)	2% SP (liters)
SCC DA0	400	785	865	0	120	8
SCC DA10				78.5	143.55	
SCC DA20				157	167.1	
SCC DA30				235.5	190.65	
SCC DA40				314	214.2	
SCC DA50				392.5	237.75	

Table 4. Mix proportions for ADW as cement replacement (SCC 30DA/CR)

Mixtures	Cement (kg)	ADW as CR (kg)	Course Aggregate (kg)	Sand (kg)	ADW as filler (kg)	Water (kg)	2% SP (liters)
SCC DA30/CR0	400	0	785	865	235.5	190.65	8
SCC DA30/CR10	360	40					
SCC DA30/CR20	320	80					
SCC DA30/CR30	280	120					
SCC DA30/CR40	240	160					
SCC DA30/CR50	200	200					

Once the self compactability of the mixes was obtained, the fresh properties of SCC were measured and evaluated. The details fresh properties test as shown in Figure 4. The slump flow test measured the slump flow diameter and time to reach 500mm diameter (t500) using Abraham slump cone apparatus. The J Ring test is similar to slump flow test but it having ring outside the slump cone to measure segregation resistant and stability of SCC. The V Funnel test is subjected to stability and also can be used to observe the segregation of SCC. Finally, the L Box test is to monitor and observe the blockage, stability and segregation resistance.

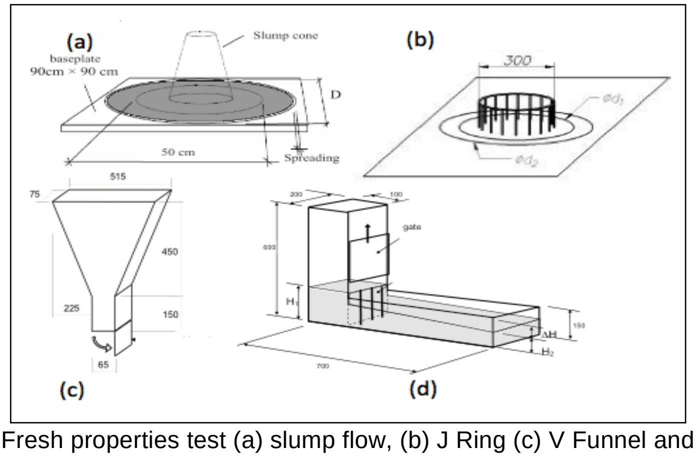

Figure 4. Fresh properties test (a) slump flow, (b) J Ring (c) V Funnel and (d) L Box

The compressive strength was measured according to BS EN 12390-3:2009 by means of 100 mm cube and tensile strength according to BS EN 12390-6:2009 by means of 100 mm diameter cylinder (200 mm high). Both hardened test are measured at 7 and 28 days only.

RESULTS AND DISCUSSION

In the first stage, the evaluation for saturated slump flow was measured in paste study by increasing w/b ratio into various percentages ADW as shown in Figure 5. The difference w/b ratios were test to the paste mixture to estimate the saturated w/b ratio for stage 2 and stage 3. As far as the characteristic fresh SCC parameter concerned, the slump flow value must achieve minimum 550 mm diameter.

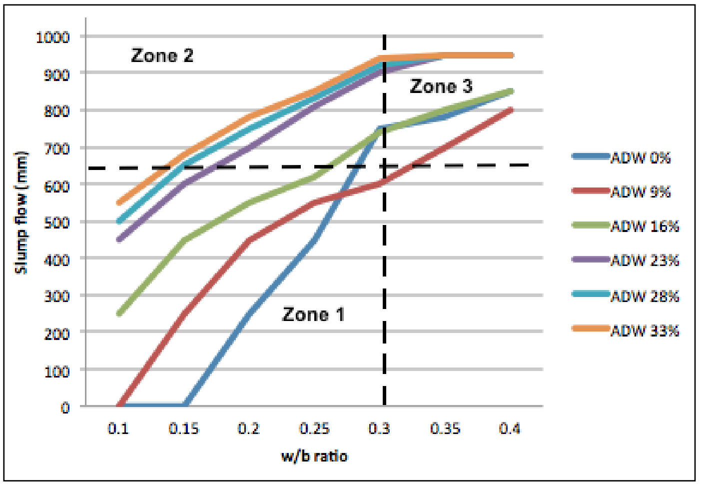

Figure 5. Effect of w/b ratio on slump flow in paste study

The slump flow diameter was clustered in three zones to estimate the w/b required before it turn to bleeding effect. In zone 1, paste using ADW 9% and ADW 16% achieve 550 mm diameter slump at w/b 0.25. However, these mixtures show high viscosity at low percentage powder, which is not good for stability of SCC. The time for 500 mm flow was too fast and less than one second. Therefore, the amount of ADW should be increase in stage 2 and stage 3 for better stability and economy benefit in paste and utilizing ADW as alternative powder in SCC.

Even though zone 3 shows better value of slump flow in paste study, it is better to consider saturated w/b ratio as 0.3 for stage 2 and stage 3 SCC. Throughout observation, it is found that w/b ratio more than 0.3 has clearly shown bleeding effect in all paste. Therefore, zone 3 can be classified as bleeding zone for 2% SP in various ADW percentages.

As conclusion, the suitable saturated w/b ratio was 0.3 before the paste start showing bleeding in mixtures using 2% SP. Interestingly, the paste studied shown better stability as increasing amount of ADW% through observation and fulfill self compactability requirements. Consequently, it shows potential result to increase ADW percentage in stage 2 and stage 3 studies. The saturated slump flow for Zone 2 was 750 mm diameter for 9% to 16% ADW and 900 mm diameter for ADW more than 20%. However, this saturated slump flow will reduced in SCC stage 2 and stage 3 due to water absorption from granular material.

According to stage 1, the final mixes were prepared for evaluation ADW as filler in aggregate for SCC in stage 2. The fresh properties of ADW as filler material are shown in Table 5. From the observation during fresh state, SCC DA30 have the best performance in term of filling ability, passing ability and stability (segregation resistance). The SCC DA30 has great flow ability (750 mm diameter), good passing ability as shown from J Ring and L Box test result. It is subjected to the ability of ADW as a paste binder to carry and transport the granular materials in gravity flows under their own weight. The shorter time flows indicated that the SCC DA30 having a low viscosity that makes this mixture easy to flow and passing the reinforcement without segregation (EFNARC, 2002). The most important thing, it is also having a good stability in V Funnel test, which mean the granular materials are not separated while transported in vertical direction and stable with friction effect from the V Funnel wall. Furthermore, it was confirmed that, there are no segregation to the mixture after passing the trapdoor and no segregation happened due to the impact of 500 mm vertical drop from the V Funnel opening trap door to the base floor.

Table 5. Fresh properties for ADW as filler material

Mixtures	Slump performance		J Ring	V Funnel		L Box		
	Flows	t500 (sec)		0 sec	5min	H2/H1	t20	t40
SCC DA0	450	-	-	-	-	-	-	-
SCC DA10	550/SF1	8/VS2	500	15/VF2	18/VF2	0.7/PA1	4	7
SCC DA20	650/SF2	5/VS2	600	10/VF1	12/VF2	0.9/PA1	2	4
SCC DA30	750/SF2	2/VS1	700	6/VF1	7/VF1	1/PA1	1	2
SCC DA40	650/SF2	4/VS2	630	9/VF1	13/VF2	0.9/PA1	2	4
SCC DA50	550/SF1	5/VS2	520	10/VF1	13/VF2	0.9/PA1	2	4

The development of hardened properties for compressive strength and tensile strength for stage 2 are presented in Figure 6. The increasing percentage of ADW as filler material in develops SCC has shown significant contribution in fresh and hardened properties. Even though SCC DA0 having slump flow diameter less than 550 mm but it is very high workability compare to normal concrete and it is under characteristic of high flow able (Elyamany et al., 2014). The SCC DA 30 achieves 57.9 MPa compressive strength, 4.03 MPa splitting tensile strength and meats the entire SCC requirement. Although the SCC DA20 reach higher compressive strength values after 28 days, it has less slump flow diameter and time flow more than 2 seconds. Furthermore, increasing percentage of ADW to 40% and 50% has shown decreasing of fresh properties especially in slump flow diameter but it is within acceptable range. As conclusion in stage 2, the 30% ADW was the optimum filler for maximum slump flow diameter and having better fresh properties in SCC with related to 0.3 w/b ratio and 2% SP to cement content. Therefore, the value 30% from 12 mm course aggregate content was the best for utilizing ADW as alternative filler material in SCC.

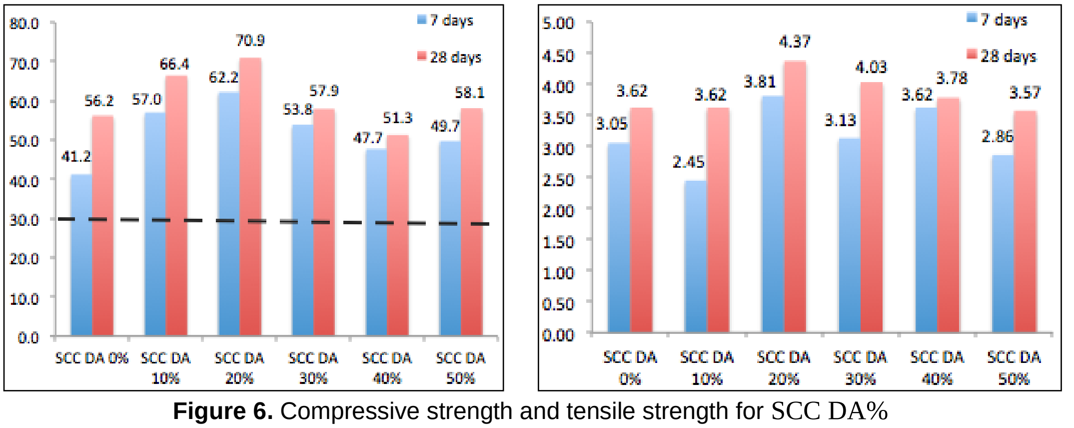

Figure 6. Compressive strength and tensile strength for SCC DA%

In stage 3, the fresh properties of ADW as filler material are shown in Table 6. The ADW has been maintained 30% as filler to aggregate content for 750 mm diameter slump flow and ADW was also design as part of cement replacement by weight. The SCC DA30/CR0 in stage 3 and SCC DA30% in stage 2 was the same proportions. It was found that, the fresh and hardened state for both mixtures is almost the same even though there are in difference mix times. This similarity performance has proven consistency of the mix design.

Table 6. Fresh properties for ADW as cement replacement

Mixtures	Slump performance		J Ring	V Funnel		L Box		
	Flows	t500 (sec)		0 sec	5min	H2/H1	t20	t40
SCC DA30/CR0	750/SF2	2/VS1	650	8/VF1	9/VF1	1/PA1	1	2
SCC DA30/CR10	750/SF2	2/VS1	650	9/VF1	9/VF1	1/PA1	1	2
SCC DA30/CR20	650/SF2	4/VS2	600	10/VF1	11/VF2	1/PA1	2	4
SCC DA30/CR30	650/SF2	5/VS2	600	10/VF1	12/VF2	0.9/PA1	3	5
SCC DA30/CR40	650/SF2	6/VS2	600	11/VF2	13/VF2	09/PA1	4	6
SCC DA30/CR50	650/SF2	6/VS2	600	11/VF2	13/VF2	0.9/PA1	4	6

The development of hardened properties for compressive strength and tensile strength for stage 3 are presented in Figure 7. The performance of ADW as cement replacement has drawn back for fresh and hardened state while maintaining w/b ratio 0.3 and 2% of SP from cementitious component. It has shown reduction of slump flow, compressive strength and splitting tensile strength proportion to increasing of ADW as cement replacement. However, the slump flow diameter is above 650 mm for all mixtures in this stage and almost similar with previous research (Uysal and Sumer, 2011; Vijayalakshmi et al., 2013; Singh et al., 2015). The reductions of slump flow were due to water demand from ADW in cement replacement. Base on previous study, increase w/b ratio will increase slump flow, influence bleeding, reduce compressive strength and reduce others SCC hardened performance (EFNARC, 2002).

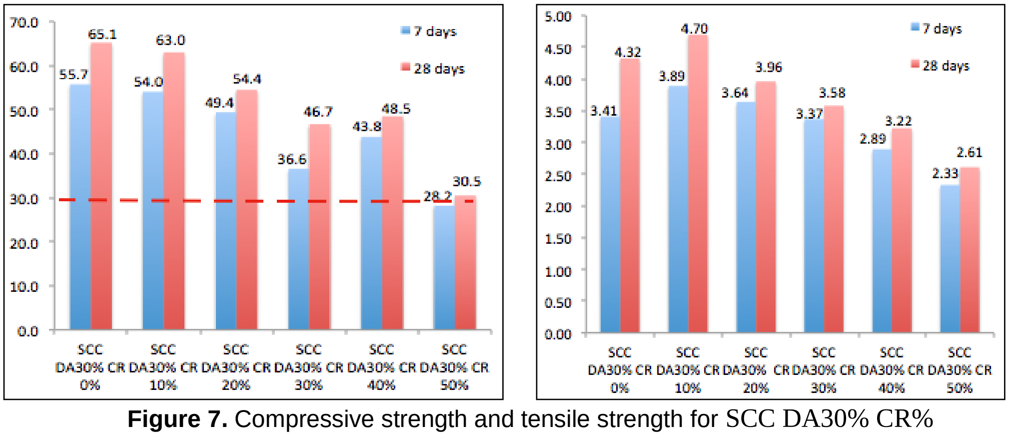

Figure 7. Compressive strength and tensile strength for SCC DA30% CR%

However, utilizing of ADW as cement replacement at low replacement percentage shows a good performance. Replacement 10% ADW has reduction of strength at 3.2%, saving 10% of cement content, compressive strength more than 60 MPa, 700 mm slump flow diameter and passing acceptable fresh properties requirement. Furthermore, 20% replacement was also benefit to up to 20% saving cement content with all fresh properties in acceptable range and compressive strength more than 50 MPa. Finally, utilization 30% cement replacement was also benefit to recycled waste in production of sustainable SCC compared with other recycled powder (Uysal and Yilmaz, 2011; Vijayalakshmi et al., 2013; Singh et al., 2016).

As conclusion, this result was satisfied when 50% cement replacement achieved 30 MPa compressive strength and 2.61 MPa splitting tensile strength at 28 days compared with other researchers (Kanellopoulos et al., 2014; Grabois et al., 2016; Alyhya et al., 2016). It is also fulfill the requirement for fresh properties with 650 mm diameter and was confirmed easy to flow and having good stability. In summary, all mixtures in stage 3 are in acceptable range for fresh properties. Finally, this is part of contribution to economy design for utilizing waste as filler and also part of cement replacement in production of sustainable SCC (Vijayalakshmi et al., 2013; Singh et al., 2016).

CONCLUSION

The aim of this work was to utilizing asphalt dust waste (ADW) as filler material into 12 mm course aggregate content and also introducing ADW as cement replacement for sustainable SCC.

It was found that the 900 mm slump flow of paste study has been reduce to 650 mm to 750 mm diameter in SCC stage 2 and stage 3. This reduction are within 27% to 17% depending on percentage of ADW in mixtures. The slump flow reduction was due to water absorption from granular materials during mixing process.

In filler studies, the optimum compressive strength for 28 days was 70.9 Mpa at 20% ADW with 650 mm slump flow diameter. On the other side of fresh state evaluation, the 30% ADW was the optimum slump flow for 750 mm diameter and compressive strength was 57.9 Mpa. Utilization 50% ADW as filler in SCC was succesfull achievement for such sustainable SCC utilizing by product waste.

In cement replacement studies, the 10% cement replacement shows very good potential in term of low range replacement. It is not only performed in hardened state but also in fresh state of SCC compared to control SCC DA30/CR0. However, increasing CR percentage will significantly reduce the hardened strength but it having good quality of fresh properties for general purpose and application.

ADW can be utilized as filler into aggregate content and also can be used as cement replacement. The use of powder waste from asphalt premix plant not only reduce environmental impact but also create cost benefit while ADW were not reused and disposed to the landfill previously. It is also an alternative and possible option to reduce natural resources and create economy saving as well.

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ALKALINE TREATMENTS ON EFB FIBRE: THE EFFECT ON MECHANICAL-PHYSICAL PROPERTIES AND FIBRE-CEMENT HYDRATION RATE

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Abstract

The natural fibres commonly used to enhance the brittleness of the cement matrix but appropriate fibre should be used for a particular purpose depending upon the type of fibre and characteristics. The Oil Palm Empty Fruit Bunch (EFB) fibre is one of the major crops in Malaysia, which contribute large scale of waste that is durable and make it reasonable for utilization in cement-based product. However, the presence of hemicellulose, lignin and extractive (oil, sugar and starch) affect the performance of EFB fibre and causes an incompatibility of EFB fibre and cement. Hence, this research is been conducted to explore the suitable proportion of Sodium Hydroxide (NaOH) treatment for EFB fibre to increase the compatibility of cement with EFB fibre. The NaOH concentration of 0.2%, 0.4%, 0.6%, 0.8%, 1%, 2%, 3% and 4% were used in this study as a chemical pre-treatment of EFB fibre for surface morphology observation and hydration rate test. Meanwhile for only untreated fibre, fibre treated with 0.4% (low concentration), 1% (medium concentration) and 4% (high concentration) of alkali treatment were tested for tensile strength of single EFB fibre. The fibre treated with NaOH has shown a significant different on the hydration temperature for EFB fibre-cement mixed compared with the untreated fibre. The higher NaOH concentration, the greater hydration temperatures obtain. The Scanning Electron Microscopy (SEM) image show that the increment NaOH concentration applied, the rougher EFB fibre surface is observed with lesser silica body remain. The tensile properties of individual fibre treated with NaOH (0.4%, 1% and 4%) has shown significant increment as compare to the untreated fibre with the highest tensile properties mean value 422.90 N/mm^2 at 4% NaOH concentration.

Keywords: *Empty Fruit Bunch; Alkaline Treatment; Tensile Strength; Hydration Rate; SEM*

INTRODUCTION

Recently, the natural fibres have received significant attention to be utilized as an alternative material for various industrial products. Fibre reinforcement of cementitious materials still remains an exciting and innovative technology because of the basic engineering properties of crack resistance, ductility and energy absorption that enhance the infrastructure construction (Swamy, 2000; Irwan et al., 2014). Nowadays, the researchers are investigating on new alternative sources of lignocellulose materials to replace previous fibres that are widely used in construction industry (Sanjay et al., 2016). The researchers has used plant fibres as an alternative source in composites products such as cement paste and concrete for increasing its strength properties (Ali, 2012). The natural fibres mainly used to enhance the physical and mechanical properties of the composite materials but appropriate fibre should be used for a particular purpose depend upon the type of fibre and characteristics. The oil palm empty fruit bunch (OPEFB) fibre is one of the major crop in Malaysia with a total plantation area of 4,304,914 hectar which contribute large scale waste such as dead fronds, empty fruit bunches (EFB), shells and chopped trunks (Goh et al., 2010). The OPEFB has high cellulose content which is durable and make it reasonable for utilization in bio-composites materials (Sreekala et al., 2004). The fresh EFB usually contains 30.5% lignocellulose, 2.5 % oil and

67% water (Ramli et al., 2002). It was noted by Chang (2014) that the component of EFB typically comprises of cellulose (23.7-65.0 %), hemicellulose (20.58-33.52 %) and lignin (14.1-30.45 %). The physical characteristics of EFB gives different values in terms of physical and mechanical properties of the bio-composite material (Shinoj et al., 2011).

The study indicates that the EFB fibre with cement could improve the performance and quality of the bio-composite materials. Whereas, many supplementary natural fibres, EFB fibres are naturally occurring composites consisting primarily of rigid, crystalline cellulose microfibrils which are embedded in a soft, amorphous matrix of hemicelluloses and lignin (Hassan et al., 2010). Furthermore, according to the Rozman et al. (2001), the oil residue from lignin contain in EFB fibre may affect the interaction between fibres and the coupling agents. This issue needs to be considered while using EFB fibre as construction material. The natural fibre cannot be used directly because of incompatibility between fibre and cement matrix due to the existence of residual oil that interrupts the penetration of binding agent and affect the properties of the final EFB-products. From the extensive literature review it was perceived that many researchers carried a series of tests to improve the compatibility of fibre and cement by using certain treatments. However, it was found that there is a gape of study on the treatment for EFB fibre while using as a replacement material with cement composites. Previous studies were conducted on the surface modification of EFB fibre (Ibrahim et al. 2015) to investigate the effect of NaOH treatment at different level of concentration on the morphological structure of OPEFB, which proved that chemical treatment using NaOH is the most appropriate method to remove the carbohydrate contain in the surface of EFB fibre.

Therefore, EFB fibre treated with different concentration of NaOH has been carried out in this research. To determine the suitable proportion of NaOH concentration for surface modification of EFB fibre as a replacement material for cement based composite products. The experimental test for this part consist of hydration rate test, surface morphology examination and tensile strength for single fibre. The correlation of hydration rate and surface morphology examination used to determine the suitable range concentration of NaOH for EFB-CB sampling. The individual EFB fibre that represent untreated fibre (UT), fibre treated with 0.4% (low concentration), 1% (medium concentration) and 4% (high concentration) of NaOH has been tested for tensile strength to obtain the correlation with related to hydration and surface morphology variations.

MATERIALS AND METHODS

The EFB fibre were supplied by Tereh Oil Palm Mill located at Kluang in Johor, Malaysia. Initially the fibres were treated with NaOH at different concentration by soaked for the period of 24 hours as suggested by Ibrahim et al. (2015) consequently, hand-washed and oven dried to obtain 5% fibre moisture contain. Concentrations of NaOH used for treatment were based on study by Ibrahim et al. (2015) as 0.2%, 0.4%, 0.6%, 0.8% and to provide the better understanding and establish the finding, additional NaOH concentration as 1%, 2%, 3% and 4% were introduced in this research.

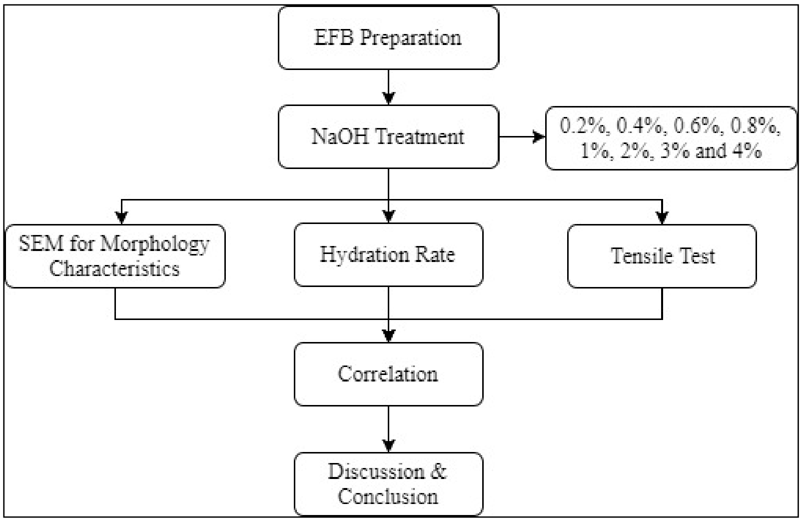


Chart 1. Research methodology flow

Surface Morphology Examination

In this study, the Scanning Electron Microscope (SEM) has been used to investigate surface morphology of EFB fibre. The EFB fibre used in this physical investigation consists of untreated EFB and EFB treated with NaOH with range concentration of 0.2% to 4% (as discussed in previous section). The SEM observations were conducted in the range of 200 to 500 magnifications to obtain the better view surface characteristic in order to gain deep understanding of the different effect at various concentration of NaOH. It was also found by Ibrahim et al. (2015) and Sreekala and Thomas (2003), that the use of SEM image of 200 to 500 magnification is appropriate to make the conclusion through observation changes of EFB fibre surface morphology due to variation treatment effect.

Heat of Hydration

The previous research has been done on the compatibility test in order to define the effect of natural fibre treated with NaOH with different concentration to the hydration rate of cement. This test is essential to determine the amount of NaOH, which can be used for EFB fibre treatment, the relationship can be judged between the change of surface morphology and the rate of EFB fibre-cement hydration. The heat of hydration test designed for this research were based on the method used by previous researchers (Ashori et al., 2011; Hermawan et al., 2001; Noor Azrieda et al., 2009). Subsequently, the different percentage of 0.2%, 0.4%, 0.6%, 0.8%, 1%, 2%, 3% and 4% NaOH concentration used to treat the EFB fibre. Control sample consists of cement (OPC type I) and the mixture of untreated EFB fibre with cement.

The Ordinary Portland Cement (OPC) of 300 grams were used in this test mixed with 15 grams of fine EFB fibre, then the mixture added with 130.5 mL of water. The water used for this test was fixed at 0.4mL/g of cement weight plus 0.7mL/g of EFB fibre weight (oven dry basis). The tests setup of hydration procedure a shown in the Figure 1. The mixture stirred for 2 minutes in polystyrene cup. After the mixing, thermal couple (type T) was immediately inserted approximately at the center core of EFB fibre-cement mixture and connected to the data logger (Midi Logger Graphtec GL220). The mixture then placed in the thermos flask. To provide better understanding of EFB-cement on hydration temperature and time taken to reach optimum temperature, the experiments were conducted for 65 hours in ambient room temperature.

Figure 1. Hydration test setup and recorded

EFB Fibre Tensile Strength

The tensile strength test for EFB fibre was conducted based on the ASTM standard (ASTM D3379) for single fibre. This test method covers the preparation test specimen (Figure 2), mounting specimen on testing jig, and apply the load to obtain the breaking load henceforth calculate the tensile value. The Instron Universal Testing Machine with a 10 kN load-cell was employed in this test. The EFB fibre used in this test were selected based on hydration temperature category and surface morphological change. There were four categories of EFB selected for tensile study; untreated EFB (UT) as a control sample and EFB treated with 0.4%, 1% and 4%. The single fiber was glued to a cardboard frame shown in Figure 2. The process of sticking/glue EFB on cardboard conducted with carefully by placing a small amount of adhesive (epoxy) at the marks on the mounting cardboard that define the gauge length. The test specimens were gripped/clamped to the load train so that the test specimen is aligned axially along the line of action of the test machine.

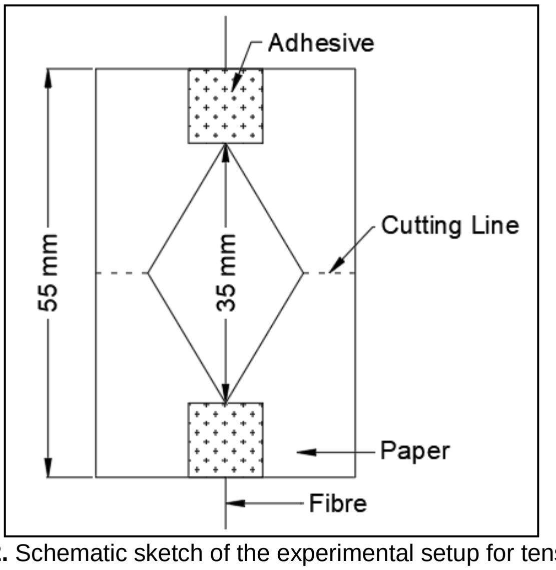
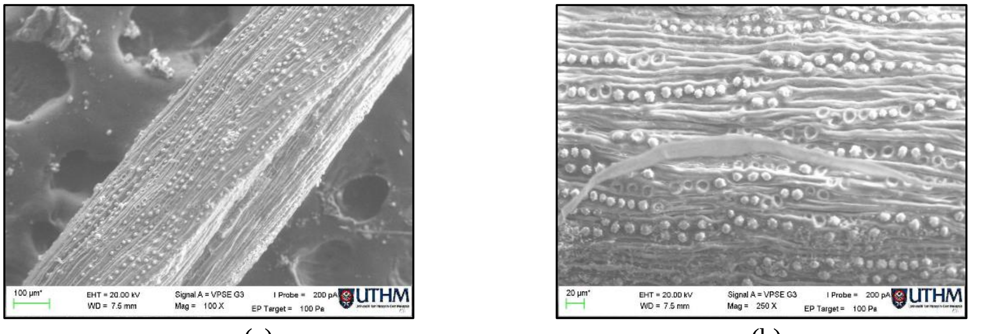


Figure 2. Schematic sketch of the experimental setup for tensile test

RESULTS AND DISCUSSION

Physical Properties

The scanning electron microscope (SEM) image with various magnification ranges from 200 to 500 was used to observe the change of surface morphology of EFB fibre before and after pre-treatment. It was found that untreated fibre clearly shown the presence of silica bodies embedded in great number on EFB fibre strand as shown in Figure 3a and Figure 3b. The silica attached to circular craters on the EFB fibre surface and spread fairly uniformly with the rounded spiky size of 10-14 μm. Whereas, the similar results was also obtained by Ibrahim et al. (2015), furthermore, Law et al. (2007) were also found the presence of silica bodies on EFB fibre strand with rounded spiky size distribution 10-15 μm.



(a) (b)

Figure 3. Mechanical performance and physical observation of EFB-CB fabricated from different length of EFB fibre

The OPEFB treated with NaOH revealed that the reduction in presence of silica body as shown in Figure 4a to Figure 4i. The OPEFB treated with NaOH at 0.2% did not show much different appearance of the silica body. However, after the treatment with NaOH started 0.4% of concentration, some of the silica body started removed from the EFB strand and leaving the effect of crater-shaped holes. The increasing concentration of NaOH used, the lower number of silica body remained as obtained by Ariffin et al. (2008), Ibrahim et al. (2015) and Sreekala et al. (1997) were found the reduction number of silica body on EFB fibre strand after treated with NaOH.

Whereas, at 1% NaOH concentration (Figure 4e), it could be seen that the silica body almost completely removed from the EFB fibre strand. However, the presence of silica bodies was not fully decomposed and the residues still remain on the fibre strand. When the NaOH concentrations reach at 2% (Figure 4f), the presence of silica body completely removed and decomposed. Figure 4g showed that EFB fibre strand that treated with NaOH at 3% concentration. It is found that, the surface of EFB fibre strand turn more rough and uneven. Crater-shaped hole on EFB fibre strand started damage and deformed at 4% concentration of NaOH as shown in Figure 4h. According to Sreekala et al. (1997), the roughness surface of fibre strand due to alkali treatment could enhance the mechanical interlocking at the interface. However, the comprehensive discussion related to the EFB fibre treatment and change of surface morphology to the cement setting very rare to find publish elsewhere.

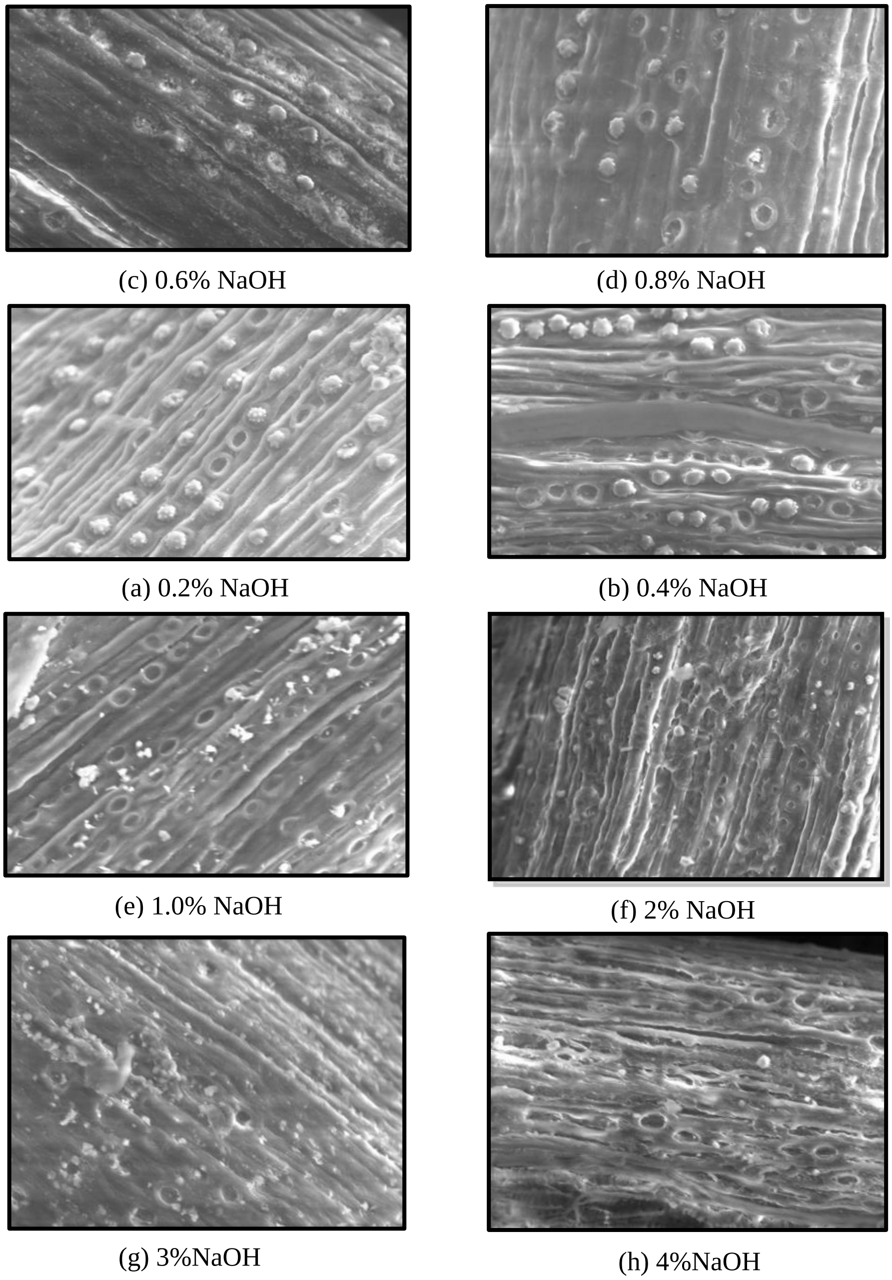


Figure 4. Scanning Electron Microscope (SEM) image at 250 magnification of Oil Palm Empty Fruit Bunch (OPEFB) treated with Sodium Hydroxide (NaOH) range 0.2% to 4% concentration

EFB Fibre – Cement Hydration Rate

The cement compatibility with natural fibre is still a major problem in cement bonded fibreboards production. While adding the certain amount of natural fibre into the cement composite could significantly reduce the hydration temperature of mixture. Hydration rate of neat cement and cement-EFB fibre were recorded for the period of 65 hours to observe the changing of temperature on the mixed material. Based on the Figure 6, the maximum hydration temperature of neat cement obtained was 52°C at 10 hours. The hydration temperatures of neat cement are depend on the type of cement used, amount of cement and water used and sample size (Schackow et al., 2016). On the other part, the temperature of untreated fibre mixed with cement shows an increasing of number at 30.5°C for first 2 hours and then gradually drop for remaining duration until reach the lowest temperature at 28°C. The same results were recorded from previous studies that clearly shows that untreated wood fibre that contain impurities, wax, fatty substances clearly reduced the maximum hydration temperature attained (Ashori et al., 2011; Hermawan et al., 2001). Obviously, EFB fibre could not be used solely as a main material for the cement bonded fibreboard due to contain inherent extractive presence in the fibre.

Conversely, the increasing of hydration temperature was obtained for the cement-EFB fibre mixed by treated EFB fibre using different concentration of Sodium Hydroxide (NaOH). Rising of temperature is depending on the amount of NaOH as interpret in the graph below. Through the experimental, the hydration temperature was increase with increment of NaOH concentration to the fibre treatment. The results can be categorized into three groups according to their graph variation that shows a minimum differentiation of temperature between each other. The first group is EFB fibre treated with NaOH at 0.2% and 0.4% that reached maximum temperature at 33°C and 34°C respectively at time taken 8 hours. The second group is the fibre treated with 0.6%, 0.8%, 1% and 2% with the range temperature from 35°C to 37°C at the time to hit the maximum temperature from 29 to 35 hours. The last group for the EFB fibre treated with 3% and 4% of NaOH. The maximum temperature obtained from this group were 42°C to 44°C at 11 to 15 hours.

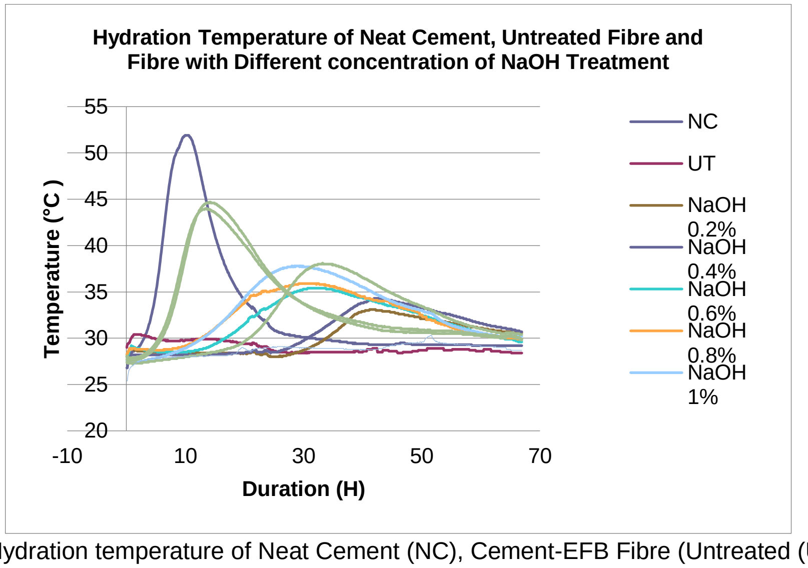


Figure 5. Hydration temperature of Neat Cement (NC), Cement-EFB Fibre (Untreated (UT), 0.2%, 0.4%, 0.6%, 0.8%, 1%, 2%, 3% and 4% of NaOH Treatment)

The EFB fibre treated with 0.4%, 1% and 4% shows a significant different on hydration temperature of cement-EFB fibre mixed as well as the time taken to reach the peak temperature. In addition, the observation that have been done earlier using SEM shows the significant changes on surface morphology of EFB fibre can be classified into four (4) groups; Untreated (UT), 0.4% NaOH, 1.0% NaOH and 4% NaOH. Therefore, further investigation for EFB fibre on tensile strength prepared based on UT (control), 0.4% NaOH (low concentration), 1.0% (medium concentration) and 4.0% (high concentration).

The Change of EFB Diameter

The physical observation on the changes of EFB fibre diameter have been done for untreated fibre (control) and fibre treated with 0.4%, 1% and 4% concentration of NaOH. The average diameter of EFB fibres was measured by using optical observations (Leica optical microscope) at three different random locations along with the single fibre as recommended by Fiore et al. (2016) and Norul Izani et al. (2013) since the natural fibre cross section has an irregular shape. With reference to the Figure 5, mean value of fibre diameters shows a reduction with increment of NaOH concentration. The average diameter for untreated fibre was found to be 0.479 mm while for treated fibre it was found to be 0.263 mm (0.4% NaOH), 0.251 mm (1% NaOH) and 0.240 mm (4% NaOH). The results may be due to the alkali treatment that removed certain amount of hemicellulose and lignin on the fibre surface as mentioned by Mohanty et al. (2001) thus reduced the diameter of fibre. The reduction of fibre diameter also discussed by Fiore et al. (2016) after several tested on alkali treatment of fibre.

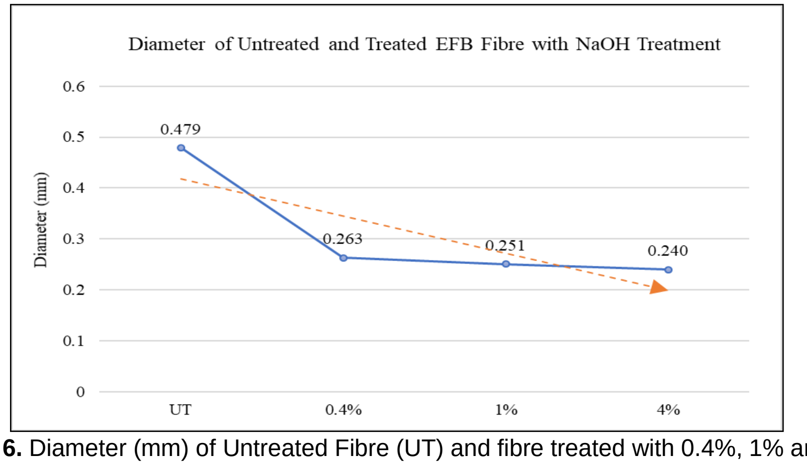


Figure 6. Diameter (mm) of Untreated Fibre (UT) and fibre treated with 0.4%, 1% and 4% concentration of NaOH

Tensile Strength of Single EFB Fibre

The mean value of tensile strength of the untreated and treated EFB fibres are presented in the Figure 7. It was observed that the fibre treatment appeared to significantly increase the tensile strength of the fibres in relation to the untreated fibres. The results also shows that the tensile strength of EFB fibers was increased markedly by increment of NaOH concentration. However, this results was contradict with findings by Nishiyama and Okano (1998) for the ramie fibre where the tensile strength decreased in alkali treated due to the damage caused by a chemical structure change such that cellulose in the fibre partially changes from crystalline cellulose into amorphous cellulose.

The results showed that the tensile strength of EFB fibres was increased markedly by alkali treatment. This may be due to the increased in crystallinity of fibres by alkali treatment. The results are in-line with findings by Norul Izani et al. (2013) that evidently stated that the tensile strength, Young’s modulus, and percent elongation at break of the NaOH soaking treated fibres are higher than the untreated fibre. As mentioned by Mohanty et al. (2001), alkali treatment may depolymerize the native cellulose and delignify the fibre excessively, which can adversely affect the strength of the fibre. It was also mentioned by Cai et al. (2015) that the NaOH treatment removed binding material on fibre such as hemicellulose and lignin.

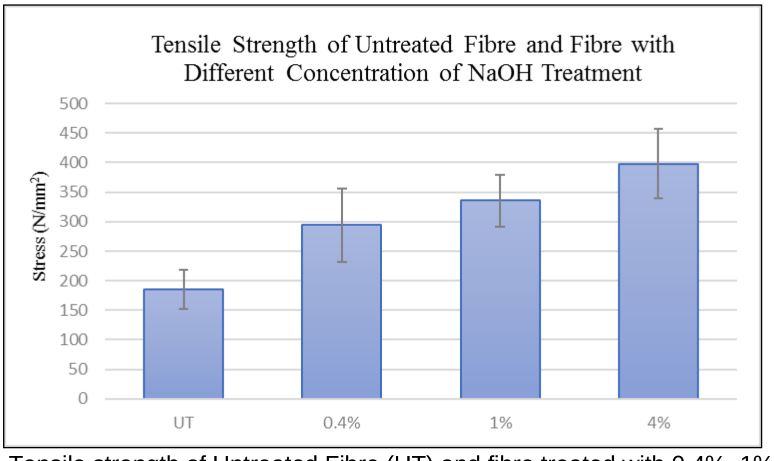


Figure 7. Tensile strength of Untreated Fibre (UT) and fibre treated with 0.4%, 1% and 4% concentration of NaOH

As refer to the Figure 8, the graph pattern shows the increasing of tensile strength when the diameter of fibre was decrease. The significant changes of EFB fibre diameter were due to depolymerize of the native cellulose. The results are in-line with the research by Kabir et al. (2013) which explained the graph pattern that trend downward was observed with the smaller of fibre diameter shows the increment of tensile strength. The increment amount of sodium hydroxide (NaOH) concentration affect the outer surface of EFB fibre thus caused the fibre to shrinkage. The changes of fibre surface due to the removal of silica body as well as reduced the residual oil in the EFB fibre (Zawawi et al. 2015) could increase the performance of fibre. However, Cai et al. (2015) and Kabir et al. (2013) was found that, at 10% concentration of NaOH the fibre became more brittle hence decrease the tensile strength of fibre.

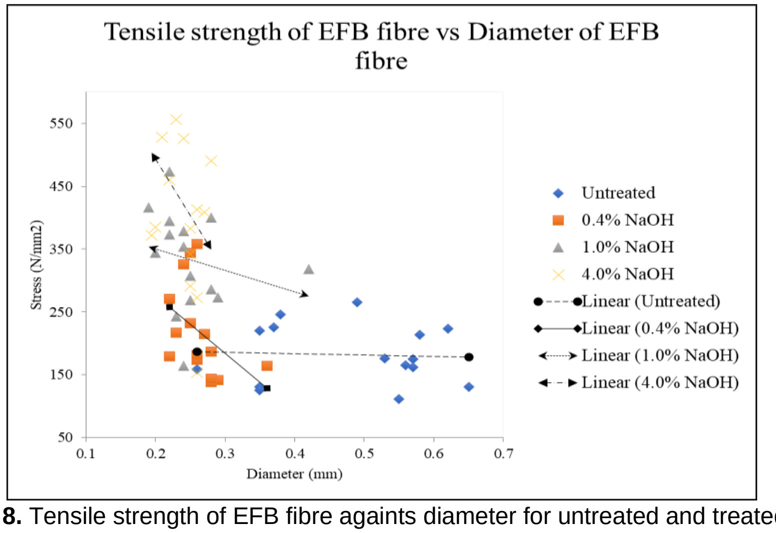


Figure 8. Tensile strength of EFB fibre againts diameter for untreated and treated fibre

CONCLUSIONS

Based on the result of this research the following conclusion can be drawn:

i. The hydration rate of EFB fibre-cement mix, is significantly decrease for the mixture containing untreated fibre, while increase with the increasing of NaOH concentration applied for fibre pre-treatment.
ii. The significant difference from hydration rate of EFB fibre-cement mixed on level of NaOH concentration which is 0.4%, 1% and 4% of alkali treatment.
iii. Alkaline treatment could depolymerize the native cellulose thus increase the tensile strength of single EFB fibre up to 4% of NaOH concentration, apart from that the EFB fibre diameters were decreased with increasing of NaOH concentration.
iv. The utilization of EFB fibre for cement based product could be accepted with modification surface of EFB fibre by alkali treatment using sodium hydroxide (NaOH).

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BIO-CONCRETE ON CHLORIDE CONDITION: EFFECT ON COMPRESSIVE STRENGTH AND WATER PENETRATION

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Abstract
In recent years, the beneficial effect of sulphate reduction bacteria (SRB) to induce calcium carbonate precipitation on the concrete structure has gradually increase great attention in the industry. In this study, different SRB concentration (0%, 3%, 5% and 7%) and water cement ratio (0.4w/c, 0.5w/c and 0.6w/c) was investigated in term of compressive strength and water penetration. Since the sample curing in saline water, thus, the sample on chloride condition also investigated accordingly. Compressive strength and water penetration test were performed at the stage of 28th, 56th, 90th, 180th and 360th day of curing period. Test results indicated that the best SRB concentration to be mixed in bio-concrete was 5% while 0.5w/c for water cement ratio. Both values enhanced significantly the compressive strength and reduced porosity and water penetration of bio-concrete. The maximum increased of compressive strength in 0.5w/c is 58.6MPa was observed with 5% of SRB on the day 180 of curing time. Meanwhile, the lowest water penetration was recorded on the last day (day 360) of curing with 2.93cm at the 5% and 0.6w/c of SRB concentration and water cement ratio, respectively. These improvements were due to deposition on the SRB cells within the pore of the concrete cube as cured in chloride water. Results of this study demonstrated the role of SRB induced calcium carbonate precipitation in improving the concrete structure cured in extreme condition in term of compressive strength and water penetration.

Keywords: *Bio-concrete, Calcium carbonate, Chloride attack, Compressive strength, Sulphate reduction bacteria, Water penetration*

INTRODUCTION

In the surrounding area, such as in the archipelago or a long coastal area that surrounded by sea water concrete structures used to be exposed to extreme conditions such as saline water. This condition leads to the reduction in durability of the concrete structure due to external factors or environment-related factors such chemicals, sulfates, chlorides and salts in the sea area. The deposition of inorganic matter around the concrete structure can threaten the life span of the concrete. Among the threatening factors like freezing and thawing, abrasion, corrosion of steel, the chemical attack may deteriorate the concrete significantly (Zivica and Bajza, 2001; Azmi et al., 2017). Chemical attack can be classified as an acidic attack, alkali attack, carbonation, chloride attack, leaching and sulphate attack. The threat to be discussed and investigated in this study is chloride attack.

Chloride is regarded as extremely dangerous in causing the corrosion of prestressing steel in concrete. Chloride ingress on concrete is one of the most important aspects for consideration especially in dealing with the durability of concrete. Chloride attack is particularly important because it primarily causes corrosion of reinforcement. Statistics have indicated that over 40 per cent of failure of structures is due to corrosion of reinforcement (Verma et al., 2013). Due to high alkalinity of concrete, a protective oxide film is present on the surface of steel reinforcement. The protective passivity layer can be lost due to carbonation. This protective layer also can be lost due to the presence of chloride in the presence of water and oxygen (Khalid et al., 2018). In reality, the action of chloride in inducing corrosion of reinforcement is more serious than any other reasons (Darmawan, 2010; Neville, 1995).

Furthermore, corrosion of reinforcing steel due to chloride ingress is one of the most common environmental attacks that lead to the deterioration of concrete structures. Corrosion-related damage to bridge deck overlays, parking garages, marine structures, and manufacturing plants information about mechanisms of deterioration most studied by the researcher (Neville et al., 2004). Deterioration of concrete more effect of the environment of the system in conjunction with the structural requirement to describe the cause of such deterioration (Suwito and Xi, 2008). Environment factors consist of seasonal variations freeze thaw cycling, relative humidity, chemicals in the ground, ground water, are the main degradation agents.

A typical durability-related phenomenon in many concrete constructions is crack formation. While larger cracks hamper structural integrity, also smaller sub-millimetre sized cracks may result in durability problems as particularly connected cracks increase matrix permeability (Jonker et al., 2012; Mazenan et al., 2017). Durable concrete with the biological approach is called bio-concrete in which microorganisms one of very important to deterioration in porous materials and sealing concrete cracks (Anneza et al., 2016; Zamer et al., 2017). However, bacteria are unable to tolerate with all environmental conditions because there are some factors restricting the growth of bacteria such as temperature, type of growth medium, extreme pH value and anaerobic conditions and bacteria successfully isolated to survive in the alkaline environment with anaerobic conditions and was called as sulphate reduction bacteria (Alshalif et al., 2016).

Theoretically, sulfate reducing bacteria (SRB) require dissolved oxygen to induce the corrosion cycle and the bacteria are not able to tolerate with extreme pH value. Under the acidic condition, these bacteria cells hydrolyze or enzyme inactivate. If the surrounding environment of the SRB is in alkaline condition, the enzymes work very well. The pH value plays an important role in microbial life that will influence the dissociation and solubility of many molecules that indirectly influence microorganism research methodologies, especially those incorporating the biological effect on concrete. The study of sulphate reduction bacteria isolated from acid mine water the results showed that optimal pH for growth condition is 9.35 (Irwan et al., 2016). The bacteria incorporation in concrete reduces mass variation, volume variation (higher age) and water absorption (Gandhimathi and Suji, 2015).

Therefore, the objective of this study is to investigate the effect of sulphate reduction bacteria (SRB) concentrations as influenced by water cement ratio (W/C) regarding the chloride attack during curing period up to 360 days on the compressive strength profile and water penetration test.

MATERIALS AND METHODS

Isolation of sulphate reduction bacteria

Sulphate reduction bacteria (SRB) used in this study was collected from acid mire water located in Sungai Pelepah Kota Tinggi, Johor, Malaysia. Sample isolation and preparation of the inoculum follows Alshalif et al. (2016). SRB was enriched under special environment to tolerate the high alkalinity and anaerobic conditions for ensuring self-sustain in the concrete. In this study, sodium chloride (NaCl) was added to simulate chloride environment during re-culturing SRB that previously isolated. Therefore as requirements BS EN 13396 :2014, it increases the durability of concrete as it would prevent water seeping as a culprit in causing carbonation of steel and end up to threaten the whole building to be less durable. In this study, reduction of water seeping also minimizing penetration of 3% sodium chloride that already dissolve in curing water. Chloride is a vital cause in creating corrosion of any steel material in concrete structure, thus reducing concrete durability (Nosouhian et al. 2015). The compositions of media in re-culturing SRB process were prepared as follow:

a) Control = 25ml(nutrient broth) + 10 ml (3% NaCl) = 35ml
b) SRB sample = 25ml(nutrient broth) + 10ml (3% NaCl) + 1 bead of SRB =35ml

Bio-concrete preparation

The bacteria growth curve was plotted based on daily absorbance of the bacteria until the bacteria reach optimum growth condition. Before fabricating concrete, the bacteria are re-grown according to the optimum growth day following bacteria growth curve. Bio-concrete mixture was designed according to DOE. The fabrication of concrete was done according to BS 1881-125:2013 (British Standard, 2013). Concrete mix proportion was using a different percentage of SRB concentration as influenced by 0.4 w/c, 0.5 w/c and 0.6 w/c to achieve the target strength as tabulated in Table 1. Ordinary Portland cement was obtained from local manufacturer which consist the composition and specification complying with all requirements defined by BS EN 197-1:2000. The size of fine aggregate was below 5mm while coarse aggregate was in the range (12-20)mm. Whereas, the required size of aggregate in concrete used sieve analysis test which conducted according to BS 882:1992. All procedure in preparation of bio-concrete based according to BS 1881-125:2013 (British Standard, 2013).

Table 1. Mix proportion of bio-concrete with different percentages of SRB prepared for fabrication

Percentage (%) of bacteria	Cement (kg/m^3)	Water (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)
0%	420	210	685	1115
3%	420	207.9	685	1115
5%	420	203.7	685	1115
7%	420	199.5	685	1115

Test procedure

Mechanical properties tests that were performed is compressive strength and water penetration test. All tests for both compressive strength and water penetration were conducted in triplicate samples. The cubes molds of 150mm x 150mm x150mm were prepared both with and without (SRB). In order to simulate chloride condition, concrete specimens were cured in 3% NaCl for 28^{th} , 56^{th}, 90^{th}, 180^{th} and 360^{th} days. Compressive strength test was performed according to BS EN 12390-3:2009 (British Standard, 2009) and BS EN 12390-8:2009 (British Standard, 2009) using universal testing machine (UTM). Water penetration test was conducted according to BS EN 12390-8 in order to determine the increase in resistance towards water penetration in concrete specimens (British Standard, 2009).

RESULTS AND DISCUSSION

Compressive strength test

Results on the compressive strength of bio-concrete specimens prepared with different SRB percentage (3%, 5% and 7%) as influenced by W/C ratio (0.4, 0.5 and 0.6) are presented in Figure 1 and Table 2. All specimens had been tested after achieving maturity of concrete on the day 28, 56, 90, 180 and 360.

As shown in Figure 1A, specimens containing 0.4 W/C ratio with bacterial (SRB) showed a considerable increase in the compressive strength (MPa) compared to control specimen. On the day 90, the highest compressive strength occurred at 3% of SRB (64.5 MPa) followed by 5% of SRB (46.97 MPa) and 7% of SRB (34.23 MPa). The drastic reduction had been observed at 3% of SRB on the day 180 of curing time where the compressive strength reduced from 64.5 MPa on the day 90 to 46 MPa on the day 180. However, this value still higher than the minimum requirement of compressive strength of concrete grade 30 (30 MPa) in which had been simulated in this study. Meanwhile, 5% and 7% of SRB were increasing gradually in compressive strength on the day 180 of curing time up to 65.57 MPa and 55.5 MPa, respectively. If compared to control specimen, the compressive strength of bio-concrete in 0.4 W/C ratio on the day 180 of curing time increased up to 12.4%, 38.4% and 27.26% for 3%, 5% and 7% of SRB, respectively. It indicated that 0.4 W/C ratio in concrete mixture highly influence the strength after 180 days of curing in 5% and 7% of SRB (Figure 1A).

Since the specimens of bio-concrete were tested after reaching the post maturity period, there was no significant increment of compressive strength for 0.5 W/C ratios (Figure 1B) on the day 28, 56, 90 and 180 of curing time. Most of the specimens with different SRB percentage including control specimen showed no major changes in the compressive strength. Yet in Figure 1B, on the day 180 of curing time, there was a clear trend of increase in compressive strength for all specimens (3%, 5% and 7% of SRB) including control specimen. By applying 0.5 W/C ratio, the compressive strength of specimens at 3% of SRB had appeared as the highest up to 63.9 MPa while for 5% and 7% of SRB was 58.57 MPa and 58.7 MPa, respectively. This increment was equivalent to 13.1%, 5.2% and 5.4% for 3%, 5% and 7% of SRB, respectively compared to control specimen (0% of SRB) on the last day of curing time (180 days).

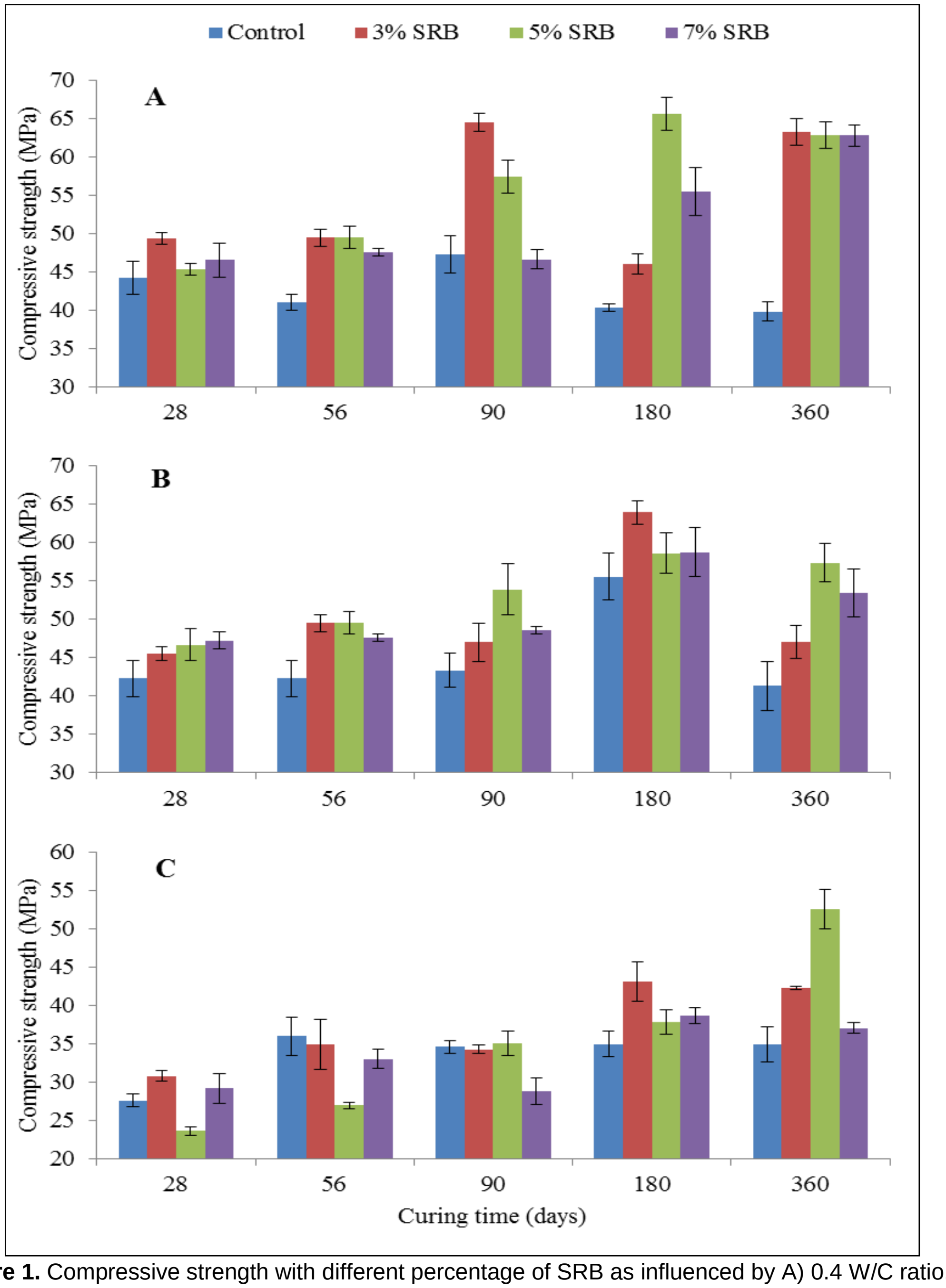

Figure 1. Compressive strength with different percentage of SRB as influenced by A) 0.4 W/C ratio, B) 0.5 W/C ratio and C) 0.6 W/C ratio

Table 2. Compression test of concrete with different SRB percentage

Curing time (days)	W/C ratio	Compression strength, MPa			
		Control	3% SRB	5% SRB	7% SRB
28	0.4	44.27±1.23	49.33±0.43	45.33±0.47	46.53±1.27
	0.5	42.23±1.36	45.50±0.53	46.60±1.21	47.20±0.67
	0.6	27.60±0.49	30.77±0.41	23.63±0.32	29.17±1.14
56	0.4	41.03±0.62	49.47±0.65	49.53±0.85	47.57±0.30
	0.5	42.23±1.36	49.47±0.65	49.53±0.85	47.57±0.30
	0.6	35.97±1.44	34.90±1.87	26.93±0.22	33.00±0.72
90	0.4	47.23±1.41	64.50±0.68	57.43±1.23	46.60±0.72
	0.5	43.30±1.27	46.97±1.44	53.87±1.92	48.50±0.29
	0.6	34.60±0.47	34.23±0.33	35.03±0.90	28.83±1.01
180	0.4	40.37±0.29	46.00±5.44	65.57±1.23	55.50±1.80
	0.5	55.53±1.75	63.90±0.86	58.57±1.53	58.70±1.84
	0.6	34.97±0.97	43.10±1.49	37.83±0.90	38.67±0.61
360	0.4	39.83±1.27	63.23±1.72	62.80±1.73	62.80±1.41
	0.5	41.23±3.14	46.97±2.15	57.33±2.47	53.37±3.13
	0.6	34.87±2.29	42.27±0.23	52.53±2.58	37.00±0.70

Similarly, specimens in 0.6 W/C ratio with different SRB percentage also depicted the same trends in term of compressive strength. After 56 days of curing, 0.6 W/C ratio specimens with different SRB percentage showed considerably increase the compressive strength compared to 28 days of curing (Figure 4.1C). The compressive strength almost constant until 90 day of curing time for all sample, except 5% of SRB. A quick increment in compressive strength at 5% of SRB specimen was recorded about 35.03 MPa on the day 90. Besides, fluctuate reading had been observed for 7% of SRB specimen during the curing time. However, on the day 180, 7% of SRB in 0.6 W/C ratio specimens had increased up to 38.67 MPa while for 3% of SRB was 43.1 MPa and for 5% of SRB was 37.83 MPa. This increment represents about 18.9%, 7.6%, 9.6% for 3%, 5% and 7% of SRB, respectively compared to control specimen (0% of SRB).

The current findings enhance the understanding of varies in W/C ratio produce different compressive strength of bio-concrete as influenced by different SRB percentage (3%, 5% and 7%). Thus, it was found that the higher W/C ratio causes a decrease in the compressive strength value as on the last day of curing time (Table 4.1). This was probably due to the excessive water content in the bio-concrete mixture leads to weak in compression strength. However, individual comparison (0.4, 0.5 and 0.6 W/C ratio) revealed that each W/C ratio was increased in term of compressive strength during the curing time. In fact, all different SRB percentage specimens indicated a good compression strength compared to control specimen. Similar findings had also been reported by another researcher (Babu and Siddiraju, 2016; Balam et al. 2017; Kalhori and Bagherpour, 2017). Babu and Siddiraju (2016) reported that 5% of bacteria (species not specify) slightly increase the compressive strength (49.5 MPa) of bio-concrete compared to concrete without bacteria at 28 days of curing time. Thus, this research contributed to a new finding of simulaed bio-concrete in 3% NaCl that was slightly different from previous studies.

Theoritically, pores in bio-concrete are partially filled up by material growth with the addition of bacteria in which leads to increased density of the concrete strength (Babu and Siddiraju, 2016). Meanwhile, (Balam et al., 2017) used bacteria (*Sporosareina pasrearii*) for remediation of lightweight aggregate concrete on compressive strength found that 26% remarkable increment of specimen tested after 150 days of curing time. Kalhori and Bagherpour (2017) applied *Bacillus subtilis* for repairing crack of concrete indicate 30% increase in the compressive strength of bacteria concrete specimens compared to control specimen. This situation occurred because of the bacteria precipitation of calcium carbonate on the surface and in the pores of the specimens (Kalhori and Bagherpour, 2017).

As in this study, 3%, 5% and 7% of SRB significantly influenced the compressive strength of bio-concrete. However, the best bacteria (SRB) percentage to be used was at 5% for 0.4 W/C ratio and 3% for both 0.5 and 0.6 W/C ratio. In theory, the inclusion of microorganisms especially bacteria has increased the compressive strength of concrete. For instance, insignificant improvement in the compressive strength of cube specimens without bacteria compared to bio-concrete. The strength improvement in bio-concrete is due to calcite precipitation by bacteria reaction in the specimens itself (Krishnapriya et al., 2015). Furthermore, this positive impact of the presence of bacteria on compressive strength of bio-concrete could be because of the internal structure of specimens. Basically, bio-concrete is still porous during initial curing time, permitted to the influx of nutrients, water, oxygen and thereby facilitated the growth of bacteria cells, resulting in the precipitation of calcium carbonate (Alonso et al., 2017). The same mechanism happened in the present study where the addition of SRB in conventional concrete improved the strength and durability of the bio-concrete. Thus, the behaviour of increased compressive strength with bacteria cell can be explained (Vijay et al., 2017). By introducing the SRB in concrete, precipitation of calcite was higher in bio-concrete compared to control specimen and simultaneously enhance the compressive strength properties of the bio-concrete.

Water penetration test

The trends for influence water cement ratio (0.4, 0.5 and 0.6 W/C) on water penetration on bio-concrete when the factors were varied over the different percentage of bacteria (3%. 5% and 7% of SRB) are shown in Figure 2 and Table 3.

From Figure 2A, it can be seen that all specimens on 0.4 W/C show no high different in water penetration test for day 28 until day 360. Bio-concrete sample indicated low in water penetration compared to control sample. For example, control sample on the day 56 to day 360 obviously higher up to 14.53cm. It proved that bio-concrete specimens do not give major effect on the water penetration after reaching the maturity level of bio-concrete which is 28 days onwards. Different for 0.5 W/C (Figure 2B) and 0.6 W/C (Figure 2C) where high water penetration test was observed on the day 28 of curing time. However, after 56 days to 360 days, there were no significant differences exist on water penetration test for bio-concrete.

By comparing the all specimens on the last day of curing time (360 days), the lowest water penetration for 3% of SRB was at 0.6 W/C which is 3.37 cm followed by 0.5 W/C (3.43 cm) and 0.4 W/C (3.63 cm). For 5% of SRB, on the day 360, 0.6 W/C also had been observed to obtaining the lowest water penetration test about 2.93 cm while 0.4 W/C and 0.5 W/C was 3 cm and 4.13 cm, respectively. Furthermore, when specimens mixed with 7% of SRB, the best value of water penetration was at 0.5 W/C with 2.6 cm. Accordingly, the overall result of water penetration test, the most appropriate water cement ratio is 0.5 W/C since this ratio obtained the lowest water penetration compared to 0.4 W/C and 0.6 W/C.

The improvements of water penetration in bio-Concrete as in this study also supported by the previous study (Alshalif et al., 2016; Irwan et al., 2016; Kalhori and Bagherpour, 2017). The mechanism of this activity can be further explained affected by calcium carbonate precipitation in the bio-concrete specimen. Therefore, the addition of bacteria (SRB) into bio-concrete has improved the concrete water permeability by producing calcium carbonate which reduces the pores at a faster rate. Since water penetration is a measurement for durability, concrete structure increase with the reduction of water penetration.

Thus, the potential of steel reinforcement to corrode and deteriorate can be avoided wisely. In this study, the significant decrease water penetration by bio-concrete compared to control specimen is due to the precipitation process which calcium carbonate plugs the pores within the concrete matrix. This plugging of pores decreased the water penetration in the concrete. Other than that, improvement in water penetration also due to the present of calcium source acts as a booster to further deposit more calcium carbonate which source apart from the availability of calcium from cement used. Consequently, once the pores are sealed, reduction in water ingress is observed. This bacterial action deposition can seal the pores, voids and microcracks where other sealants are unable to work through it. Therefore, it increases the durability of concrete as it would prevent water from seeping into concrete to cause carbonation of steel and chlorination which would jeopardize the strength of the structure and cause the whole building to be less durable than intended (Chahal et al., 2012).

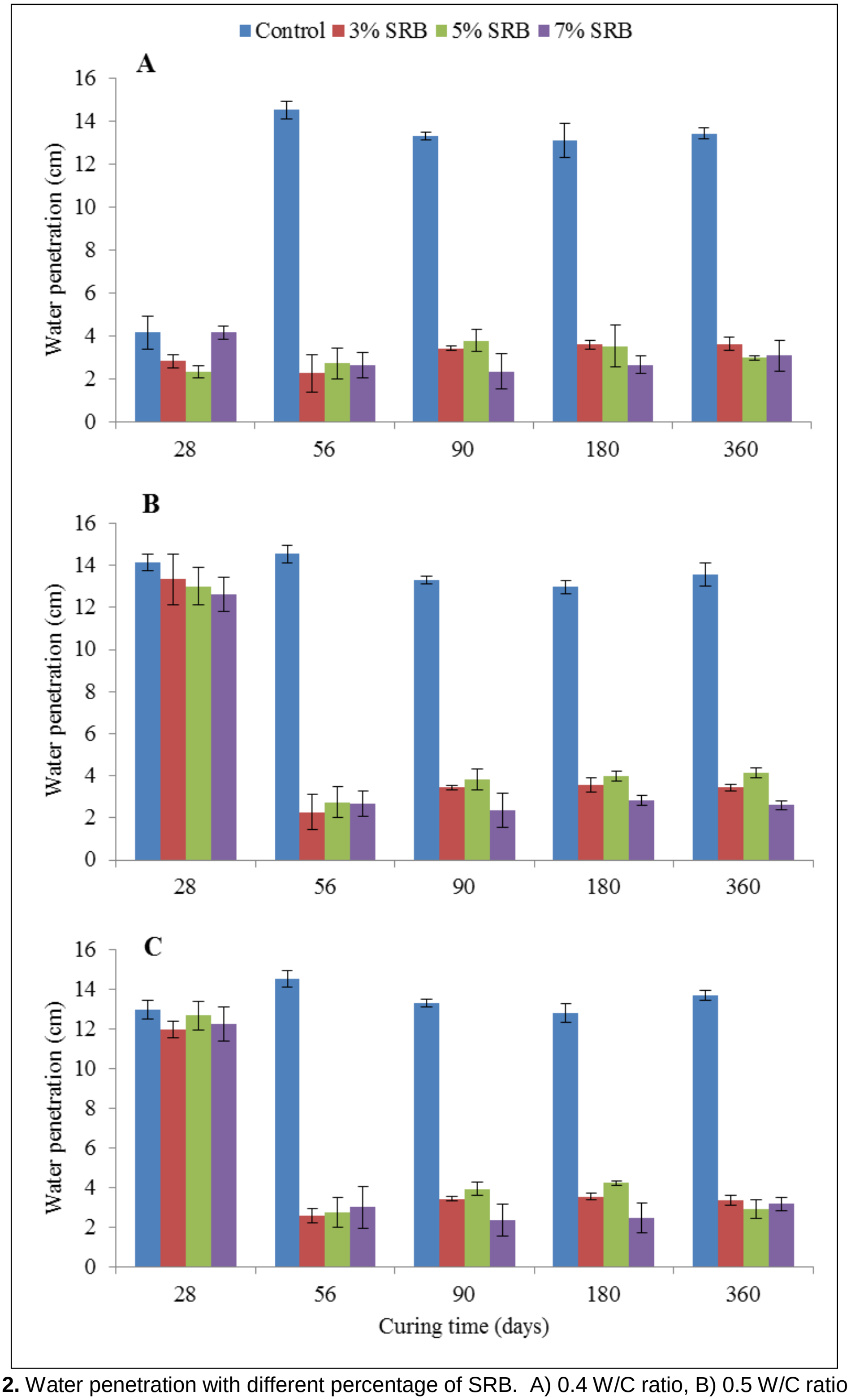

Figure 2. Water penetration with different percentage of SRB. A) 0.4 W/C ratio, B) 0.5 W/C ratio and C) 0.6 W/C ratio

Table 3. Water penetration test of concrete with different SRB percentage as influenced by W/C ratio

Curing time (days)	W/C ratio	Compression strength, MPa			
		Control	**3% SRB**	**5% SRB**	**7% SRB**
28	0.4	4.17±0.44	2.83±0.17	2.33±0.17	4.17±0.17
	0.5	14.13±0.23	13.33±071	13.00±0.51	12.63±0.47
	0.6	12.97±0.28	11.97±0.24	12.67±0.43	12.27±0.50
56	0.4	14.53±0.23	2.27±0.49	2.73±0.43	2.67±0.34
	0.5	14.53±0.23	2.27±0.20	2.73±0.43	2.67±0.34
	0.6	14.53±0.23	2.57±0.07	2.73±0.43	3.00±0.61
90	0.4	13.30±0.10	3.43±0.07	3.80±0.29	2.37±0.47
	0.5	13.30±0.10	3.43±0.07	3.80±0.29	2.37±0.47
	0.6	13.30±0.10	3.43±0.12	3.93±0.19	2.37±0.47
180	0.4	13.10±0.46	3.60±0.19	3.53±0.56	2.67±0.23
	0.5	12.97±0.19	3.57±0.09	3.97±0.13	2.83±0.15
	0.6	12.80±0.26	3.53±0.18	4.23±0.07	2.47±0.44
360	0.4	13.43±0.15	3.63±0.18	3.00±0.06	3.10±0.42
	0.5	13.57±0.32	3.43±0.09	4.13±0.15	2.60±0.12
	0.6	13.67±0.15	3.37±0.15	2.93±0.27	3.17±0.19

CONCLUSION

The present study was designed to investigate the effect of sulphate reduction bacteria (SRB) on compressive strength and water penetration as influenced by water cement ratio (w/c) regarding bio-concrete structure on chloride condition. The results of this investigation showed that the best water cement ratio is 0.5w/c while the most appropriate concentration of SRB is 5%. These findings were obtained according to the highest compressive strength observed during curing period and the lowest water penetration results. More broadly, research is needed to determine the flexural strength test on the bio-concrete combined with further investigation in aggressive condition such as chloride attack. Thus, this innovation can be applied as the newest alternative in construction especially for structures built on coastal areas.

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CHARACTERIZATION OF COAL BOTTOM ASH (CBA) HEAVY METAL ELEMENT AFTER ACID LEACHING TREATMENT

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Abstract
Coal ash is a residue which was produce during the combustion of coal. Coal fly ash (FA), coal bottom ash (CBA) and boiler slag was primarily produced from the combustion process. Unfortunately, the existences of metallic elements in CBA cause the limitation to use CBA in industries. Extensive researches have been carried out to reduce heavy metal element in CBA. From previous studies, the strong acid leaching treatment was carried out to remove metallic impurities in CBA. The usage of strong acid could significantly hazardous to human and environment. In this study, the leaching process was done by replacing the usage of strong acid with citric acid which acted as weaker acid. Experimental result shows that the acid leaching treatment has potential to be used in reducing metallic elements in CBA under optimum solution temperature of 40^{0}C with 4% acid concentration at 60 min reaction period. The characteristic of CBA was also determined by comparing the SEM, particle distribution, specific gravity, water content and setting time of raw CBA with treated CBA.

Keywords: *Coal bottom ash; Heavy metal; Leaching treatment*

INTRODUCTION

Combustion of coal in power plant station produces coal bottom ash CBA consisting of mostly incombustible inorganic and organic matter that is not fully burned. With a growth in coal burning power station, a huge amount of CBA which considered as hazardous waste material were conventionally disposed on site without any commerlization purpose. Instead of dumping, CBA have a huge potential to be used in industries including as fine aggregate replacement (Cheriaf et al., 1999; Senin et al., 2016) and construction material (Aggarwal and Siddique, 2014). Reviewed made by (Kim and Lee, 2015) proved that CBA has been conventionally used as soil replacement, gravel for embankment, structural fill, road construction and various cement product. Besides, a good pozzolanic reaction between CBA, OPC and water makes it possible to be used as cement replacement in concrete mixture (Bajare et al., 2013). The use of industrial by-product as partial replacement material is found to be significant for the development of alternative construction components as substitution to the traditional materials (Pahroraji et al., 2016; Shahidan et al., 2016; Sheikh Khalid et al., 2017; Shahidan et al, 2018).The major problems related to CBA disposal are probably due to the presence of heavy metals in the residue. The existence of heavy metal elements in CBA such as Copper (Cu), Nickel (Ni), Chromium (Cr), Zinc (Zn), Lead (Pb), Manganese (Mn), Ferum (Fe) and Arsenic (As) resulting the classification of CBA in Malaysia under the Schedule Waste (SW 104) Environmental Quality Act. After the coal burnings process, toxic pollutants mainly released into the atmosphere which consequently leach out and contaminate soils, as well as surface water and groundwater (Baba and Kaya, 2004).These elements contained in the CBA have the ability to leach from them and thus enter the environment where they can accumulate in the environment. In addition, the existence of those elements in the ash could pollute the environment and pose a danger to public health and therefore an effective treatment need to be done to overcome this problem. Leaching treatment is a proper route to reduce heavy metal element in CBA. Previous researcher have proved the usage of strong acid such as sodium oxide (NaOH) (Rashidi and Yusup, 2016), Nitric acid (Amfo-Otu et al., 2015) and ammonium oxide (Augustine et al., 2016) were conventionally used in acid treatment to reduce heavy metal in CBA. However, these agents is hazardous that may affect the environment and human health.

MATERIAL: COAL BOTTOM ASH

Tanjung Bin power plant is one of the power plant that using coal to produce electricity. Previous research reported that, Tanjung Bin power plants need approximately 18, 000 ton/day of coal to generate electricity(Muhardi et al., 2010; Shahidan et al., 2011; Ramzi et al., 2016). Due to the large usage of coal, a huge amount of waste such as fly ash, bottom ash and boiler slag will be produced and it is considerable in disposal concern because of the increasing requirement for an ash storage space (Muhardi et al., 2010), hence, this will increase the expanse to manage coal waste in large area that could lead to environmental problem to the future (Faizul et al., 2014).

Table 1. Heavy metal compound in CBA

Compound	Concentration (%)
Pb	0.0016
Cu	0.0055
Ni	0.0071
Zn	0.0092
Cr	0.0118
As	0.0014
Fe	-
Mn	-

METHODOLOGY: ACID LEACHING TREATMENT

Generally, leaching refers to the disposal of a substance from solid via a liquid extraction media. The required component diffuses into the solvent from its natural solid form. In the leaching process, there are three important parameters to be focused which are the temperature, contact time per area and solvent selection. The concentration and temperature and temperature of citric acid solution and stirring time were selected as the operating parameter since the chelate reaction on metal impurities depending on the stated parameters (Faizul et al., 2013). The temperature can be adjusted to optimize the solubility and mass transfer throughout leaching process. There are two categories of leaching which are percolation and dispersed solid. For percolation, the solvent was contacted with solid in a continuous or batch method. Percolation method normally used for huge amount of solid. In dispersed solids, the solid usually crushed into solid small pieces before being contacted with solvent. Previous studies clarified that the heavy metal content in ash can be reduce by extraction and leaching treatment (Sarode et al., 2010; shahidan et al., 2016). Coal bottom ash that was collected from Tanjung Bin was used as raw material which contains heavy metal compound. According to method proposed by several reserchers (Yahya et al., 2017 ; Faizul et al., 2013), 40g of CBA with mean particle size of 45um was weighted and put into 1000 mL citric acid solution in beaker. The concentration and temperature of citric acid solution were selected as operating parameter during leaching process while the stirring period was recorded at 60 min. The acid concentration was gradually controlled from 2% to 5% by changing the mixing ratio between distilled water and citric acid powder. Then, the beaker was placed on the hot plate magnetic stirrer and the solution temperature was changed from 30^0C to 60^0C. After leaching process was done, water rinsing treatment was carried out in the distilled water to removes excessive citric acid content from the ash. The treated CBA was dried in the oven at 60^0C for 60 min. the material then was combusted at 800^0C for 30 min in furnace. The air flow rate in the combustion was set at 2 ml/s until the combustion process end.

TESTING

All testing was done by comparing the result of OPC, Raw CBA and Treated CBA. The characterizations of raw and treated CBA were determined by using Scanning Electron Microscopy (SEM), Specific gravity, and Particle size distribution test. The specific gravity of OPC, raw and treated CBA was determine by following the procedure defined in ASTM D854-98. Particle size distribution was carried out by using particle analyzer (Analizette 22 Micro Tec Fritsch GmbH) on the original software. While the chemical properties of treated CBA were determined by using XRF analysis. 40 pellet sample of treated CBA were prepared and compared to the raw CBA sample. Water consistency test, setting time test was done on cement paste by referring to ASTM C187.

RESULT AND DISCUSSION

Chemical compositions

Effect of Solvent with Acid Concentration

The reaction between solvent and acid concentration is one of the important factor to achieve good result after leaching treatment. Table 2 shows the chemical composition of CBA after acid treatment via citric acid solution with controlled concentration of 2, 3, 4 and 5 %. From the experimental result, the heavy metal concentration of Pb, Cu, Ni and Cr tend to increase compared to control CBA when 2% of acid concentration was used during leaching treatment. Expectedly, the acid concentration increase at 3% and 4%, most of heavy metal concentration decreases including metallic compound of Pb, Cu, Ni and Cr. But the heavy metal concentration of Pb, Cu, Ni and Cr unexpectedly increase when the acid concentration increase to 5%. The heavy metal concentration of Zn and As maintain decrease when the acid concentration increase.

Table 2. Concentration of heavy metal elements in CBA after acid leaching treatment with differences acid concentration

(a) 2% of acid concentration

Compound	**Heavy metal concentration (%)**					
	Control	**2%, 30^0C**	**2%, 40^0C**	**2%, 50^0C**	**2%, 60^0C**	**2%, 70^0C**
Lead (Pb)	0.0016	0.0027	0.0028	0.0026	0.0018	0.0017
Copper (Cu)	0.0055	0.0063	0.0062	0.006	0.0058	0.0055
Nickel (Ni)	0.0071	0.0077	0.0131	0.0126	0.0073	0.0082
Zinc (Zn)	0.0092	0.0055	0.0048	0.0047	0.0068	0.0048
Chromium (Cr)	0.0118	0.0090	0.0514	0.0338	0.0450	0.0370
Arsenic (As)	0.0014	0.0009	0.0011	0.0008	0.0011	0.0009
Mangenese (Mn)	-	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-	-

(b) 3% of acid concentration

Compound	Heavy metal concentration (%)					
	Control	**3%, 30^{0}C**	**3%, 40^{0}C**	**3%, 50^{0}C**	**3%, 60^{0}C**	**3%, 70^{0}C**
Lead (Pb)	0.0016	0.0015	0.0013	0.0025	0.0017	0.0018
Copper (Cu)	0.0055	0.0056	0.0054	0.0059	0.0055	0.0052
Nickel (Ni)	0.0071	0.0068	0.0069	0.0118	0.0072	0.0087
Zinc (Zn)	0.0092	0.0062	0.0059	0.0046	0.0066	0.0049
Chromium (Cr)	0.0118	0.0096	0.0171	0.0357	0.0133	0.0099
Arsenic (As)	0.0014	0.0011	0.0012	0.0009	0.0008	0.0011
Mangenese (Mn)	-	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-	-

(c) 4% of acid concentration

Compound	Heavy metal concentration (%)					
	Control	**4%, 30ºC**	**4%, 40ºC**	**4%, 50ºC**	**4%, 60ºC**	**4%, 70ºC**
Lead (Pb)	0.0016	0.0015	0.0014	0.0016	0.0019	0.0018
Copper (Cu)	0.0055	0.0056	0.0055	0.0057	0.0053	0.0051
Nickel (Ni)	0.0071	0.007	0.0071	0.0076	0.0078	0.0085
Zinc (Zn)	0.0092	0.0063	0.0057	0.0049	0.0050	0.0048
Chromium (Cr)	0.0118	0.0092	0.0087	0.0094	0.0113	0.0100
Arsenic (As)	0.0014	0.0008	0.0010	0.0010	0.0008	0.0010
Mangenese (Mn)	-	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-	-

(d) 5% of acid concentration

Compound	Heavy metal concentration (%)					
	Control	**5%, 30^{0}C**	**5%, 40^{0}C**	**5%, 50^{0}C**	**5%, 60^{0}C**	**5%, 70^{0}C**
Lead (Pb)	0.0016	0.0016	0.0018	0.0017	0.0018	0.0017
Copper (Cu)	0.0055	0.0055	0.0054	0.0057	0.0054	0.0052
Nickel (Ni)	0.0071	0.0074	0.0078	0.0073	0.0085	0.0083
Zinc (Zn)	0.0092	0.0053	0.0051	0.0051	0.0050	0.0052
Chromium (Cr)	0.0118	0.0097	0.0109	0.0106	0.0100	0.0111
Arsenic (As)	0.0014	0.0008	0.0010	0.0010	0.0007	0.0008
Mangenese (Mn)	-	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-	-

Effect of Solution Temperature

Table 3 shows the chemical composition of CBA after leaching treatment was done under different solution temperature. Some researchers had done leaching treatment under ambient temperature. The solution temperature highly effects the heavy metal concentration after leaching treatment especially for heavy metal Pb, Cu and Ni. At 30^{0}C, concentration for Cu shows some increasement but the heavy metal of Pb and Ni decrease at certain acid concentration. But, the concentration of heavy metal Pb, Cu and Ni continuously show a higher value compared to control CBA when the solution temperature was changed to 50, 60 and 70^{0}C.

Result shows that the use of high temperature can increase the leaching performance but the use of organic acid with high temperature seems to be limited due to low boiling temperature and the decomposition of citric acid. The temperature tends to vaporize the acid and distilled water (Gharabaghi et al., 2010).

Table 3. Concentration of heavy metal elements in CBA after acid leaching treatment with differences temperature

(a) 30ºC of solution temperature

Compound	Heavy metal concentration (%)				
	Control	**30^{o}C, 2%**	**30^{o}C, 3%**	**30^{o}C, 4%**	**30^{o}C, 5%**
Lead (Pb)	0.0016	0.0027	0.0015	0.0015	0.0016
Copper (Cu)	0.0055	0.0063	0.0056	0.0056	0.0055
Nickel (Ni)	0.0071	0.0077	0.0068	0.0070	0.0074
Zinc (Zn)	0.0092	0.0055	0.0062	0.0063	0.0053
Chromium (Cr)	0.0118	0.0090	0.0096	0.0092	0.0097
Arsenic (As)	0.0014	0.0009	0.0011	0.0008	0.0008
Mangenese (Mn)	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-

(b) 40ºC of solution temperature

Compound	Heavy metal concentration (%)				
	Control	**40^{o}C, 2%**	**40^{o}C, 3%**	**40^{o}C, 4%**	**40^{o}C, 5%**
Lead (Pb)	0.0016	0.0028	0.0013	0.0014	0.0018
Copper (Cu)	0.0055	0.0062	0.0054	0.0055	0.0054
Nickel (Ni)	0.0071	0.0131	0.0069	0.0071	0.0078
Zinc (Zn)	0.0092	0.0048	0.0059	0.0057	0.0051
Chromium (Cr)	0.0118	0.0514	0.0171	0.0087	0.0109
Arsenic (As)	0.0014	0.0011	0.0012	0.0010	0.0010
Mangenese (Mn)	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-

(c) 50ºC of solution temperature

Compound	Heavy metal concentration (%)				
	Control	**50^{o}C, 2%**	**50^{o}C, 3%**	**50^{o}C, 4%**	**50^{o}C, 5%**
Lead (Pb)	0.0016	0.0026	0.0025	0.0016	0.0017
Copper (Cu)	0.0055	0.006	0.0059	0.0057	0.0057
Nickel (Ni)	0.0071	0.0126	0.0118	0.0076	0.0073
Zinc (Zn)	0.0092	0.0047	0.0046	0.0049	0.0051
Chromium (Cr)	0.0118	0.0338	0.0357	0.0094	0.0106
Arsenic (As)	0.0014	0.0008	0.0009	0.0010	0.0010
Mangenese (Mn)	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-

(d) 60ºC of solution temperature

Compound	Heavy metal concentration (%)				
	Control	**60^{o}C, 2%**	**60^{o}C, 3%**	**60^{o}C, 4%**	**60^{o}C, 5%**
Lead (Pb)	0.0016	0.0018	0.0017	0.0019	0.0018
Copper (Cu)	0.0055	0.0058	0.0055	0.0053	0.0054
Nickel (Ni)	0.0071	0.0073	0.0072	0.0078	0.0085
Zinc (Zn)	0.0092	0.0068	0.0066	0.0050	0.0050
Chromium (Cr)	0.0118	0.0450	0.0133	0.0113	0.0100
Arsenic (As)	0.0014	0.0011	0.0008	0.0008	0.0007
Mangenese (Mn)	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-

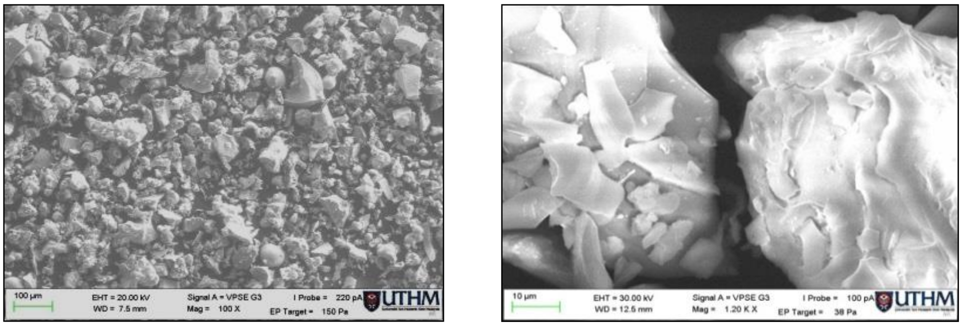
(e) 70ºC of solution temperature

Compound	Heavy metal concentration (%)				
	Control	**70^{o}C, 2%**	**70^{o}C, 3%**	**70^{o}C, 4%**	**70^{o}C, 5%**
Lead (Pb)	0.0016	0.0017	0.0018	0.0018	0.0017
Copper (Cu)	0.0055	0.0055	0.0052	0.0051	0.0052
Nickel (Ni)	0.0071	0.0082	0.0087	0.0085	0.0083
Zinc (Zn)	0.0092	0.0048	0.0049	0.0048	0.0052
Chromium (Cr)	0.0118	0.0370	0.0099	0.0100	0.0111
Arsenic (As)	0.0014	0.0009	0.0011	0.0010	0.0008
Mangenese (Mn)	-	-	-	-	-
Ferrum (Fe)	-	-	-	-	-

Characterization of CBA

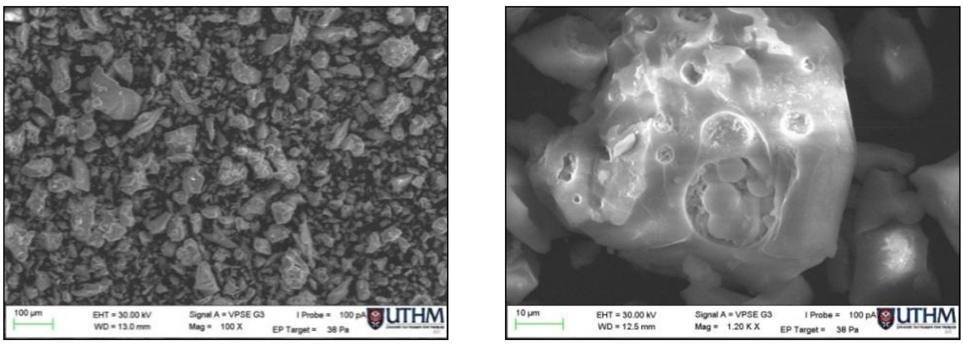
Scanning Electron Microscope (SEM)

Figure 2 and Figure 3 shows a SEM photomicrograph of Tanjung Bin CBA with mean particles of 75 µm which was taken at magnification of 500,300 and 50. Result shown in Figure 1 (a) and Figure 2 (a) shows that, majority of raw CBA particle were irregular, spherical and angular in term of shape. The ash consists of a series of spherical vitreous particles of different size with diameters ranging from 20 µm to 100 µm. Bottom ash particles appeared to be in three type which is fine fraction of shattered bottom ash particle, spherical particles like fly ash and agglomerates of bonded particles (Muhardi et al., 2010). The existence of finer particle shows an appearance of metallic lustre and could be the metals presents in coal bottom ash (Pollock et al.,2000) .Figure 1 (b) shows a lot of finer particle which were observed on the surface of larger particle compared to the treated CBA on Figure 2 (b). The less existence of finer particle on Figure 2 (b) may be due to the successful of acid leaching treatment using citric acid that has been done.



(a) 100 magnification (b) 1200 magnification

Figure 1. SEM image of raw CBA



(a) 100 magnification (b) 1200 magnification

Figure 2. SEM image of treated CBA

Particle Distribution

Figure shows the comparison of particle distribution between raw CBA and treated CBA. From the graph, the maximum coarse particle ranges between 20 to 50 µm does not show significant differences where the weight for raw CBA was 35.06% while treated CBA shows 37.56%. Meanwhile, 37.62% of raw CBA and 27.50% of treated CBA was observed with particle range from 10 to 20 µm. The average value percentage of the particles smaller than 5µm of raw CBA and treated CBA was 17.79% and 16.49 % respectively.

From this test, the results show for raw CBA obtained from Tanjung Bin power plant contained the least fine particles (5% < 2.439 µm). After the CBA was treated, the least fine particle decreases to (5% < 2.226 µm) The mass median diameter (MMD) was determined from the particle diameter at 50% of the cumulative fractions, D50 which is 16.584 µm for raw CBA and 16.793 µm of the particles

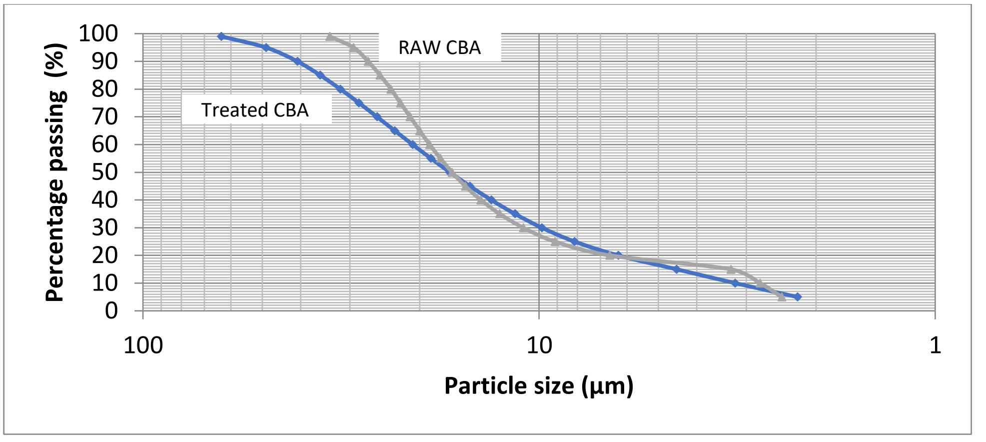


Figure 3. Comparison of particle distribution between raw CBA and treated CBA

Specific gravity

The graphical data in Fig 4 shows that, the specific gravity of treated CBA was the lowest with value 2.28 compared to OPC and raw CBA. While OPC shows the highest specific gravity with value of 2.82. previous research also shows that the specific gravity of CBA was lower than OPC (Abubakar and Baharudin, 2012). The lower specific gravity of CBA may due to the porous structure and pop-corn like particles where it could be easily degraded under loading compaction. The state of the material at the time it was utilize could affect the specific gravity since raw CBA was dried using oven and treated CBA was combusted in furnace. Normally, the specific gravity of coal bottom ash depending on their type, origin, size, handling and processing technique, boiler size and disposal method and storage method (Abubakar and Baharudin, 2012). Previous researcher reported that, the wide range in specific gravity was associated with the chemical composition of CBA. Low iron oxide content resulting in lower specific gravity and vice versa.

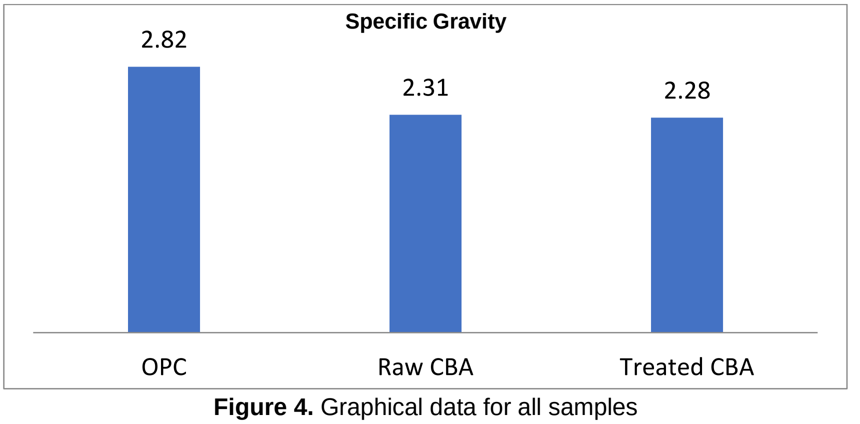


Figure 4. Graphical data for all samples

Water Consistency and setting time

Normal consistency and setting time of OPC, raw CBA and treated CBA are shown in Table 4. The normal consistency of cement paste was 26.8%. it was proved by previous research where they found that the water consistency of normal cement paste usually in range of 26-33% (Abubakar and Baharudin, 2012). Meanwhile, results in Table 4 shows that the water consistency for raw CBA and treated CBA were increase with the increase of CBA replacement. It indicate that the increase of CBA replacement on OPC resulted in higher water requirement (Chai Jaturapitakkul, 2003). Meantime, it was identified that the addition of CBA to cement material increase the initial and final setting time in the relation to the reference mix. It was found that the initial and final setting time for normal cement paste was 80 min and 180 min. While the setting time for CBA was increase with increasing the percentage of replacement. The initial setting time for raw CBA was recorded at 90,120 and 180 min the final setting time was recorded at 235,270 and 300 min for replacement of 10, 20 and 30% respectively. Result for treated CBA shown at the replacement of 10, 20 and 30%, the initial setting time was at 80, 12 and 160 min respectively. Meantime, the final setting time was at 220, 240 and 275 min respectively. Although the high water requirement for treated CBA, but the initial and final setting time much quicker compared to raw CBA. the result obtained by previous researcher proved that the use of CBA to replace OPC is slightly retarded the setting time of cement paste (Chai Jaturapitakkul, 2003).

Table 4. Water Consistency and Setting Time of OPC, Raw CBA and Treated CBA

	Replacement (%)	Water Consistency (%)	Initial Setting Time (min)	Final Setting Time (min)
OPC	-	26.8	80	180
Raw CBA	10	30	90	235
	20	35.8	120	270
	30	41.9	180	300
Treated CBA	10	33	80	220
	20	37.5	120	240
	30	43.5	160	275

CONCLUSION

The experimental study shows that there are possibilities to reduce heavy metal element in CBA through acid leaching treatment especially for heavy metal Zn, Cr and As. The change of acid concentration and solution temperature does not affect the percentage value although there are few state where the percentage value increase compared to control CBA. The heavy metal of Pb, Cu and Ni was a optimum state where the heavy metal concentration were increase when 2% and 5% of acid concentration was used while the heavy metal concentration were decrease when 3% and 4% of acid concentration was applied. When higher temperature was applied, the percentage of concentration of heavy metal Pb, Cu and Ni was increase. The comparison of SEM micrograph between raw CBA and treated CBA proved that the heavy metal content in CBA was reduced. The specific gravity of treated CBA was the recorded as the lowest due to the complicated sample preparation itself.

ACKNOWLEDGEMENT

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IMPROVEMENT OF STRENGTH AND WATER ABSORPTION OF INTERLOCKING COMPRESSED EARTH BRICKS (ICEB) WITH ADDITION OF UREOLYTIC BACTERIA (UB)

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Abstract
Interlocking Compressed Earth Brick (ICEB) are cement stabilized soil bricks that allow for dry stacked construction. This characteristic resulted to faster the process of building walls and requires less skilled labour as the bricks are laid dry and lock into place. However there is plenty room for improving the interlocking bricks by increase its durability. Many studies have been conducted in order to improve the durability of bricks by using environmentally method. One of the methods is by introducing bacteria into bricks. Bacteria in brick induced calcite precipitation (calcite crystals) to cover the voids continuously. Ureolytic Bacteria (UB) was used in this study as a partial replacement of limestone water with percentage of 1%, 3% and 5%. Enrichment process was done in soil condition to ensure the survivability of UB in ICEB environment. This paper evaluates the effect of UB in improving the strength and water absorption properties of ICEB and microstructure analysis. The results show that addition of 5% UB in ICEB indicated positive results in improving the ICEB properties by 15.25% in strength, 14.72% in initial water absorption and 14.68% reduction in water absorption. Precipitation of calcium carbonate ($CaCo_3$) in form of calcite can be distinguish clearly in microstructure analysis.

Keywords: *Compressive strength, Calcite precipitation, ICEB, ureolytic Bacteria, , Water absorption*

INTRODUCTION

Interlocking compressed earth brick (ICEB) masonry has the potential to provide affordable construction around the world. Comprised of basic, inexpensive materials, such as soil, the bricks can provide homes and other facilities at low cost (Haron et al., 2009; Sheikh Khalid et al., 2017). By creating interlocking joints between layers of bricks, ICEBs allow for the bricks to be dry stacked, without the need for mortar. ICEB is a cost effective and sustainable construction material. ICEB construction has the potential to bring durable and affordable homes to developing countries around the world (Laursen et al., 2012). Today, ICEB construction is becoming increasingly popular in developing countries including Malaysia.

Despite the advantages, ICEB also has some deficiencies. According to Irwan et al., (2016) there are other problems associated with ICEB namely low strength, higher water absorption, low fire resistance and high porosity. Water absorption is a function of clay and cement content and usually related with the strength and durability of earth bricks (Riza et al., 2010). The higher rate of water absorption will results in low compressive strength and durability (Khalid et al., 2018).

Production ICEBs by factory were coated with chemical substances which is used to counter erosion due to long periods of high relative humidity. Consequently, the maintenance costs or even early rebuilding costs of deteriorated ICEBs structures are undesirable and unsustainable (Riza et al., 2010*)*. Therefore, to achieve sustainable construction, environmental friendly solution should be implemented to the ICEB.

Previous studies had taken different approach to improve the properties of construction material by introducing the used of bacteria. The use of bacteria is one of new fundamental research in improving construction material in order to pursuit sustainable construction. Researcher such as Muynck et al. (2008) used *Bacillus sphaericus*, Navdeep et al. (2012) used *Bacillus megaterium*, Mukherjee et al. (2013) used *Bacillus megaterium* and Bernardi et al., (2014) used *Sporosarcina paseurii*. All bacteria used by previous studies resulted in increasing on compressive strength and reduction on water absorption by comparing control sample and treated sample with bacteria. Positive results from previous studies indicated that the successfulness of using bacteria as an environmental friendly solution in improving the durability of construction material. According to Siddique et al. (2011), bacteria are able to promote the precipitation of calcium carbonate ($CaCO_3$) in the form of calcite. These calcite acts as bio-sealant by filling the pores which lead to reduction in water absorption, porosity, permeability, enhance the strength and prevent water ingression. Hence it will improve the durability of the material properties.

Even extensive of research incorporated bacteria in construction material, less knowledge were found on application of bacteria in ICEB. Ureolytic bacteria (UB) was selected for this study as an approach toward environmental solution in improving the engineering properties of ICEB. The origin of bacteria used was the same as Irwan et al. (2016b) study which state that UB origin was from fresh urine. Irwan et al. (2016b) study also reported that UB are able to improve the construction material properties by bacterial activities to producing calcium carbonate. Due to availability, locality and positive results from previous research, promote to the selection of UB toward this study. The understanding on fundamental precipitation calcium carbonate ($CaCO_3$) had been applied in this research with the use of ureolyric bactreria (UB) to improve the durability of ICEB.

Therefore, this research hopes that an environmental friendly solution to improve the durability and properties of ICEB would be produced by introducing the used of bacteria in ICEB.

EXPERIMENTAL

Optimum growth condition was a first attempt before using the bacteria in ICEB. Optimal growth need to be measured for ensuring high survival of the bacteria after addition in the brick. Optimum growth condition was measured throughout enrichment process. The enrichment process is important in order to simulate the condition for the bacteria to growth in ICEB environment. This was to ensure the bacteria can acclimatize under ICEB environment. Two sets of UB enrichment were prepared namely control and treatment specimen. The composition of the treatment enrichment consists of 300ml nutrient broth added with 120ml of 40% urea, 1mg of soil substances and lastly 1 cyrogenic bead of ureolytic bacteria. The differences between the compositions of control were the enrichment was done without the soil sample. The method of enrichment process follows the previous study (Irwan et al., 2016 and Othman et al., 2017) by adjusting the pH in alkaline and anaerobic condition for ensuring the survival of ureolytic bacteria in ICEB environment.

The compressive strength, initial rate of water absorption and water absorption test was determined by using ICEB (100mm x 125mm x 250mm) specimen and was measured for 7th, 14th and 28th days. The initial rate of water absorption was done to determine the ability of ICEB to allow the water to pass through it in 1 minute whereas the water absorption test conducted to determine the saturation coefficient which is defined as the percentage of pore volume filled in 24-hour of soaking. The test was done in Structure and Material Laboratory, Universiti Tun Hussein Onn Malaysia (UTHM). There are four set batches of ICEB represented by 0% (control), 1% UB, 3% UB and 5% UB which was partially replaced of limestone water during ICEB production. All batches were tested with a total of 10 samples for each test and batches. The average results are presented and discussed. The test was carried out according to the BS EN 771-1:2011+A1:2015, which specify for the testing for masonry units. The microstructure analysis was done in determination of pore size and morphology precipitation of calcium carbonate ($CaCO_3$) in ICEB. Small fragment of ICEB from the compressive strength test were taken and analyses using Scanning Electron Microscopy (SEM) to determine the position, size, shape and characteristics of the UB in depositing calcite or calcium carbonate in ICEB. The SEM analysis was done for each 0% (control), 1% UB, 3% UB and 5% UB respectively.

RESULT AND DISCUSSION

Compressive strength

Figure 1 shows the results of compressive strength for 7th, 14th and 28th days of testing. The addition of UB as a liquid culture of 1%, 3% and 5% in ICEB increased the compressive strength within time compared to control specimen. The patterns of compressive strength increment are the same for all bacterial ICEB specimen. The highest compressive strength recorded was 6.35 N/mm^2 at the 28th days of testing for 5% addition of UB.

The increment of compressive strength with addition of bacteria agreed with Navdeep et al. (2013) and Ramachandran et al. (2001) studies which states that compressive strength can be significantly increased by application of bacteria calcite. Studies from Siddique and Chalal (2011) have stated in their studies that application of bacterial in material construction would give higher compressive strength compared to control samples due to bacterial activities in depositing calcium carbonate. Bacteria in ICEB induced calcite precipitation (calcite crystals) to cover the cracks continuously. Thus improve the strength. Positive results from the addition of UB in ICEB showed the enhancement of compressive strength properties. Therefore the results of compressive strength increment with addition of bacteria are in accordance with previous research findings.

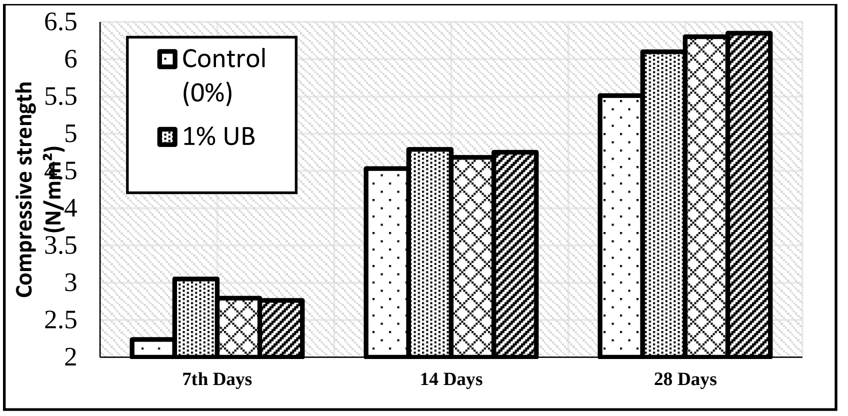


Figure 1. Compressive strength of control and bacterial ICEB

Initial rate of water absorption

Figure 2 shows the results of initial water absorption for 7th, 14th and 28th days of testing. The addition of UB as a liquid culture of 1%, 3% and 5% in ICEB reduced the initial water absorption within time compared to control specimen. The reduction pattern of water absorption rates are the same for all bacterial ICEB specimens. The addition of 5% UB indicates the highest reduction of water absorption rates by 14.72% at 28th days compared to control sample follows with 3% of UB by 11.11% and 1% of UB by 7.5%. The highest reduction of initial water absorption recorded was 3.07kg/m^2.min at the 28th days of testing for 5% addition of UB.

Significant reduction in the extent of water absorption was seen in bacterial cases which clearly indicated that the expected result of bacterial calcification, carbonate crystals were deposited and the pores have reduced within the bricks. However, as a result of the alkalinization of the environment, heterogeneous precipitation can also occur on the surface of the pores. This could result in the plugging of smaller pores (Muynck et al., 2008) which have led to decreased water absorption rates. Muynck et al. (2008) and Achal et al. (2011) studies also observed decreasing in initial water absorption rate upon treatment with bacteria.

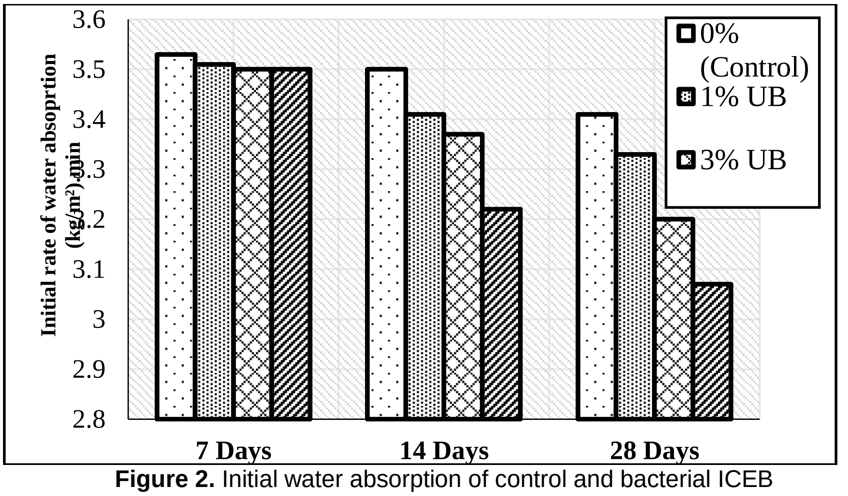


Figure 2. Initial water absorption of control and bacterial ICEB

Water absorption by 24-hour soaking

Figure 3 shows the results of water absorption by 24hour soaking for 7th, 14th and 28th days of testing. The addition of UB as a liquid culture of 1%, 3% and 5% in ICEB reduced the percentage of water absorption within time compared to control specimens. The reduction pattern of water absorption rates are the same for all bacterial ICEB specimen. The addition of 5% UB indicates the highest reduction by 14.68% in water absorption at 28th days compared to control sample follows with 3% of UB with 11.83% and 1% of UB with 10.73%.

According to Mukherjee et al. (2013), the calcite crystal (calcium carbonate) acts as biosealent by filling the pores which leads to reduction in water absorption, porosity, and permeability. The presence of bacteria resulted in a significant decrease in the water absorption compared to control specimens. The deposition of a layer of calcium carbonate on the surface and inside pores of the ICEB specimens resulted in a decrease of water absorption and permeability.

Study was conducted by Nemati and Voordouw, (2003) noticed a decrease in the permeability after injecting calcium carbonate forming reactants. Therefore, it has similarity between present study and the study reported that the presence of a layer of carbonate crystals by bacterial cells has the ability to improve the resistance of penetration of water absorption.

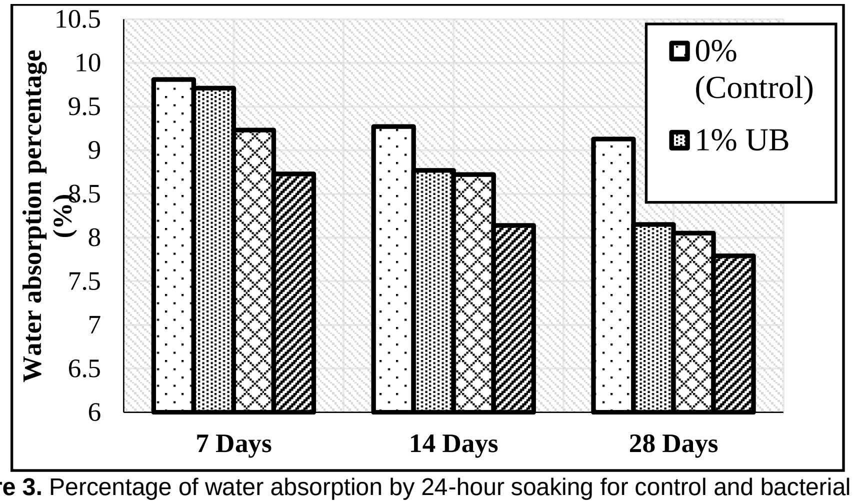
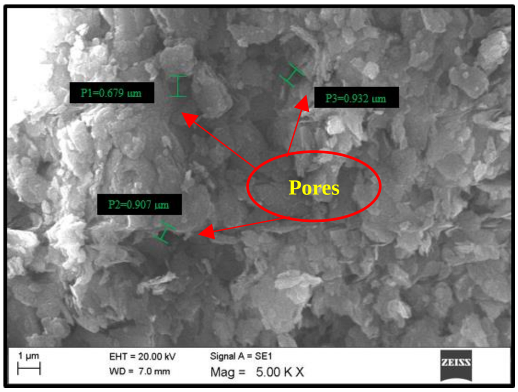


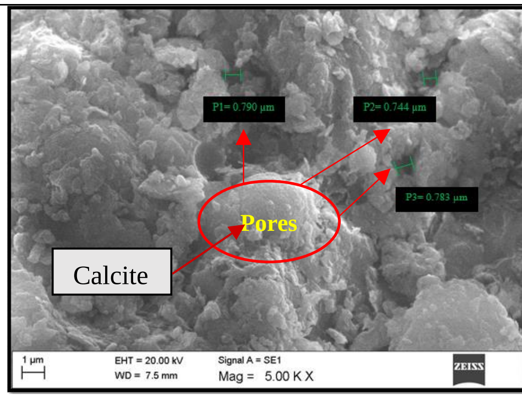
Figure 3. Percentage of water absorption by 24-hour soaking for control and bacterial ICEB

Microstructure analysis

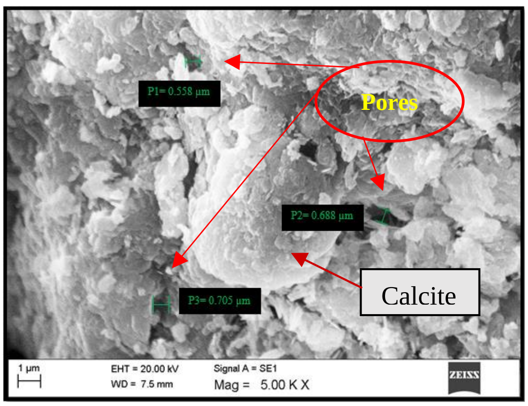
Microstructure analysis shows that UB successful to survive inside the ICEB specimens in anaerobic condition and also capable to tolerate in alkaline environment. Figure 4 shows the average results for pore size analyses on control ICEB and bacterial ICEB. Control image generally showed rough and uneven surface. This porous surface from control samples led to increase of water absorption ability and hence decrease in strength and durability. In bacterial ICEB some part of the porous surface has been filled by calcite precipitation, thus resulted in reducing the pore size. There are also appearances of calcite precipitation which can be seen as smooth surface of round sphere in bacterial ICEB microstructure image in Figure 5. The smooth surface of round sphere indicated the bacterial forms of calcium carbonate. The average size of calcite forms was evaluated and compared between control and bacterial ICEB. Control sample which can be seen without precipitation of bacterial calcite due to 0% percentage of UB addition. Addition of 3% UB indicate highest average size of bacterial calcite form which is 12.469µm follows with 1% of UB with 9.212µm and 5% of UB by 7.815µm. The calcium carbonate precipitation was to prove that addition of UB in ICEB promote to bacterial calcite precipitation.



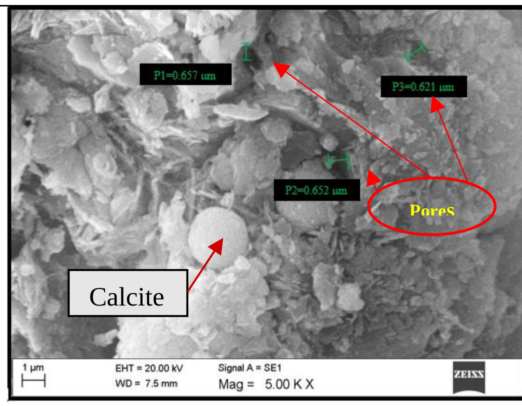
a) Control: (Average pore size = 0.839 µm)



b) 1% UB: (Average pore size = 0.772 µm)



c) 3% UB: (Average pore size = 0.650 µm)

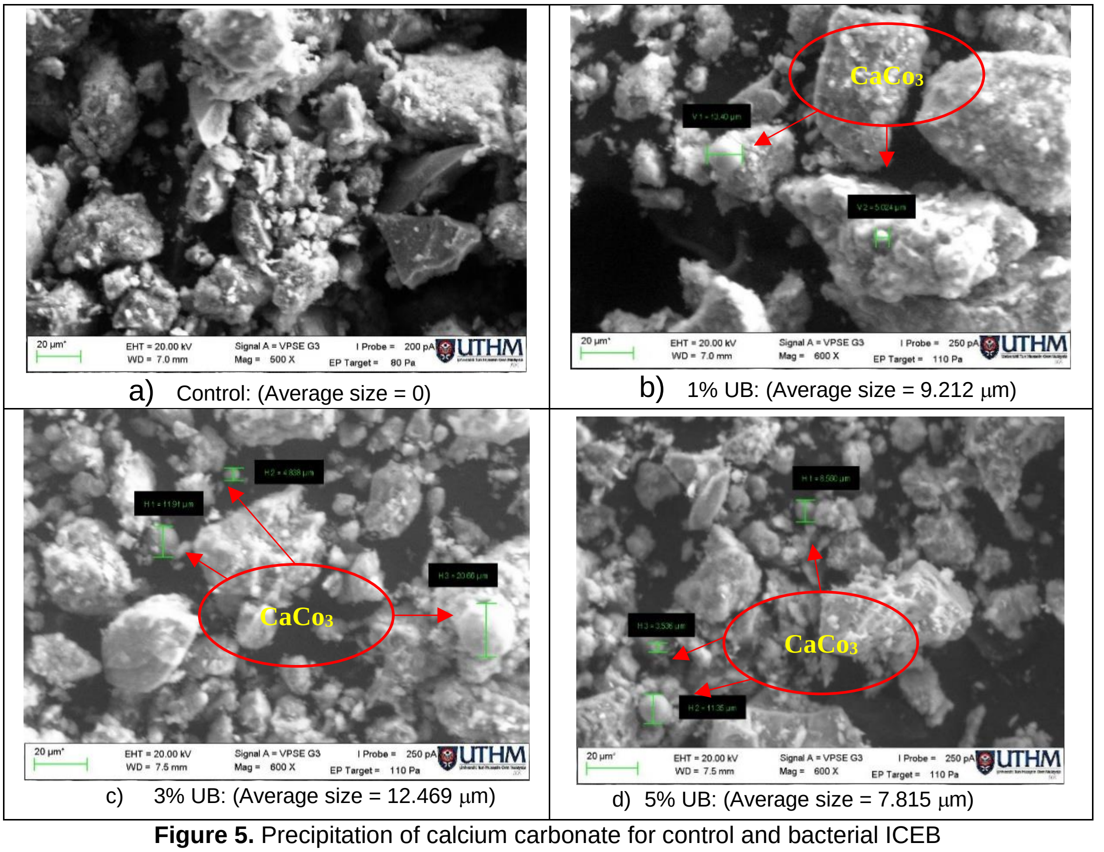


d) 5% UB: (Average pore size = 0.643 µm)

Figure 4. Pore measurement for control and bacterial ICEB

The pores has being filled by bacteria in ICEB and surface for these samples in Figure 4 appear more compact and smooth compared to control which is rough and porous in nature. Decrement in pore size of bacterial ICEB compared to control ICEB was also observed. Addition of 5% UB resulted highest reduction in average pore size with 0.643µm follows with 3% UB by 0.650µm and 1% UB with 0.772µm compared to the control specimen which was 0.839µm. Addition of 5% UB in ICEB indicated the highest reduction of pore size compared to the control sample by 23.36%. Therefore, the results from microstructure analysis prove that by addition of UB in ICEB resulted in reducing the pores and precipitation of bacterial calcite. The reduction of pores resulted into the improvement of ICEB properties such as strength, water absorption and durability as the calcite occupied the pores thus, created new bonding between mixed materials and prevented the water penetration. These findings have been proved by the engineering properties testing that was done toward ICEB which indicates that addition of UB improve engineering properties of ICEB compared to control sample.

These findings agreed with Navdeep et al. (2012) studies which indicated the improvement of material engineering properties due to deposition of calcium carbonate.



a) Control: (Average size = 0)

b) 1% UB: (Average size = 9.212 μm)

c) 3% UB: (Average size = 12.469 μm)

d) 5% UB: (Average size = 7.815 μm)

Figure 5. Precipitation of calcium carbonate for control and bacterial ICEB

CONCLUSION

The addition of ureolytic bacteria (UB) significantly improves the strength of ICEB by 10.71% - 15.25% increment compared to control sample. Addition of 5% UB has contributed to an increased of 15.25% in strength at 28th days compared to control sample. The compressive strength increased due to microbial activity which precipitated calcium carbonate, and deposited calcite within the ICEB pores. Evaluation of deposited calcite precipitation was confirmed through microstructure analysis.

Addition of UB in ICEB has significantly effects the water absorption of ICEB. Overall result for initial and water absorption testing, the addition of bacteria (UB) indicated that reduction of water absorption by 7.5% -14.72% of initial absorption and 10.73% -14.68% of water absorption of 24-hour of soaking compared to control sample. The highest reduction recorded was with addition of 5% UB for initial water suction and water absorption at 28th days of testing by 14.72% and 14.68% respectively. The reduction of water absorption was due to the plugging of pores within ICEB also by the precipitation of calcium carbonate in form of calcite.

Microstructure analysis showed that the reduction of pore size in bacterial ICEB sample compared to control sample from average pore of 0.839 to 0.643 (23.36%). Morphology analysis of calcium carbonate shows that precipitation of calcium carbonate ($CaCO_3$) in bacterial specimen seen clearly compared to control sample with average size from 0 to 7.815μm.

ACKNOWLEDGEMENT

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THE EFFECT OF CEMENT AND BAGASSE ASH ON THE ENGINEERING PROPERTIES OF CONTAMINATED SOIL

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Abstract
This study was conducted to investigate how the addition of cement and bagasse ash in contaminated landfill soil affect its engineering properties such as the Atterberg limit, specific gravity, linear shrinkage and soil compaction. Tests were conducted on three different groups of soil samples: landfill soil as the control sample, samples which contained cement only (5%, 10%, 15% and 20% of cement) and samples which contained a mixture of cement and bagasse ash (BA) (2.5% cement + 2.5% BA, 5% cement + 5% BA, 7.5% cement + 7.5% BA and 10% cement + 10% BA). The tests were carried out according to BS 1377 (1990). Initial tests revealed that the soil sample was silty clay soil with low plasticity. It was found that the specific gravity of the soil samples increased from 2.49 (control sample) to 2.79 (soil sample with 20% of cement) when cement was added. However, the specific gravity slightly decreased when bagasse ash was added from 2.49 (control sample) to 2.46% (sample containing 10% cement + 10% BA). The liquid limit increased significantly to 48.80% when the soil sample containing 10% cement + 10% BA was used. In contrast, the plastic limit decreased from 21.31% (control sample) to 19.01% (soil sample containing 20% cement) and 17.92% (soil sample containing 10% cement + 10% BA). There was a decrease in linear shrinkage for samples containing 20% cement and samples containing 10% cement + 10% BA respectively. The addition of cement and bagasse ash increased the optimum moisture content and reduced the maximum dry density of soil. To sum up, the use of cement and bagasse ash in this study improved the engineering properties of contaminated landfill soil.

Keywords: *Landfill Soil Remediation, Engineering Properties, Geo-environmental Engineering*

INTRODUCTION

It is a common practice to utilise agricultural waste for soil stabilisation. Due to the pozzolanic reaction of ash towards cement and lime, agricultural waste ashes have been used as a partial replacement of cement in concrete (Azhar et al., 2016a). Ash can be obtained through the combustion of sugarcane bagasse. Sugarcane bagasse ash consists mostly of silica, followed by other oxides and unburned carbon. Each tonne of crushed sugar cane can produce up to 0.066 tonnes of ash (Azmi et al., 2016). Malaysia produces large quantities of bagasse ash every year. A limited quantity of bagasse ash has been used for soil improvement whereas the rest are usually disposed-off in open landfills. This can lead to environmental problems. Nowadays, bagasse ash has multiple uses. For example, it can be used in the production of bricks or concrete (Sheikh Khalid et al., 2017). It may also have the potential to improve the mechanical properties as well as the durability of concrete (Shahidan et al., 2017). Using bagasse ash as a replacement for cement in concrete production can also help create a more sustainable environment by reducing the need for bagasse ash disposal (Sales and Lima, 2010). In addition, bagasse ash may be effective as a partial cement replacement as previous studies have shown that plant-based fibers can be incorporated with ordinary Portland cement (OPC).

Numerous studies have been done on the use of bagasse ash for the stabilisation of contaminated soil. The studies have shown that bagasse ash has the ability to enhance the engineering properties of contaminated soil (Cordeiro et al., 2008; Chen et al., 2009; Tajudin et al., 2016). However, further research is needed in order to obtain a deeper understanding on the properties of soil incorporated with cement containing bagasse ash. Hence, this study aims to find out the effect of cement and bagasse ash on the geotechnical properties of contaminated soil obtained from landfills.

MATERIALS AND EXPERIMENTAL PROGRAM

Collection of Soil Samples

The contaminated soil samples were collected from Bukit Bakri Landfill Site (BBLS), Muar, Johor. The top of the soil to a depth of 1 meter was removed in order to avoid taking the humus, waste and plant roots. Then, the soil was placed in polystyrene containers. The soil samples were later dried in the oven at 105°C for 24 hours at the laboratory. After that, the dried soil was crushed using a rubber hammer before being decimated into 2 mm in size using a grinder machine. The soil which passes the 2 mm sieve size was stored in polyethylene plastic. The location of the soil sample collection at BBLS, Muar, Johor is shown in Figure 1.

Figure 1. Location of soil sample collection at BBLS, Muar, Johor

Cement and Bagasse Ash (BA)

The primary binder reagent used in the stabilisation/solidification (S/S) method is Ordinary Portland Cement (OPC) type 1. When soft soil is mixed with cement, cement and water react to form cementitious calcium silicate and aluminate hydrates that bind soil particles together (Tajudin et al., 2016b; Bolan et al., 2014). This results in the stabilisation of soil. Sugarcane bagasse ash, which is a type of agricultural waste, was chosen to be incorporated with Ordinary Portland Cement (OPC) in the S/S technique. Sugarcane bagasse ash is also known as a mineral additive in cementitious materials (Tajudin et al. 2016b).

Furthermore, sugarcane bagasse ash is both cost effective and environmentally friendly (Faria et al., 2012; Venkaramuthyalu et al., 2012). Raw sugarcane bagasse was first dried naturally before it was burnt to produce ash. Later, the bagasse ash (BA) was burnt under a controlled temperature of 650o C for an hour. The carbon content was reduced by at least 4.9% due to the burning process (Ganesan et al., 2007). The ash was grounded to 90 μm using a grinder machine after it cooled down.

Production of Soil Samples

The percentage of cement used in the soil samples was partially replaced with bagasse ash (BA). Each sample was carefully weighed to avoid any material waste (Azhar et al., 2016; Tajudin et al., 2015). The quantity of each component in the sample is indicated in terms of percentage in Table 1. Sample batches were triplicated for 3 hydration durations namely 7, 14 and 28 days. To obtain relative homogeneity, the samples were mixed in bulk prior to packing and storage (Azhar et al., 2016b). The water cement ratio in this research ranged between 0.20 to 0.40 (w/c) depending on the quantity of bagasse ash (BA) added. The ratio was fixed based on the optimum moisture content (OMC) obtained from the compaction test. The raw materials were then mixed using a small mixer to ensure the homogeneity of the soil samples. The stabilised soil samples were compacted in a split mould to form samples measuring 38 mm in diameter and 76 mm in height. The mixture was compacted into 4 layers, 50 blows per layer, using a miniature hand compacting tool. The extruded specimens were wrapped and stored for 7, 14 and 28 days respectively before further tests were conducted. The mix design of the soil samples is presented in Table 1.

Table 1. Mix Design of soil samples

Mixing Type	Sample Name	Percentage of Binder (%)		
		Soil	OPC	BA
Soil only (control)	Soil	100	0	0
Cement only	soil + 5% cement	95	5	0
	soil + 10% cement	90	10	0
	soil + 15% cement	85	15	0
	soil + 20% cement	80	20	0
Cement with BA	soil + 2.5% cement + 2.5% BA	95	2.5	2.5
	soil + 5% cement + 5% BA	90	5	5
	soil + 7.5% cement + 7.5% BA	85	7.5	7.5
	soil + 10% cement + 10% BA	80	10	10

RESULTS AND DISCUSSION

Physical and Chemical Characterisation of Raw Materials

At this stage, the physical characteristics of the raw materials used were investigated. The physical specification tests that were conducted included a moisture content test, loss of ignition (LOI) test, Atterberg limit test, specific gravity test and particle size distribution test. The analysis for each test was carried out three times and the results are reported in Table 2.

Table 2. Physical characterisation of raw materials

Test	Soil	OPC	BA	Method
Moisture Content (%)	24.25	ND	10.5 – 15.8	BS1377: Part 2: 1990: 4.3
Specific Gravity	2.49	3.17	1.52	BS1377: Part 2: 1990: 8.3
Liquid Limit, LL (%)	47.15	NT	NT	BS1377: Part 2: 1990: 4.3
Plastic Limit, PL (%)	21.31	NT	NT	BS1377: Part 2: 1990: 4.3

Test	Soil	OPC	BA	Method
Plasticity Index, PI (%)	23.84	NT	NT	BS1377: Part 2: 1990: 4.3
Standard Proctor test (%)	18.69	NT	NT	BS 1377: Part 4: 1990: 3.3
LOI (%)	7.33	4.58	13.04	(Banerjee et al. 2012)

*ND – not detected, NT – not tested

The moisture content was obtained to determine the initial water condition of the raw materials. The moisture content is the mass of pore or free water in a given mass of soil expressed as a percentage of the mass of the dry soil in solid form after drying in oven at 100°C for 24 hours. Table 2 shows the moisture content for landfill soil and bagasse ash which was 24.55% and 10.5-15.8% respectively. The moisture content for OPC was not detected since the value was too small to be measured. This is because OPC is a commercial product available in a dry condition.

There were two separate tests that were conducted for the Atterberg limit test which include both liquid limit, (LL) and plastic limit, (PL). The plastic limit of soil is the moisture content, expressed as a percentage of the weight of the oven dry soil, at the boundary between the plastic and semisolid states of consistency. Based on the test results obtained from RECESS, UTHM, the average value of the plastic limit was 21.31%. Thus, the soil sample was classified as silty clay soil with low plasticity. The plasticity index for clay soil in Bukit Bakri landfill, Muar, was in the range between 17% and 27%.

The loss of Ignition (LOI) value for some materials at a temperature of 650°C was shown in Table 1. The loss of Ignition (LOI) of bagasse ash was found to be high. This is because bagasse ash is an organic material. Furthermore, one of the minerals is calcite ($CaCO_3$) which is volatile at a temperature of 650°C. From the study, the LOI value for OPC was recorded as the lowest (4.58) and the results were almost the same as the values reported by Tarmizi et al. (2012). The LOI content of landfill soil is 7.33 and this soil was found to be almost similar to the LOI for clay.

The particle size distribution for raw samples of soil, cement and ash is shown in Figure 2. The figure shows that the particle size of bagasse ash (BA) is rougher than cement (OPC).

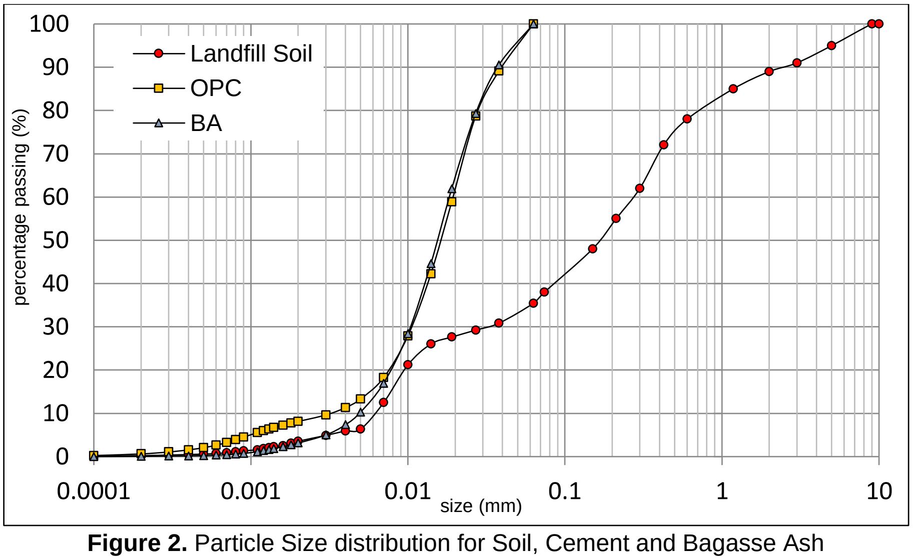

Figure 2. Particle Size distribution for Soil, Cement and Bagasse Ash

The figure also shows that 100% cement, 100% ash and 38% of the soil consisted of silt-sized particles which are less than 75μm. According to the ordinary Portland cement standards, they stipulated that 90% of the particles should be able to pass the 90μm sieve. Therefore, the cement samples used in this study meet the required standards. In the same way, a study conducted by Ganesan et al. (2007) showed that the cement particles exceeding 50μm were unable to be fully hydrated even after the longest curing period. Since 99% of the cement particles used in this study were less than 50 μm, they needed to be hydrated properly. Apart from that, the study showed that soil contains more clay-sized particles.

Due to pH sensitivity during chemical interactions, it was necessary to evaluate the pH of raw material (Du et al., 2014). The table 3 shows the pH value of raw materials used in this study. It was observed that the soil was in an acidic condition with a pH of 4.8. The OPC used in this study was alkaline with a pH of 12.6. The pH of bagasse ash was also slightly alkaline with a value of 10.5. The conductivity tests show that the soil is classified as a non-saline material while ash is moderately saline. On the other hand, cement is strongly saline. This shows that the cement consists of highly soluble solid materials which can affect chemical reactions (Basha et al., 2005; Yao et al., 2012).

Table 3. pH and conductivity of raw materials

Sample	pH	Conductivity (mS/cm)
Soil	4.8	0.16
OPC	12.6	15.66
BA	10.5	5.07

The chemical composition of raw material was determined using the X-Ray fluorescence (XRF) test. Table 4 shows a composition of soil, Ordinary Portland cement (OPC) and sugarcane bagasse ash (BA) as the raw materials used in this study. Based on the XRF results, a similarity was found in terms of the chemical composition for OPC and BA. The results indicated that there are four major elements namely SiO_2, Al_2O_3, Fe_2O_3 and CaO present in OPC and BA. According to a study by Antemir et al. (2010) and John et al. (2011), the main factors which contribute to pozzolanic activities in a binder are silicon dioxide (SiO_2), Aluminium oxide (Al_2O_3), Iron oxide (Fe_2O_3) and a small amount of calcium oxide (CaO). However, the pozzolanic element is often unable to harden on its own. It will harden in the presence of calcium hydroxide and water. From the table, it can be concluded that BA can be used as a partial replacement material for OPC due to the presence of raw pozzolanic elements.

Table 4. Chemical composition of soil, OPC and bagasse ash (BA) (in percentage)

Element	Concentration (%)		
	Soil	**OPC**	**BA**
SiO_2	71.7	20.7	20.37
Fe_2O_3	4.38	3.14	5.63
SO_3	-	-	3.38
K_2O	2.54	1.00	18.78
CaO	0.16	60.8	21.32
Al_2O_3	12.9	7.10	4.85

Element	Concentration (%)		
	Soil	OPC	BA
Cl	-	-	5.3
P_2O_5	-	0.11	6.80
TiO_2	0.64	0.66	0.32
ZrO_2	0.16	-	-
LOI	7.33	4.58	13.04
Total	99.72	98.09	99.79

Effect on Specific Gravity

According to Figure 3, the specific gravity of the soil in this study is 2.49 (control sample) which falls within the range of 2.4 and 3.4 as reported by Basha et al., (2005) for lateritic soils. Adding cement and bagasse ash with a specific gravity of 3.17 and 1.52 respectively to soil produced mixtures with lower specific gravity. Specific gravity values of 2.37, 2.40, 2.43 and 2.46 were obtained for samples containing 2.5% OPC + 2.5% BA, 5% OPC + 5% BA, 7.5% OPC, + 7.5% BA and 10% OPC + 10% BA respectively. In contrast, the specific gravity increased significantly for soil samples containing cement only. The specific gravity values obtained were 2.51, 2.58, 2.63 and 2.79 for samples containing 5%, 10%, 15%, and 20% of cement respectively. The low specific gravity of bagasse ash may have resulted in reduced unit weight (Fatahi et al., 2013).

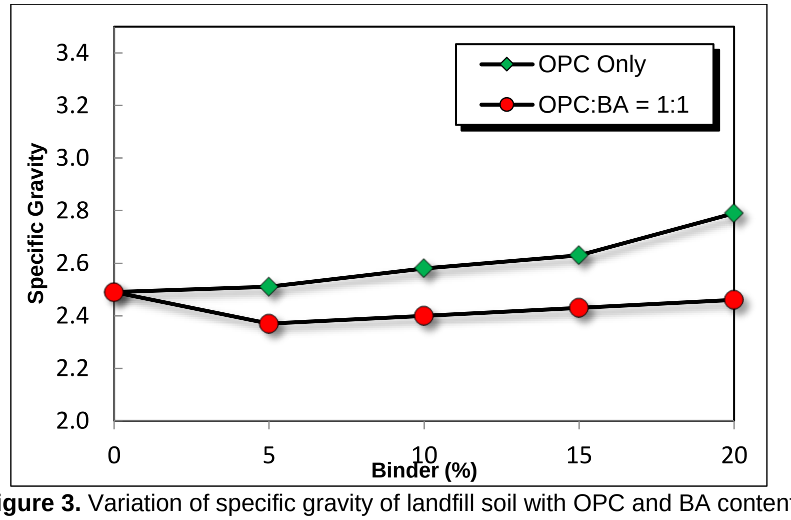

Figure 3. Variation of specific gravity of landfill soil with OPC and BA content

Effect on Atterberg Limits

Atterberg limits are especially useful for controlling soils used in engineered fills. LL is the measure of water content at which the soil behaves like a liquid while PI indicates the magnitude of water content range over which the soil remains plastic. To put it simply, the higher the plasticity index, the higher the potential to shrink due to moisture content fluctuations in soil (Azhar et al., 2016).

The Atterberg limits of landfill soil as well as mixtures containing varying percentages of OPC and BA are reported in Figure 4 to Figure 6. It was found that landfill soil progressively loses its plasticity when the amount of bagasse ash increases due to cation exchange which leads to increased interparticle attraction. For the control sample, the consistency test established the LL as 47.15%. For the soil mixtures containing only cement, the LL values obtained were 50.23%, 50.90%, 51.12%, and 52.77%, for 5%, 10%, 15%, and 20% of cement content respectively. For another group of samples, the LL values obtained were 47.90%, 47.90%, 48.32%, and 48.80% for 2.5% OPC + 2.5% BA, 5% OPC + 5% BA, 7.5% OPC, + 7.5% BA and 10% OPC + 10% BA respectively. Similarly, the plastic limit value for all mixed samples was consistently large enough to increase the plasticity index. Therefore, the resulting PI increased along with the higher OPC content from 30.22% (5% OPC content) to 32.51%, 33.09% and 34.85% for soil containing 10%, 15% and 20% of OPC respectively. In another batch, the plasticity index of samples containing soil mixed with OPC and BA also increased steadily from 26.40%, 27.77%, 28.43% and 29.79% respectively for 2.5% OPC + 2.5% BA, 5% OPC + 5% BA, 7.5% OPC, + 7.5% BA and 10% OPC + 10% BA. To sum up, the increment in Atterberg limits of both OPC samples and OPC + BA samples were small. This may be caused by the calcium ion concentrations in OPC and BA samples used in this study (Sadeeq et al. 2015).

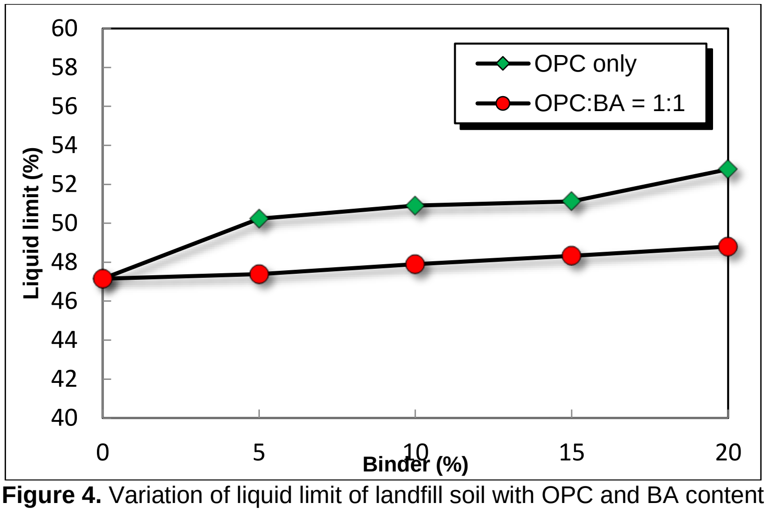

Figure 4. Variation of liquid limit of landfill soil with OPC and BA content

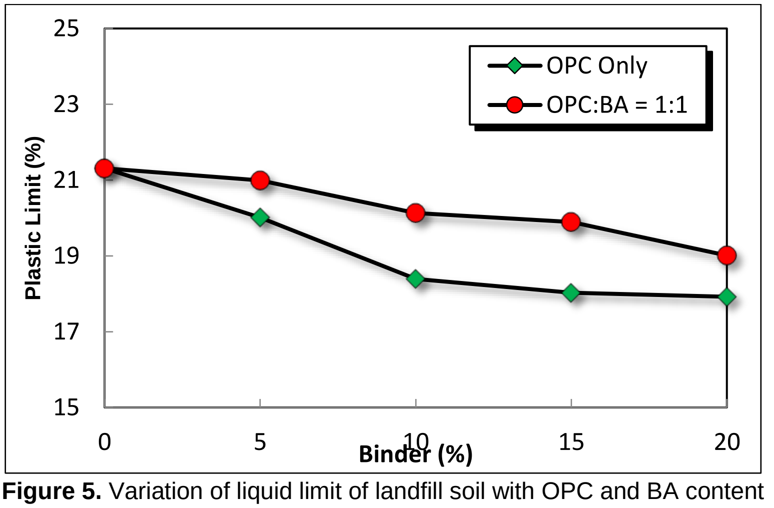

Figure 5. Variation of liquid limit of landfill soil with OPC and BA content

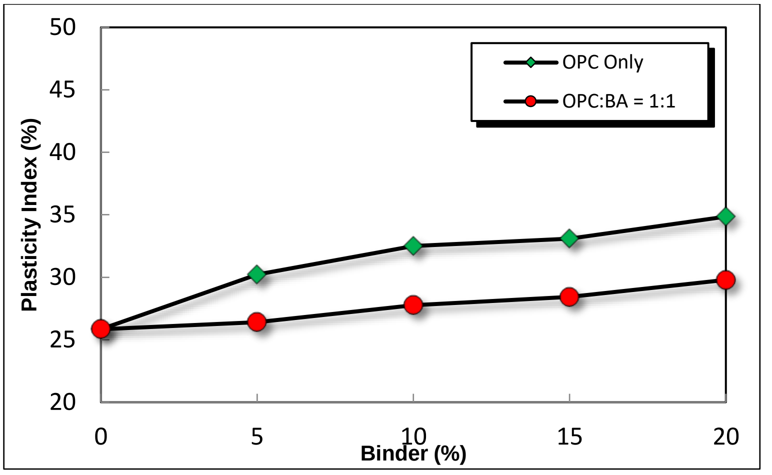

Figure 6. Variation of plasticity index of landfill soil with OPC and BA content

Effect on Linear Shrinkage

A comparison between the linear shrinkage of samples containing only cement and the linear shrinkage of samples containing cement with bagasse ash as additives after a curing period of 28 days is shown in Figure 7. The linear shrinkage decreased considerably when increased quantities of cement and bagasse ash were used. The linear shrinkage values of landfill soil mixed with cement were 15.71%, 13.57%, 11.43% and 6.43% for 5%, 10%, 15% and 20% of OPC respectively. These values were significantly lower compared to the shrinkage value of the control sample (landfill soil only) which was 21.43. Similarly, the linear shrinkage of samples containing cement with bagasse ash showed significantly reduction. Linear shrinkage values of 17.86%, 15.71%, 12.14% and 9.29% were obtained for 2.5% OPC + 2.5% BA, 5% OPC + 5% BA, 7.5% OPC, + 7.5% BA and 10% OPC + 10% BA respectively. The presence of hydrated cement in landfill soil was found to have a more pronounced effect. For example, when 2.5% cement and 2.5% bagasse was added to the soil sample, the linear shrinkage value decreased to 17% compared to the linear shrinkage value of the soil sample containing 5% cement which was 27%. In short, the addition of cement and bagasse ash to landfill soil produced positive results.

Furthermore, the Atterberg limit is widely linked to soil behavior. The linear shrinkage reduced considerably when BA content increased as shown in Figure 7. Linear shrinkage decreased with increasing OPC and OPC + BA content from 21.43% for soil only (control sample) to 6.43% (20% of OPC) and 12.14% (10% OPC + 10% BA content).

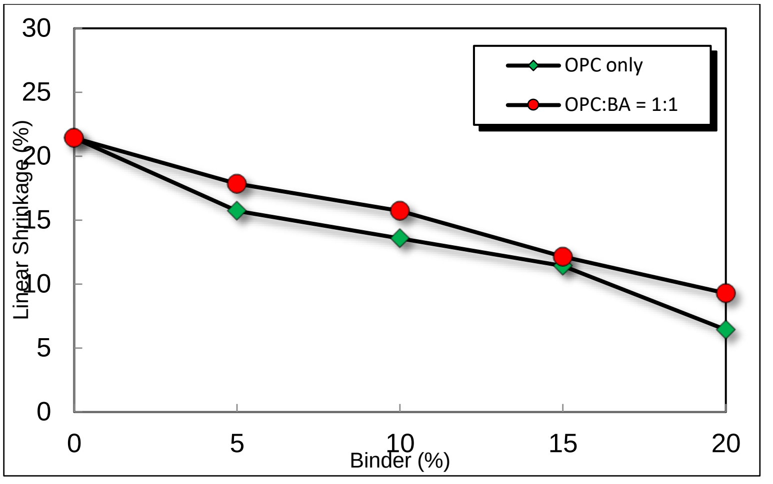

Figure 7. Variation of linear shrinkage of landfill soil with OPC and BA content

Effect on Soil Compactability

The moisture and density at which soil is compacted can influence its geotechnical properties. In general, the geotechnical parameters of soil can be enhanced if there is a high level of compaction (Sadeeq et al., 2015). Therefore, it is important to achieve the desired degree of compaction. The percentage of water present in soil mass at which a specific compaction force can dry the soil mass to its maximum dry weight is known as optimum moisture content (OMC) (Hossain et al., 2011) . The OMC and maximum dry density curves are different according to the type of soil used. In this study, the OMC was obtained via the standard Proctor compaction test.

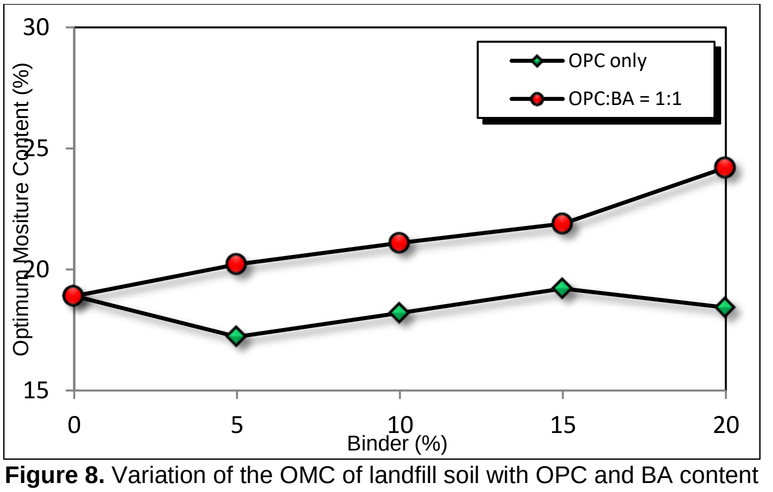

Figure 8. Variation of the OMC of landfill soil with OPC and BA content

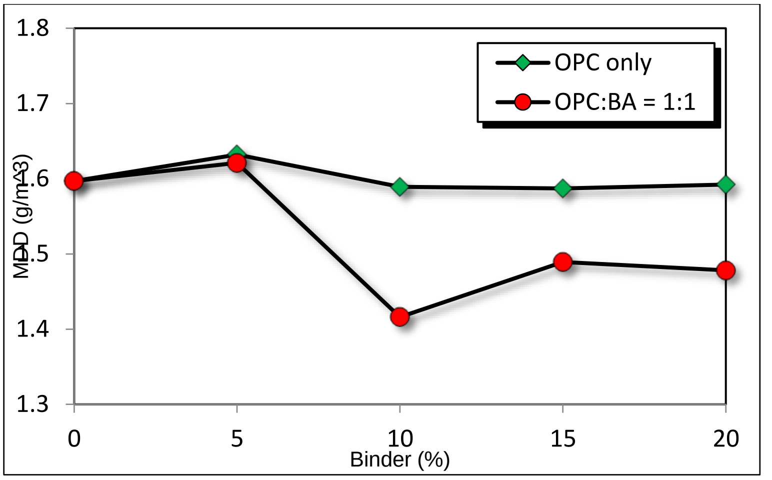

Figure 9. Variation of MDD of landfill soil with OPC and BA content

The effect of the addition of cement alone and the addition of cement with bagasse ash on the compaction characteristics of landfill soil are shown in Figures 8 and 9. The addition of cement and bagasse ash increased the OMC and decreased the amount of MDD. The increase in OMC may be due to the additional water held by the flocculant soil structure resulting from cement interaction as well as excessive water absorption by bagasse ash due to its porous properties, as reported in Hossain et al. (2011). Principally, an increase in dry density indicates improvement. However, both cement and bagasse ash reduce the dry density of all samples except the sample containing 5% OPC and 5% BA. Tarmizi et al. (2012) stated that dry density decreases due to the particle size and specific gravity of the soil and binder. Reduced dry density also indicates that the soil samples need low compactive energy (CE) in order to attain its MDD (Sadeeq et al., 2015). This helps to reduce the cost of compaction.

CONCLUSION

The findings indicate that the samples containing bagasse ash and cement improved the engineering properties of soil a little more compared to the samples containing cement alone. A few conclusions can be made based on this experimental study:

i. Samples containing only cement and samples containing cement and bagasse ash were able to improve the properties of landfill soil. The plasticity index for soil containing cement alone was the highest at 34.85% whereas the plasticity index for soil containing cement and bagasse ash was 29.79%. However, the specific gravity of soil decreased steadily when cement and bagasse ash were added to it.

ii. The linear shrinkage of stabilised soil decreased in samples containing cement alone as well as samples containing cement and bagasse ash. A reduction in linear shrinkage is able to prevent cracks from occurring to concrete structures.

iii. The presence of cement and bagasse ash reduced the maximum dry density of soil. On the other hand, the presence of cement and bagasse ash increased the optimum moisture content of soil.

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THEME 3:

STRUCTURAL ENGINEERING

THE NATURAL FREQUENCY ANALYSIS OF REINFORCED CONCRETE BEAM WITH ADDITIONAL OF LIGHTNING PROTECTION CABLE

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Abstract
Lightning is a phenomena that happens naturally typically during thunderstorms. Malaysia is a country which located near to equator with high lightning activites. Lightning protection system (LPS) plays a vital role to protect the structure by dissipate the lightning current to underground safely. New installation method of LPS has implement in Malaysia which to embed the lightning protection cable in a concrete structure. This research study carried out to determine the natural frequecy of additional reinforcement concrete beam by using impact hammer test. This research study involved the strike of electric current into the additional reinforcement concrete beam which indicate the actual lightning current source to evaluate and compare the natural frequency of beam. The results shown that the natural frequency of additional reinforcement concrete beam has slightly decreases after strike of electric current. From the results also found out that the location of lightning protection cable to attach in the concrete beam will influence the natural frequency of concrete beam.

Keywords: *Lightning Protection System, Natural Frequency, Impact Hammer Test, Strike of Electric Current, Concrete Beam*

INTRODUCTION

Lightning discharge is a natural phenomenon that happens about evert day in the world. It has been estimated that lightning discharge toward the surface on earth around 100 times every second (Hassan et al., 2011). There is also another review stated that in every progressing of 2000 thunderstorms at any time will result in 100 lightning discharges to surface of earth per second, it is approximately equal to 8 million per day (Ab-Kadir, 2016). In addition, an observation from space by using Optical Transient Detector (OTD) reveals that there are nearly 1.4 billion flashes happens annually over the whole world (Christian, 2003). Around 1000 lightning discharges per second or 5000 storms per day happen throughout the world. In France especially in Spain or Italy, approximately 2 million lightning discharges to the ground surface annually (Legrand, 2009). According to a Lightning Protection Guide, it declared that an estimated amount about 1.5 million lightning discharges over a Germany every year (Dehn, 2014). The thunderstorm days of the whole world and it noted that the country located at or near to the equator has the higher lightning density. It reveals that the country that near to equator also has the higher number of lightning. Lightning discharge is been a riddle to people for hundreds years, it is an ordinary but magnificent natural phenomena (Soon, 2004). Malaysia is been described as a country which has huge number of thunderstorm and lightning activities and it is located near to the equator (Abdullah et al., 2008). Besides, Malaysia is also described as lightning core in Asia and the capital, Kuala Lumpur rank 5th in the world in respect of lightning density (Zainudin, 2016). Based on another scholar report, it also stated that Malaysia has been positioned among the main three in the world, more than some other nations in Asia area with referring to the statistic of lightning density (Ab-Kadir, 2016). Lately, they are numerous passing of human life and ruin of electrical equipment aroused by lightning strike had been accounted for.

Lightning strike is a typical occurrence that cannot be hindered, however the safeguard of building can be ensured by install the lightning protection system (LPS). The comprehension of the physical nature of vibration occurrence has constantly been vital for engineer and researchers in industry, considerably more so today as structure are getting to be distinctly lighter and more adaptable because of increment requests for proficiency, speed, security and relaxation (Ashory, 1999; Lemat et al., 2016; Ismail et al., 2016; Shahidan, 2017). To better comprehend the nature of vibration, it comprised the study and analysis of natural frequency. Lightning protection system can provide a safe path for the electric discharge from lightning strike to dissipate to earth, it allow the lightning bolt to bypass the component of building to hinder from damaged. Basically, an entire system of LPS composes of air termination rod, conductor cable, earthing system, connecting among conductors, surge protection devices, and any other connectors that necessary to build the entire system (Arif, 2009; Md Nor et al., 2011; Ramzi et al., 2016; Shahidan et al., 2016). According to IEC/EN 62305-3, down conductors can be embedded into components of reinforced concrete structure such as concrete column (Kokkinos et al., 2006; Shahidan et al., 2011). This method consists of connection among air termination system and embedded LPS with steel reinforcement bars in concrete column, ground beam and pad footing to dissipate the lightning into the earth (Rahim et al., 2016). Embedded system also apply the method of Faraday Cage as the lightning protection cable is install in every corner of the structure or building. An important thing to take note for application of this system is the electricity continuity of steelwork in the reinforced concrete structures in accordance to clause 4.3 in MS IEC 62305-3. The continuity of electricity in steelwork is important because it allows the lightning current to pass through and safely transfer to ground. The connection can be achieved by welded, clamped and etc. referring to the designer of LPS.

There are three types of LPS commonly used in the industry which are Franklin rod, tight wire and faraday cage method. According to IEC/EN 62305-3, there is also another method for installation of down conductors which embed the down conductors into reinforced column of structure (Kokkinos et al., 2006; Senin et al., 2016; Shahidan et al., 2018). There is no specific name for this method of installation but may found in words of embedded system or structural bonding. The embedded system of LPS which the lightning protection cable embedded into the concrete beam is arose due to the unfavourable circumstance of conventional LPS. Conventionally, LPS is installed externally and being expose to the environment has led to the degradation and corrosion of down conductor over the time. Besides, corrosion also occurs gradually as there are existence of galvanized distinct metals in the similar electrolyte (Ghavamian et al., 2015). Due to the external installation method of LPS, it is vulnerable for materials theft because the copper cable generally are exposed barely and accessible on the external of building (Sueta et al., 2007). As a result, extra cost is needed to replace the materials for continue the service life of LPS. There was a literature shown that there are an alternatives installation and design to avoid material theft by using the steelwork frames, foundations and metallic part of the building to be part of the LPS. The embedded system of LPS has brought up because it can be hinder all the unfavorable circumstance of conventional LPS. Impact hammer test sometimes also called as modal test. It is a test that used to determine the modal parameter which natural frequency, modal damping ratios, modal masses and mode shape of test structure. Impact testing always use for experiments to determine the forms of vibration of structure as it is a quick and low-priced (Ahmed and Mohammad, 2015) The objective of this research is to determine, compare and analyze the natural frequency of additional reinforcement concrete beam by using impact hammer test.

MATERIAL AND METHODS

Additional Reinforcement Concrete Beam

There are two samples of concrete beam used in this research and the dimension and details are shown in Figure 2 and 3. The different between sample 1 and 2 are the location of lightning protection cable attached to the steel reinforcement bar. For sample 1 the lightning protection bar ia attached on the top left corner while sample 2 is on bottom left corner. The lightning protection cable will be attached to the steel reinforcement bar by using U-clamp. The material of lightning protection cable is Galvanized Iron (GI). The concrete used in this research are Grade 35. In order to obtain the optimum quality of concrete, ready mix concrete are used. Total 6 number of additional reinforcement concrete beam are prepared for impact hammer test. The detailing of additional reinforcement concrete beam are shows in 2 and 3.. Overall, there are two samples of additional reinforcement concrete beam which sample 1 and 2. The difference between both samples are the location of lightning cable to attach with the steel reinforcement bar; sample 1 is attached at the top left corner while sample 2 is attached at the bottom left corner. Important thing to be take note is the additional reinforcement should be install longer than the actual casting length of concrete beams to ensure it can be connect to the high voltage machine for injection of electric current.

First step of the preparation of sample is the installation of rebar regarding to the detailing of reinforced concrete beam and the additional cable must be attached to the reinforcement steel bar by using U-clamp. Next, follow by the installation of formworks and then the rebar structures are placed inside the formwork and spacers are insert between the rebar and formwork to ensure the nominal cover of concrete is achieved. Then, RMC are ordered from the concrete plant for casting of beam. Once the concrete batch reached, slump test and 3 number of concrete cube are made to test for compression strength in 28 days. During the casting, a vibrator is used to compact the concrete. It is used to eliminate the air bubbles in the fresh pour concrete by shaking vigorously and also to avoid the formation of honey comb. After casting, the reinforced concrete beam will be left for at least one day to dry and then the formwork will be removed. To ease the experiment work, the reinforced concrete beams are placed outside the lab building of electric engineering for connection to the high voltage machine.

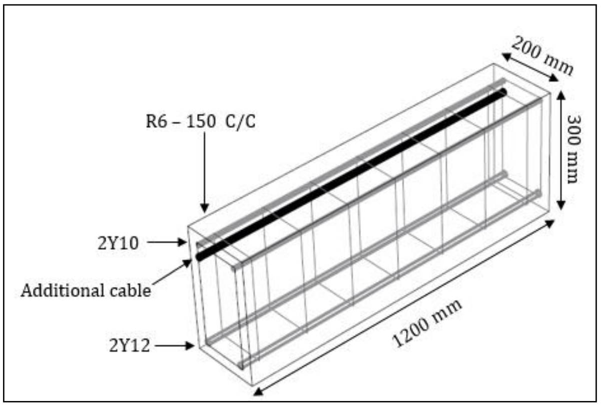

Figure 2. Sample 1

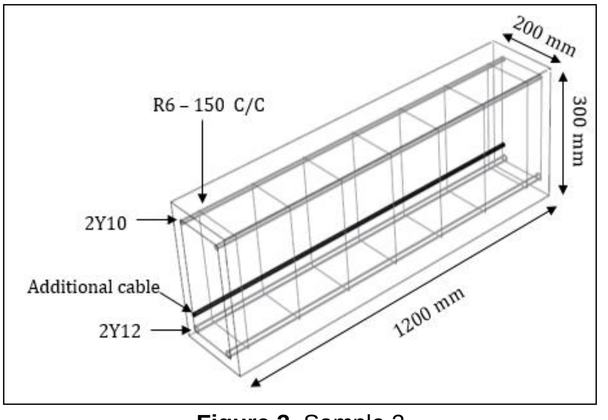

Figure 3. Sample 2

Lightning Stimulation

Each sample has three beam labelled as Beam A, B and C which will subjected to electric current with 15 kA, 30 kA and 45 kA. The electric current indicate the actual lightning current strike to the concrete beam. Thus, the additional reinforcement concrete beam undergo the impact hammer test twice to obtain the results for before and after strike of electric current.

Impact Hammer Test

The vibration behavior, natural frequency, damping ratio and mode shape of the beams were determine using impact hammer test. But the main purpose is to measure the different of the natural frequency value obtained before and after the lightning simulation testing had conducted. The test had been performed in two stages. The first stage is to obtain the natural frequency result at normal condition before the lightning simulation test and the second stage is immediately after the high electric current passed thru the concrete beam. The main idea to conduct this experimental is to obtain as soon as possible any immediate effect of the lightning impact to the concrete beam. The beam specimen is excited by using impact hammer to exert an impact force. The impact hammer struck at two point on the beam as shown in Figure 4. One is on the quarter of beam and another one is on the mid-span of beam. In Figure 5 shows the set-up of equipment. Equipment involved are impact hammer, data logger, accelerators, laptop installed with software and electric current machine. Initially after completed arrangement of the concrete beam, the procedure was started with surface preparation on top of the concrete beam. By using electric grinder the specific location that remarked before was smoothed for the accelerators. This is because the accelerators are too sensitive and the result is not accurate for uneven surface, therefore this process is essential to be conducted. Basically the beam was remarks with the permanent ink for two items. The first remarks is for the location of the hammer points and the secondly remarks for the accelerometers. General arrangement for the remarks was shown in the Figure. The hammer points at the middle span o (1/2 length) and at quarter (1/3 length) of the beam. And the accelerometers point at 5 locations, starting from one end of the beam to the others end of the beam at the interval 300 mm each.

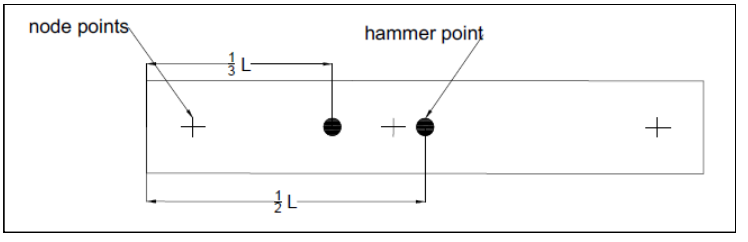

Figure 4. Hammer Point for Impact

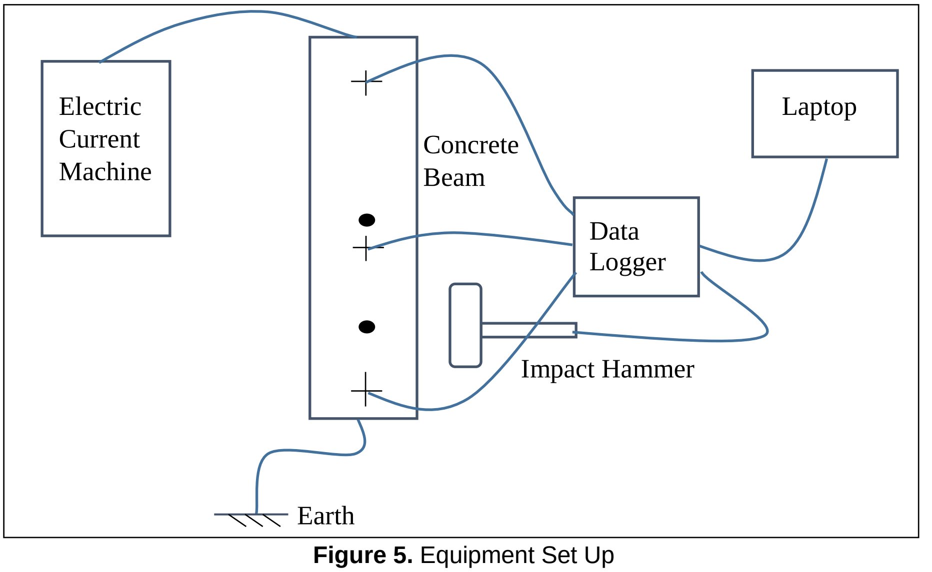

Figure 5. Equipment Set Up

Data Acquisition

The raw data collected from the impact hammer test are acceleration responses in term of time domain. Then, the raw data is transfer to the laptop installed with software to convert into Frequency Response Function (FRF) to extract the modal parameters. The computer-based software is act as an analyzers and recorders to acquire the data by using curve-fitting method. Peak picking method is used to obtain the natural frequency from the FRF by selecting the higher peak of the graph. Raw data is transfer to the laptop installed with software to convert into frequency domain which typically known as Frequency Response Function (FRF) to extract the modal parameters. The computer-based software is act as an analyzers and recorders to acquire the data by using curve-fitting method. Then peak picking method is used to obtain the natural frequency from the FRF by selecting the higher peak of the graph. The obtained results of additional reinforcement concrete beam will be comparing before and after injection of electric current. It is to determine the strength of concrete among the two types of beam. According to Wang et al. (2012), concluded that as the natural frequency of concrete beams increase, the concrete strength will increases. Thus, when the natural frequency of concrete beams decrease indicate that the strength of concrete also decrease.

RESULTS AND DISCUSSION

The obtained results for sample 1 are show in Figure 6 to Figure 8 for Beam A, B and C. For every figure, the FRF in left hand side is the result that obtained before strike of electric current while the right hand side is the result for after strike of electric current. Beam A is subjected to 15 kA of electric current. In Figure 6 shows that before strike of electric current, the natural frequency of beam are 56.1 Hz and 152 Hz while after strike of electric current the natural frequency of beam have reduce to 55.3 Hz and 150 Hz. There are slightly reduction in natural frequency which 1.4 % for first mode and 1.3 % for second mode. The results indicated that the strength of additional reinforced concrete beam reduced when it is subjected to 15 kA of electric current due to the decreasing in natural frequency.

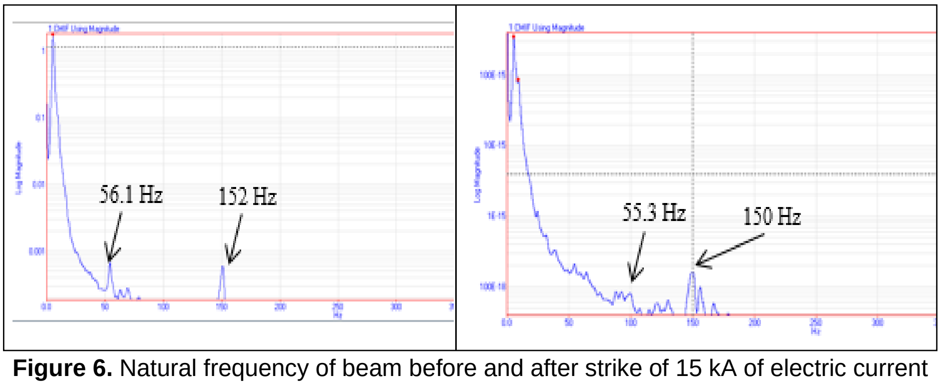

Figure 6. Natural frequency of beam before and after strike of 15 kA of electric current

Beam B is subjected to 30 kA of electric current. In Figure 7 shows that before strike of electric current, the natural frequency of beam are 73.5 Hz and 272 Hz while after strike of electric current the natural frequency of beam have reduce to 71.4 Hz and 254 Hz. There are slightly reduction in natural frequency which 2.7 % for first mode and 6.6 % for second mode. The results indicated that the strength of additional reinforced concrete beam reduced when it is subjected to 30 kA of electric current due to the decreasing in natural frequency.

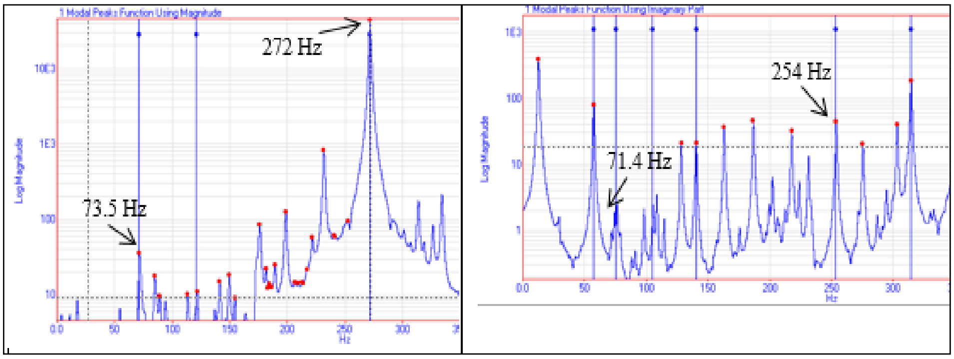

Figure 7. Natural frequency of beam before and after strike of 30 kA of electric current

Beam C is subjected to 45 kA of electric current. In Figure 8 shows that before strike of electric current, the natural frequency of beam are 69.6 Hz and 155 Hz while after strike of electric current the natural frequency of beam have reduce to 68.9 Hz and 155 Hz. There are slightly reduction in natural frequency which 1.0 % for first mode while for second mode it remain unchanged. The results indicated that the strength of additional reinforced concrete beam reduced when it is subjected to 45 kA of electric current due to the decreasing in natural frequency.

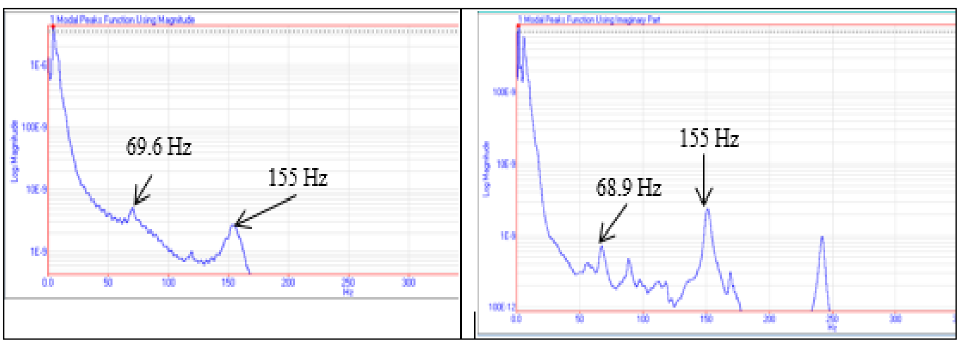

Figure 8. Natural frequency of beam before and after strike of 45 kA of electric current

From the Table 1, it demonstrate that the natural frequency are decrease after strike of electric current for every sample of beam. It indicate the strength of concrete also decreases as the natural frequency decreases. But, the reduction of natural frequency for all sample of beam are in small range which from 1.0 % to 6.6%. Comparing the beam among sample 1, shows that the reduction of natural frequency is decrease as the higher electric current is strike into the concrete beam. It means that the embedded lightning protection cable can withstand the higher electric current without lower the strength of concrete beam. The result shows same to the beam among sample 2 as the reduction of natural frequency is also decrease as the higher electric current is strike into the concrete beam. Comparing the natural frequency of every beam in sample 1 and sample 2, it show that the natural frequency of concrete beam in sample 2 have greater value than sample 1. It means that the strength of additional reinforcement concrete beam in sample 2 is greater than sample 1. It conclude that the location of lightning cable attached in the concrete beam will affect the strength of additional reinforcement concrete beam.

Table 1. Natural Frequency of Additional Reinforcement Concrete Beam

Test Specimen		Natural Frequency (Hz)				Reduction of Natural Frequency in Percentage (%)	
		Before Strike of Electric Current		After Strike of Electric Current			
		Mode 1	Mode 2	Mode 1	Mode 2	Mode 1	Mode 2
Sample 1	Beam A	56.1	152.0	55.3	150.0	1.4	1.3
	Beam B	73.5	272.0	71.4	254.0	1.2	6.6
	Beam C	69.6	155.0	68.9	155.0	1.0	0
Sample 2	Beam A	74.5	151.0	70.7	151.0	5.1	0
	Beam B	97.0	152.0	95.7	150.0	1.3	1.3
	Beam C	166.0	243.0	166.0	240.0	0	1.2

CONCLUSIONS

Through the test of impact hammer, it shown that the objectives of this research study had achieved. From the result of impact hammer test, it shown that the after strike of electric current the natural frequency of concrete beam only has slightly reduction. The reduction of natural frequency also can be interpret as the decrease in strength of additional reinforcement concrete beam. Besides, the comparison of results among the beam of both sample shown that the natural frequency of concrete beam in sample 2 have greater value than in sample 1. It reveals that the location of lightning protection cable attached in the concrete beam will affect the strength of concrete beam. In short, it indicate that the installation method of LPS for sample 2 is effective than sample 1. Besides, the collected result also compared among beams between two samples with strike of same amount of electric current. From the results, it demonstrated that all the concrete beam only has slightly reduction in natural frequency after strike of electric current as compared with natural frequency before strike of electric current regardless of beams in sample 1 or sample 2. For Beam A in sample 1 shown the reduction percentage of natural frequency for mode 1and 2 are 1.4% and 1.3%; Beam B had reduction percentage of 1.2% and 6.6% while Beam C had 1.0% and 0% consecutively. Next, the reduction percentage of natural frequency for beam in sample 2 are 5.1% and 0% for Beam A, 1.3% and 1.3% for Beam B and 0% and 1.2% for Beam C. In addition, when compared the beams between two samples which strike with same amount of electric current, it shown that all the reduction percentage of natural frequency of beam in sample 2 are smaller than beam in sample 1. From the comparison results, it demonstrated that all the beams in both samples are reduced in the natural frequency after strike of electric current regardless of the amount of electric current strike. The reduction of natural frequency can be interpret as the decrease in strength of additional reinforcement concrete beam. Besides, the comparison of results among the beam of both sample which strike with same amount of electric current shown that the natural frequency of concrete beam in sample 2 have greater value than in sample 1. It indicated that the strength of beams in sample 2 higher than strength of beams in sample 1 after strike of electric current. Thus, the location of additional reinforcement attached in concrete beam will influence the strength of concrete beam after strike of electric current. In concluded, additional reinforcement which attached with the bottom reinforcement in concrete beam will result an effective embedded system of LPS than attached with the top reinforcement in concrete beam. In this field. Typical LPS has been well recognized but the recent implementation of embedded system for LPS still has low recognition in Malaysia. It is due to lack of information and researches for the embedded system. In order to design an effective embedded system of LPS, more research related to this system are needed. The following recommendations are offered for research study that related to this field.

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DAMAGE OF REINFORCED CONCRETE BEAMS CONSISTING MODIFIED ARTIFICIAL POLYETHYLENE AGGREGATE (MAPEA) UNDER LOW IMPACT LOAD

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Abstract
The impact damage of reinforced concrete beams subjected to low velocity impact loading at the ultimate load range are explored. In this study, an impact tests is carried out on reinforced concrete beam consisting Modified Artificial Polyethylene Aggregate (MAPEA), where, an approximately 100 kg of impact weight were dropped three times onto the beam specimens until its fails. The waste plastic bags, that encapsulated by glass powder as known as MAPEA were used as the replacement of coarse aggregate. There are twelve beam specimens of size 120 mm x 150 mm x 800 mm are categorized into three groups, where each group consists of 4 specimens. The three groups denoted as normal reinforced concrete (NRC), reinforced concrete with MAPEA concrete block infill (RCAI) and reinforced concrete with 9% of MAPEA as a coarse aggregate (RC9A). All specimens were tested under low velocity impact loads under 0.32 m and 1.54 m (2.5 m/s & 5.5 m/s velocities) drop height of impact weight. The comparisons were made between the three types of beams under the aspect of failure (shear and flexural) and its final displacement. The result of the laboratory test showed that the RC9A beams produced less crack and low value of residual displacement.

Keywords: *Impact, Low velocity, MAPEA, Polyethylene aggregate, Reinforced Concrete beam*.

INTRODUCTION

One of the important considerations for the effective use of structural materials is impact loading. Generally, an impact loading predominantly one of the accountability focuses for structural elements both in steel and concrete. In fact, reinforced concrete (RC) structure under impact loading shows the dissimilar behaviour than statically loaded structures. Thus, this kind of severe loading is one of the dynamic factors that should be considered in the analysis and design of structures. The impact loading is related to conservation energy and momentum law. While, the impactor with mass and velocity strikes the target, it generated the kinetic energy. When the impactor start to touch the target surface, then the impact force is produced and the kinetic energy is partially converted to strain energy in the target. The other part of kinetic energy is converted in any form of energy such as sound, heat, and dissipation through local plastic deformation of target and/or impactor (Lu and Yu, 2003). The strain energy propagated away as stress waves in target. The stress waves as a compressive stress is important role in local damage behaviour of target. If the surface target is unconstrained condition, then the free surface target transmits the reflection of stress wave in longitudinal direction to be a tensile and propagates to the point of surface where it loaded by impact force from the distal face (Beppu et al. 2008; Lu, 2003). If the slab target is brittle and low in tensile, the reflection as tensile wave produce the fracture and part of materials around surface target will be separated and fly away or spalling. Along with that, the compression stress wave also propagate through to rear surface of target or transverse direction. This stress wave will be "carried" and dissipated by the thickness of target. The transverse compressive stress wave obtain the flexural and/or shear failure of the target. However, these failure will not occur if the target is high in tensile (Yankelesvsky, 1997). With regards to this, there has been a growing interest in the past few decades among the engineering community to understand the response of RC structures subjected to extreme loads due to impact. Furthermore, the application of impact tests on concrete had been found in many investigations, such as by Beppu et al. (2008); Damasceno et al. (2014); Dancygier and Yankelevsky (1996); Haldar and Hamieh (1984); Hughes (1984); Mokhatar and Abdullah (2012); Yankelevsky (1997); Zaidi et al. (2014) and Zang et al. (2005).

On the other hand, many countries had issues with waste materials. Generally, the waste plastic bags are one of the major contributions to the increasing number of solid waste production in this country, and only 1 to 3 percent are ever recycled (Khalid et al., 2018). Plastic bags can be a nuisance in the society. It can cause a wastage problem as most plastic bags are not biodegradable if buried. Use of plastic bags caused an adverse impact on the environment as well as being a big problem in landfills. Therefore, innovative solutions have to be developed to solve this problem. One of the considerable/substantial actions is by using waste plastic bags as artificial aggregates in concrete. However, the strengths and behaviour of concrete containing polyethylene (PE) aggregates should be extensively investigated in order to understand the ability of this concrete as a structural material and to ensure that the concrete structure can carry meaningful load without experiencing severe failures under impact loads. Mustafa et al. (2011) found that the compressive strength of concrete containing PE aggregates are lower than normal concrete, however, PE aggregate are suitable for coarse aggregate replacement in concrete through their physical analysis; thus, it is can be used as infill block to replace ineffective volume of RC beam. The incorporation of PE aggregate can significantly improve some properties of concrete because this type of artificial aggregate has high toughness, good abrasion behaviour, low thermal conductivity and high heat capacity (Saikia and Brito, 2012; 2013).

The direct applications of PE-concrete as a concrete structure are still uncertain due to its strength. However, PE-concrete can be used as infill for RC beam. The combination of PE-concrete infill and normal concrete produces so-called composite-based-concrete-structure that has advantage due to lighter weight. RC beam is highly designed to carry compression while steel reinforcements transfer tension stress and loadings. The relationship between transfer tension stress and strain in normal concrete cross-section is almost linear at small value of stress (for stresses less than 40% of the compressive concrete strength). In this condition, the inner part, under neutral axis, only acts as a passive volume that has a small contribution on the crack and failure resistances (Zainorizuan et al., 2016). In addition, when an RC structure is subjected to flexural and shear, the concrete volume under neutral axis of the cross-section is considered ineffective when it is in tension at ultimate limit states.

In this study, modified artificial polyethylene aggregate (MAPEA) is used as the replacement of coarse aggregate in concrete beam, which is made from waste plastic that encapsulated by glass. The use of the waste plastic and glass for constructions can reduce the cost of construction materials and keep the environment clean (Hadipramana et al., 2012). The waste materials such as plastics and glasses could be used as construction materials (Liguori and Iucolano, 2014; Patil et al., 2014; Ali and Al-Tersawy, 2012). In addition, the used of MAPEA results in lighter concrete structure when compared to conventional coarse aggregates. Thus, the aim of this paper is to presents the experiment results of impact behaviour of RC beam with MAPEA as a coarse aggregate in concrete and RC beam with MAPEA concrete block as infill. The details of specimen can be obtained in the following section.

MATERIALS AND BEAMS

Modified Artificial Polyethylene Aggregate (MAPEA)

Most of the products in the world are manufactured from petrochemicals and are non-biodegradable (Kim and Kim, 2008). This products are significant source of environmental pollution and waste in nature (Siti et al., 2017; Ismail, 2016). One of the non-biodegradables materials is the Low Density Polyethylene (LDPE). One of the application of LDPE is in carrier bags and general packaging. Therefore, it was estimated that 12,000 tons LDPE was dump into landfills per day with an average of 20 percent of glass waste.

In this research study, MAPEA was produced from waste plastic bags (Figure 1a), where, the plastic were compacted and heated about 150°C for 20 minutes in oven. Then, the heated plastic hardened and formed the Artificial Polyethylene Aggregate (APEA), as shown in Figure 1b. In order to form MAPEA, the APEA was coated with glass powder and paint (Figure 1c) to produce rough surface. The details of efficiency of glass-covered artificial aggregate has been extensively discussed by Mohamed et al. 2016. Finally, MAPEA was used as a coarse aggregate in the concrete mixture of the RC beam specimens.

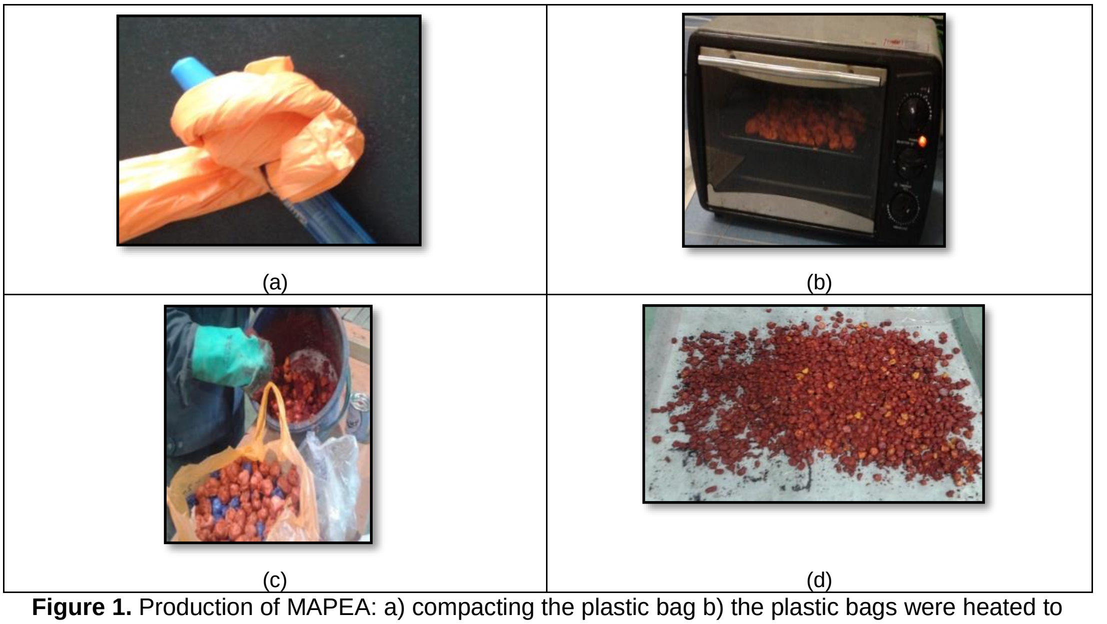


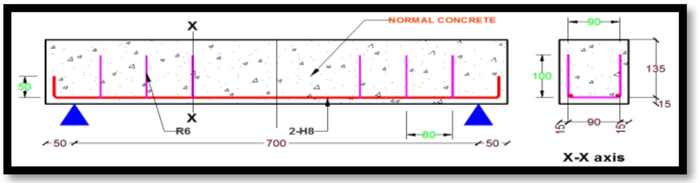
Figure 1. Production of MAPEA: a) compacting the plastic bag b) the plastic bags were heated to form the Artificial Polyethylene Aggregate (APEA) c) APEA encapsulated by glass and paint to produce MAPEA d) MAPEA ready to use as coarse aggregate

Concrete Mix Design

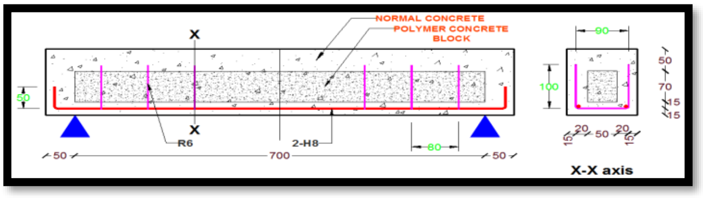
All the RC beams specimens are designed in accordance to Eurocode 2 (EC2) to achieve a compressive strength of 25 N/mm^2 on day 28. All the beam specimens are prepared with the same dimensions of 800 mm length, 120 mm width and 150 mm height. The first group of specimen is normal concrete (NRC) as a control specimen and the second group of specimen is RCAI, which consists of concrete block with 50 mm width, 70 mm width and 790 mm utilizing 100% of MAPEA. The third group is RC beam with 9% of MAPEA as a coarse aggregate replacement (RC9A). The details dimension and volume of each sample are tabulated in Table 1.

Table 1. Size and volume of sample

Sample	Size (Width x Height x Length) (m^3)	Volume (m^3)
Cube	0.15 x 0.15 x 0.15	3.375 x 10^{-3}
Beam	0.12 x 0.15 x 0.80	0.0144
Block	0.05 x 0.07 x 0.79	0.0027



a. Dimension of NRC and RC9A



b. Dimension of RCAI

Figure 2. Dimension of NRC, RC9A, and RCAI beams

IMPACT TEST

Tests for specimen of NRC, RCAI and RC6A were carried out at Jamilus Research Centre, University Tun Hussein Onn Malaysia in order to investigate the low-velocity impact behavior and failure mechanism of the beams. Several measuring devices and testing items are required for development of impact tests in order to guarantee a reliable method of testing and their ability to portray the experiments so that it was similar to real conditions as shown in Figure 3. The test/experiment was individually designed to be able to test beam specimens under impact condition. This research study focused on two impact velocities; (1) 100 kg of impactor was dropped onto the mid span of the beam at different height and (2)the beam specimens were suffered under multiple (3 times) velocity at 2.5 m/s and 5.5 m/s.

Figure 3. Drop weight Impact test equipment

All the specimens were placed on the rigid support and the top surface of the specimens were unconstrained. During the impact tests, the steel frame structures and support conditions should be stiff enough to support the load without significant deformation. The vertical movement of the support conditions during the impact process will affect the displacement and another measurement value. The final displacement and mode of failure for each beam specimens were recorded. To ensure that the test performed correctly, the following checks are carried out:

- Assemble and place the support position in the correct dimension. Set the beam specimen on the support system and align it from the both directions.
- Ensure that the connections between all the components work properly.
- Lift up the drop weight to the desired height and measure the height, finally, set it to fall.

RESULT AND ANALYSIS

Impact Test Results

Table 2 shows the average displacement for each type of beam specimens according to their velocity of impact loading. The displacement of the beam was recorded at the mid-span of the beam as the critical point occurred at this zone. The comparison of the value of displacement was made between the three types of beams. Based on the result, the lowest displacement value was observed in RC9A for all 2.5 m/s impact velocity. This indicates that the MAPEA absorb flexural energy impact for 2.5 m/s impact velocity. Whereas, for bigger energy impact velocity, RC beam with MAPEA block (RCAI) possessed the lowest displacement after the first impact. It indicates that the compressive stress wave in transverse direction was distributed by MAPEA block in RCAI. However, the scabbing and spalling occurred after the second impact on RCAI compare to others beams (Table 3). This local failures occurred due to the poor bonding between MAPEA and the cement matrix inside the beam. This local failures, the so called ‘bell plug shape’ were also observed by other researchers (Yankelevsky, 1997).

Table 2. Velocity and average displacement

Beam	Velocity (m/s)	Displacement (average) (mm)		
		1st impact	2nd impact	3rd impact
NRC	2.5	4.71	4.83	6.32
	5.5	13.74	27.10	N/A
RCAI	2.5	3.85	5.54	8.86
	5.5	11.40	26.17	N/A
RC9A	2.5	2.71	3.45	4.49
	5.5	12.36	19.27	N/A

N/A: Not available

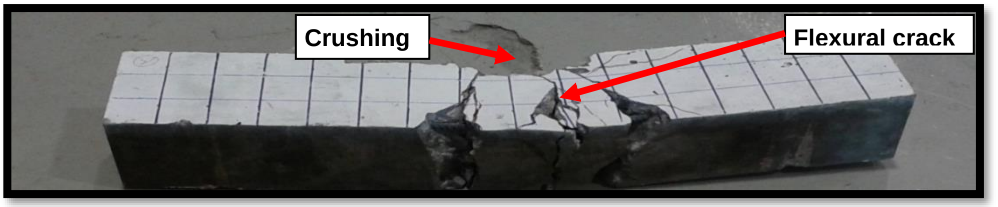
Table 3. Local failure of RC Beam

Beam	Impact Velocity (m/s)	Shear Crack	Flexural Crack	Scabbing	Spalling	Crush
NRC	2.5	√	-	-	-	-
	5.5	√	√	-	√	√
RCAI	2.5	√	√	-	-	-
	5.5	√	√	√	√	√
RC9A	2.5	√	√	-	-	-
	5.5	√	√	-	-	-

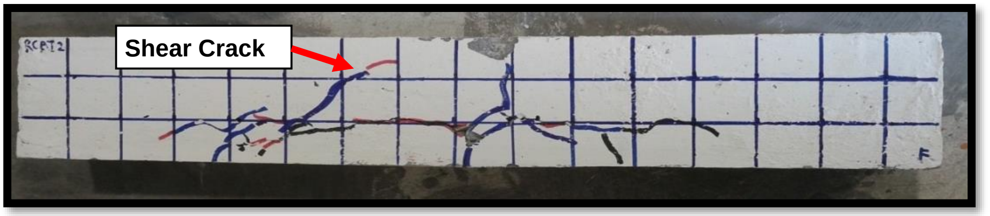
As can be seen in Figure 4, Figure 5 and Figure 6, shear crack occurred at all beams. However, RC9A could withstand the scabbing, spalling, and crushing by both impact velocities (Figures 6a and 6b). In contrast with RCAI and RC9A, the flexural crack did not occur at NCR, when it was subjected to 2.5 m/s impact loading (Figure 4a). This is due to elastic stress of NCR where it could overcome the compressive stress wave. Nevertheless, the 5.5 m/s impact velocity was beyond the elastic stress of NCR, therefore the flexural crack was observed in this beam specimens (Figure 4b).

Figure 5a shows that scabbing and spalling did not occur when RCAI was subjected to 2.5 m/s impact loading. It indicates that the MAPEA block increase the plastic stress of beams and had overcome the transverse stress wave due to 2.5 m/s impact loading. However, the plastic stress that generated by MAPEA was not enough to withstand the longitudinal (Lu, 2003) and transverse stress wave that produced by 5.5 m/s impact loading. Hence, the spalling and scabbing was observed in MAPEA block (Figure 5b).

a) NCR subjected to 2.5 m/s velocity

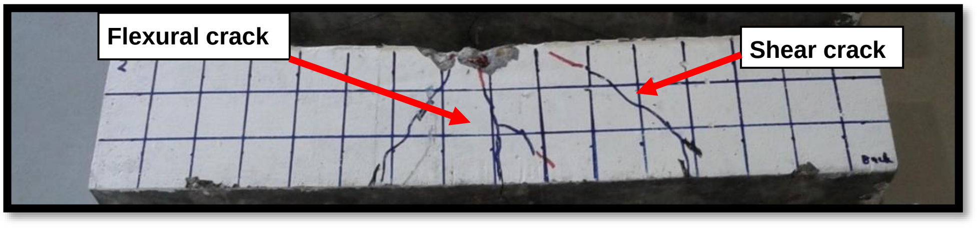


b) NCR subjected to 5.5 m/s velocity
Figure 4. NRC beam under impact



a) RCAI subjected to 2.5 m/s velocity

b) RCAI subjected to 5.5 m/s velocity
Figure 5. RCAI beam under impact



a) RC9A subjected to 2.5 m/s velocity

b) RC9A subjected to 5.5 m/s velocity
Figure 6. RC9A beam under impact

CONCLUSION

The used of MAPEA in RC beam leads to 10% reduction in weight when compared to traditional RC beams. More research study needed to be done on RCAI beams so that it can absorb more energy during the impact, thus make it high in strength. As for RC9A beams, it possessed lowest displacement under the impact load test of velocity 2.5 m/s and 5.5 m/s compared to RCAI beams. The difference in trend of displacement for beams specimens under 5.5 m/s velocity was due to the difference in effectiveness when there was increased in velocity.

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EXPERIMENTAL STUDY ON THE VIBRATION RESPONSES AND ENERGY DISSIPATION OF FOAMED CONCRETE COMPOSITE SLABS

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Abstract
Composite slab is an efficient replacement of the conventional reinforced concrete slab. It is usually consists of corrugated steel deck and concrete topping. In this experimental study, foamed concrete was introduced as concrete topping to reduce the self-weight of composite slab. More focus was given on the effect of thickness toward the natural frequency, damping ratio and energy dissipation of composite slab. The composite slab was cast with the size of 840mm width, 1800 length and five different thicknesses range between 75mm to 175mm. Foamed concrete was deliberately designed for the density of 1800kg/m^3 with the utilization of rice husk ash (RHA) as partially sand replacement and polypropylene mega-mesh (PMM) as fiber reinforcement. Throughout the compression and splitting-tensile tests, foamed concrete achieves a decent quality with value of 35.03MPa and 2.01MPa for the compressive and tensile strengths respectively. It was observed that the natural frequency decreases with the increment of thickness. On the other hand, the damping ratio and energy dissipation indicate a significant increment correspond to the thickness of composite slab.

Keywords: *Natural frequency; Structural damping; Energy dissipation; Composite slab; Foamed concrete; Corrugated steel deck*

INTRODUCTION

In recent years, composite slab has become one of the important structures in building. Usually composite slab is used in commercial and industrial buildings, but can also be extended to the residential and leisure buildings. The motivation for using the composite slab is mainly due to the time frame of construction and to achieve the required economical aspect. Composite slab is known as a structure that consists of reinforced concrete on the top of corrugated steel deck. Therefore, composite slab is widely referred as corrugated steel-concrete slab. BS EN 1994-1-1 (2005) defined the composite slab as a structure like a slab in which corrugated steel deck is used initially as permanent shuttering, subsequently combine structurally with the hardened concrete and act as tensile reinforcement.

In conventional practise, normal concrete is used to give a perfect interaction with corrugated steel deck. Normal concrete is the mixture of cement, sand, coarse aggregate and water. The density of normal concrete is around 2400kg/m^3 while the compressive strength can achieve as high as 50MPa. It is widely known that normal concrete contributes to the weight penalty. Therefore, for an efficient design and casting of composite slab, foamed concrete is preferable as it can offer lightness and versatility. Foamed concrete comprises of mortar matrix with minimum 20% of entrapped air-void and can be manufactured easily with good workability, excellent performance of thermal insulation, fire resistance and absorption (Narayanan & Ramamurthy, 2000; Jones & McCarthy, 2005; Mydin & Wang, 2012).

However, the utilization of foamed concrete in composite slab is relatively new and require comprehensive investigations. There is a grave concern that the application of foamed concrete may cause a greater transient behaviour on composite slab and probably lead to undesirable vibration responses. Until recently, the literature reviews and past investigations on the vibration responses are solely limited to the composite slab made of normal concrete. Therefore, this study aims to investigate the vibration responses of foamed concrete composite slabs in term of natural frequency, structural damping and energy dissipation. Moreover, the susceptibility on the vibration responses of foamed concrete composite slab must be tackled to prevent discomfort and outright structural failure.

COMPOSITE SLAB

Composite slab, as shown in Figure 1, is structurally efficient because it exploits the tensile resistance of corrugated steel deck and compressive strength of concrete (Zoltan & Istvan, 1998). It is widely used due to its advantage compared to reinforced concrete slab where it can reduce size of structural components and increase the resistance of stresses. According to Rackham et al. (2009), composite slab able to reduce around 30% of weight if properly associated with the steel frame structures. As a result, the demand on composite slab increases in construction industry (Lam et al., 2017). However, the interaction between concrete and corrugated steel deck become paramount important in load transfer, effective utilization of material, interfacial characteristics and durability of composite slab (Yoshitake et al., 2012).

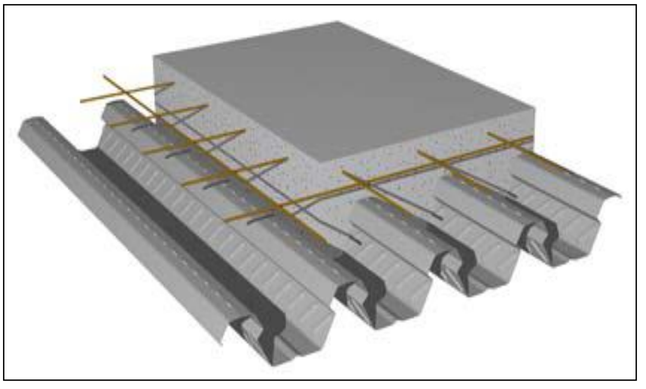


Figure 1. Composite slab consists of corrugated steel deck and reinforced concrete

On the other hand, composite slab is regarded as the best diaphragm strengthening method for the building. Previous experience indicates that this kind of diaphragm generally allows lower slab thickness, higher load bearing capacity and size reduction for structural components of beams, columns and foundations (Abbas, 2014). Therefore, composite slab is very useful for seismic building system. If fully associated with steel beams and infilled steel columns, it can create high ductility of structural components to resist seismic events. Chaudhari et al. (2014) stated that after the recent earthquakes in New Zealand, there is wide acceptance of steel frame structure with composite slab, where the high performance can be achieved using steel studs welded onto the beam and cast into the concrete slab.

Many investigations were conducted on the design, structural behaviour and performance of composite slab made of normal concrete. Bayasi et al. (2001), Sanchez et al. (2011), Khalaf et al. (2013) and Chaudhari et al. (2014) investigated the structural behaviour of composite slab. On the other hand, Jeong et al. (2009), Cifuentes & Medina (2013), Johnson & Shepherd (2013), Lakshimikandhan et al. (2013) and Abbas et al. (2014) investigated the interaction and shear resistance of normal concrete with corrugated steel deck. However, Johnson & Li (2012) proposed the application of foamed concrete in composite slab and found that the bearing capacity is identical with counterpart from normal concrete. For the better performance and durability, foamed concrete can be reinforced using fibers.

The strength of composite slab is fully governed by shear interaction between concrete and corrugated steel deck, rather than by yielding of the corrugated steel deck. Therefore, the connection and bond of concrete to corrugated steel deck play a crucial role in creating the compositeness and to prevent the slip of composite slab. Composite slab is normally used to span between 3.0m to 4.5m, where the thickness can be varied from100mm to 250mm for shallow decking and in the range 250mm to 300mm for deep decking (Feldmann et al. 2013). This type of slab has spanning behaviour for one-way with no account taken on the continuity offered by reinforcement. That make it more vulnerable to vibration compare to the two-way span reinforced concrete slab (Silva & David, 2009).

Usually, composite concrete is designed using static method which will not reveal the true behaviour under dynamic load that induced by vibration. Generally, vibration is a repeated motion and extensive movement in a period of time (He and Fu, 2001). Acceleration, velocity and displacement are related matters to vibration. The external source of vibration can be generated from traffic, ground-borne activities and high localised loads. Meanwhile, the internal source of vibration is mostly about the human-induced resonant or machine-induced vibration. The vibration from traffic and seismic are considered the most critical sources of excitation of high vibration that need major concern (Hao & Cheng, 2001; Pridham, 2009). Hunaidi (2000) stated that the external sources of vibration are likely to affect structures and the sensitive equipment that housed in the building.

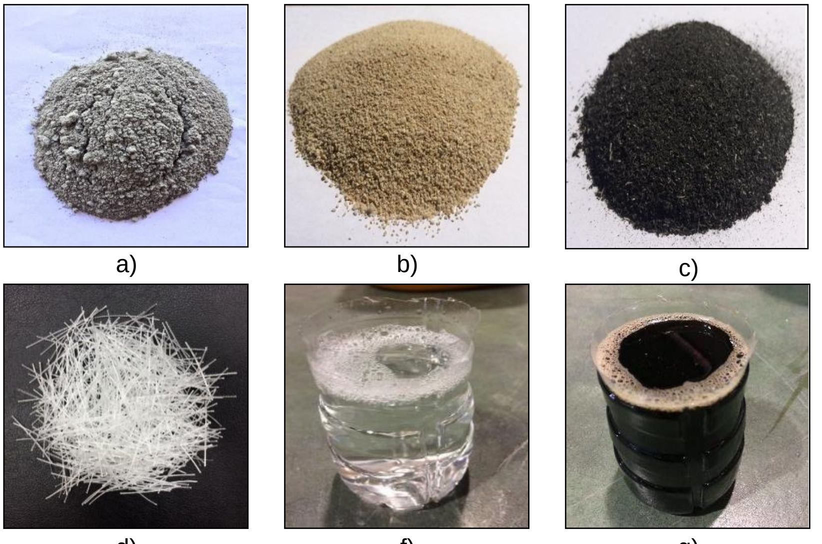
Varela & Battista (2011), Fahmy & Sidky (2012) and Abeysinghe et al. (2013) investigated the vibration responses of composite slab made of normal concrete under human-induced resonant. The consequences of investigations reach a similar outcome that indicate the occurrence of multi-modal vibration. It was also found that the vibration caused by the higher harmonics of the frequent activities can also cause discomfort and excessive deflections. Chaudhari et al. (2014) emphasized that the continuous vibration can increase the demand on the connection, panel zone and column, hence possibly resulting in some undesirable inelastic deformation mechanisms. However, a substantial reduction of vibration can be achieved by controlling the structural stiffness.

Any study on the vibration responses has a close look with the natural frequency and associated mode shape as structural degrees of freedom. The vibration can be commonly sorted by the amount of energy that is activated by the oscillation. Therefore, the natural frequency is that on the lowest energy level and thus the most likely to be activated. Murray et al. (2003) and Amick & Grodon (2005) proposed the acceptance criteria of vibration for the specific function of slabs using the correlation of peak acceleration and natural frequency. On the other hand, Klein & Rianer (1995) recommended that the natural frequency of concrete slabs to be between 10Hz and 30Hz. According to Feldmann et al. (2013), for the comfort criteria of composite slab, the natural frequency should be not less than 8Hz.

EXPERIMENTAL STUDY

Material preparation

In preparing the materials for the casting of cube and cylinder specimens as well as the composite slabs, the mix design of foamed concrete was based on the volume rather than the weight. In addition, the mix design was purposely designed to produce foamed concrete with targeted density of 1800kg/m^3. The materials, as can be seen in Figure 2, include the ordinary Portland cement (OPC), sand, rice husk ash (RHA), polypropylene mega-mesh 55 (PMM), foam agent, superplasticizer (SP) and water. A mix design as suggested by Abd Rahman et al. (2015) and Jaini et al. (2017) was employed in the production of foamed concrete, where the cement-sand ratio is 0.50, water-cement ratio is 0.55, foam agent-cement ratio is 0.07 and foam agent-water ratio is 0.05.



a) b) c)

d) f) g)

Figure 2. Main materials in foamed concrete; a) OPC, b) Sand, c) RHA, d) PMM), f) Foam agent and g) SP

The detail about required quantity of materials can be referred in Table 1. In this experimental study, foamed concrete was produced using prefoaming method. The preformed foam was produced by diluting the foam agent with water in an LCM generator. The function of preformed foam is as a stable substance that added in mortar matrix for the desired density of foamed concrete. RHA was utilized as partially sand replacement at 40% of volume fraction. RHA was obtained from the combustion process at 700°C for 6 hours. The composition of RHA consists of 89.90% SiO_2, 0.46% Al_2O_3, 0.47% Fe_2O_3, 1.01% CaO, 0.79% MgO, 0.11% Na_2O^3 and 0.32% LOI, which indicates the presence of highly amorphous silica. Meanwhile, 9kg/m^3 PMM was added in foamed concrete to enhance the mechanical properties.

Table 1. The required quantity of materials

Material	Quantity for 1m³ (kg)	Required quantity (kg)	Remarks
Cement	453.30	809.61	Ordinary Portland cement, Type I based on BS EN 197-1
Sand	728.00	1355.17	Size less than 3mm based on BS 882:1992
RHA	141.30	263.09	Grey colour with average particle size around 5µm to 105µm
PMM	220.00	7.84	Synthetic fiber with length of 55mm and tensile strength of 425MPa
Foam agent	N/A	N/A	Sika AER 50/50 which is a synthetic surfactant and polymer-based
SP	2.49	4.45	ESTOP Admix AP with consistence content along the mixing
Water	249.30	445.28	Tap water free from impurity based on BS EN 1008:2002

Specimen preparation

A total of 9 cube specimens of foamed concrete containing 40% RHA and 9kg/m^3 PMM were prepared using mould of 100mm length, 100mm width and 100mm depth. The cube specimens were underwent air curing at 7, 14 and 28 days. On the other hand, a total of 9 cylinder specimens with size of 300mm height and 150mm diameter were also produced The cylinder specimens were placed at the ambient condition for the curing process as similar duration as cube specimens. Meanwhile, a total 10 composite slabs with dimension of 1800mm length, 840mm width and various thicknesses of 75mm, 100mm, 125mm, 150mm and 175mm were prepared.

The schematic design of composite slab is illustrated in Figure 3. The PEVA45 with thickness of 1mm was employed as the corrugated steel deck and at the same time acts as formwork. In addition, foamed concrete was reinforced with mesh reinforcement of H8-250. The composite slab was allowed to adopt the nominal cover of 35mm. No shear connections or studs were placed in between foamed concrete and corrugated steel deck. Therefore, the shear strength of composite slab is solely depended on the bond of foamed concrete and corrugated steel deck. Figure 4 shows the formwork that ready to be filled with foamed concrete and hardened composite slab that underwent the air curing process along 28 days.

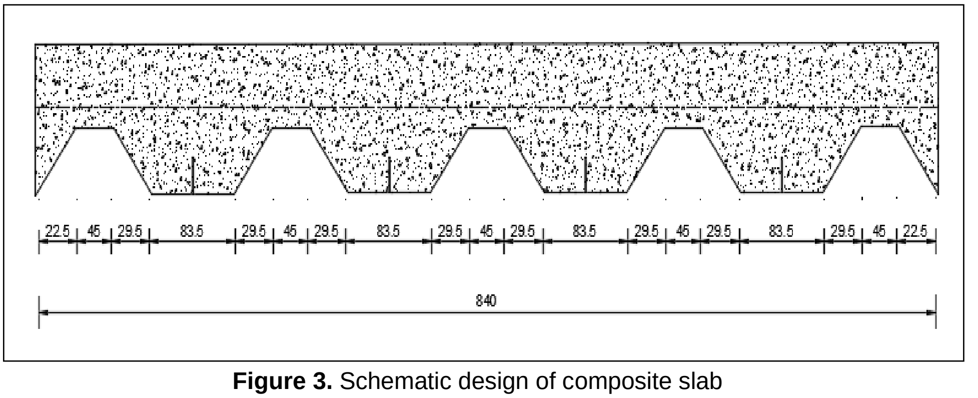
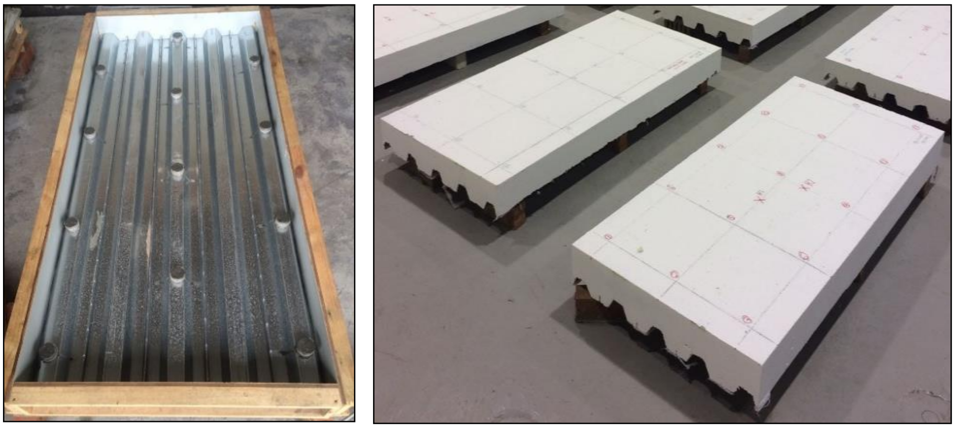


Figure 3. Schematic design of composite slab



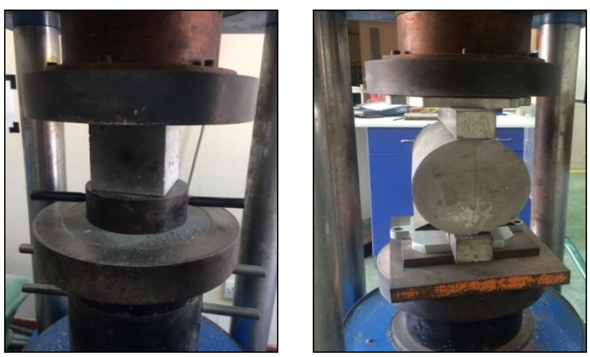
(a) (b)

Figure 4. Specimen preparation of composite slab: (a) Mould and (b) Hardened composite slab

Test Programme

Compression and Splitting-Tensile Tests

The compression test on cube specimens was conducted according to BS EN12390-3:2009. On the other hand, BS EN12390-6:2001 was referred for the splitting-tensile test to obtain the tensile strength of foamed concrete. Both compression and splitting-tensile tests were performed using Ele Compact Machine 1500 as can be seen in Figure 5. Before the tests, cube and cylinder specimens were visually inspected, labelled, sized and weighted.



a) b)

Figure 5. Test programme on cube and cylinder specimens, a) Compression test and b) Splitting-tensile test

Hammer-Impact Test

The composite slabs were tested under the hammer-impact test after 28 days of curing process. The hammer-impact test requires special instruments of hammer, accelerometers and data logger as can be seen in Figure 6. The hammer was designed for use with delicate structure. The force transducer on hammer measures the impact force that applied to the surface of composite slab. Meanwhile, the accelerometers were used to measure the wave vibration due to the impacted of hammer. The wave vibration was recorded as acceleration-time history (in relation to location) by data logger and can be displayed in both plot and digital using QuickDAQ. The set-up of hammer-impact test can be seen in Figure 7.

a) Hammer b) Accelerometer c) Data logger

Figure 6. Instrument of hammer-impact test

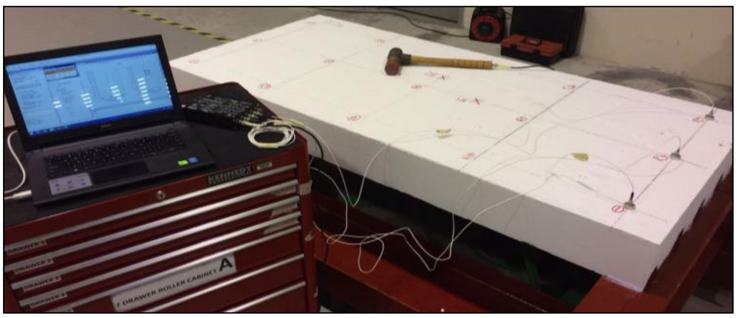


Figure 7. Test set-up for hammer-impact test

Fifteen points for accelerometers were drawn on top surface of composite slab as shown in Figure 8. Points of A1, A2, A3, A4 and A5 are classified as Series A, while Series B consists of B1, B2, B3, B4 and B5. Points for Series C are dictated by C1, C2, C3, C4 and C5. The hammer was stroked at point X which located at one-third span of composite slab. The hammer was stroked for ten times for the accuracy of data and the wave vibration at A1, B1 and B2 were simultaneously captured. The processes were repeated for A2, B2 and B3 until A5, B5 and C5.

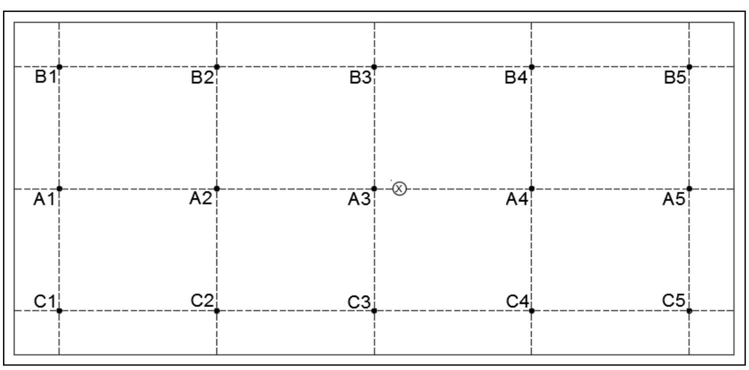


Figure 8. The position of the accelerometers during the test

The raw data obtained from the impact hammer test were examined to get the oscillation of wave propagation and acceleration-time history, consequently the vibration responses and energy dissipation can be determined. The vibration responses are referred to the natural frequency and structural damping. In order to determine the natural frequency, the average time of natural period, T needs to be determined using the following Equation 1:

$$T = \frac{(T_f - T_i)}{n} \tag{1}$$

where T_f is the final time of last completed sine cycle, T_i is the initial time of first completed sine cycle and n is the number of completed sine cycle. The natural frequency, f of composite slab can be easily solved using the following Equation 2:

$$f = \frac{1}{T} \tag{2}$$

On the other hand, the empirical approach provides a simple calculation of the natural frequency, such as Equation 3:

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{3}$$

where k is the structural stiffness while m is the mass of composite slab. The structural damping, D_i can be determined using the following Equation 4:

$$D_i = \frac{1}{2\pi n}\ln\frac{a_i}{a_j} \tag{4}$$

where a_i and a_j are the accelerations that generated from the hammer-impact test. Alternatively, the structural damping can be determined using the following Equation 5:

$$D_i > 35 f a_i + 2.5 \tag{5}$$

The energy dissipation, E_p can be measured using the following Equation 6:

$$E_p = 4 f \xi K_e \tag{6}$$

where K_e represents the kinetic energy that can be calculated using the following Equation 7:

$$K_e = \frac{1}{2}M_e\left[a_p\left(\frac{T_1}{2\pi}\right)\right] \tag{7}$$

where M_e is the effective mass of composite slab, a_p is the peak acceleration and T_1 is the corner period for 1 complete cycle of vibration.

RESULTS AND DISCUSSIONS

Compression and Tensile Strengths

The compressive strength of foamed concrete correspond to the curing age is shown in Figure 9. At 7 days, the compressive strength achieves 26.18MPa that reveal the early strength development. The compressive strength boost around 19.52% to reach 31.59MPa at 14 days. Foamed concrete accomplishes a decent quality with the compressive strength of 35.03MPa at 28 days. This 94.61% increment than conventional foamed concrete surpasses the minimum requirement as set by the code of practise, in which Eurocode 2 enforces 22MPa. It can be confirmed that the addition of RHA improves the compressive strength of foamed concrete as agreed by Bayuaji (2015). Other than that, PMM and SP are also contribute to the better material properties of foamed concrete (Jaini et al., 2017; Rum et al., 2017).

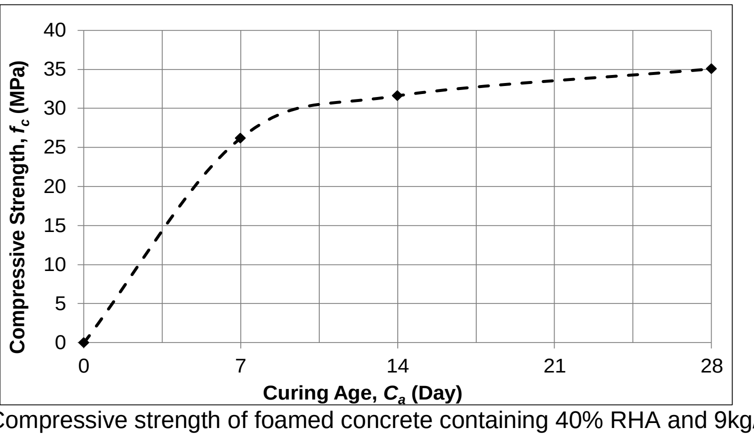


Figure 9. Compressive strength of foamed concrete containing 40% RHA and 9kg/m^3 PMM

Meanwhile, Figure 10 shows the tensile strength of foamed concrete. The tensile strength was observed to increase throughout the curing age. Foamed concrete developed tensile strength at 1.46MPa within 7 days. At 14 days, the tensile strength increases around 13.1% to has value of 1.67MPa. It can be observed that the tensile strength is progressively increased at 28 days to achieve as high as 2.01MPa.

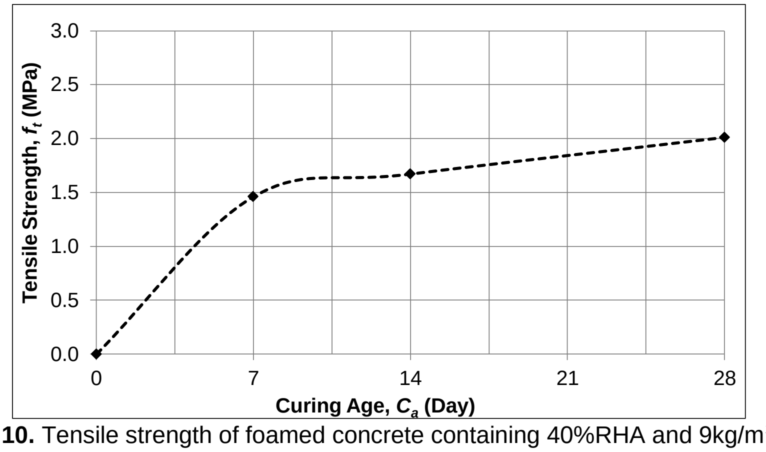


Figure 10. Tensile strength of foamed concrete containing 40%RHA and 9kg/m^3 PMM

In this experimental study, foamed concrete achieves an exceptional tensile strength and almost similar with that offer by normal concrete. Hilal et al. (2015) revealed that the tensile strength of foamed concrete is basically quite low with the average value around 1.5MPa only. It can be realized that there is a substantial increase in the tensile strength due to the addition of RHA. However, Jaini et al. (2017) suggested that the presence of PMM that obviously lead to the satisfactory of the tensile strength. The confinement provided by PMM was found able to increase the bonding of foamed concrete and consequently improve the toughness of foamed concrete in arresting the crack propagation.

Natural frequency

Based on the acceleration-time history from the hammer-impact test, the natural frequency was analysed using Equation (2). Figure 11 shows an example of acceleration-time history that was measured at the center of composite slab CS-125. This acceleration-time history was occurred subjected to the impact force that depicted in Figure 12. The acceleration-time history resembles the transient response that regularly happen on the floor system due to excitation of heel drop or single-step of working person.

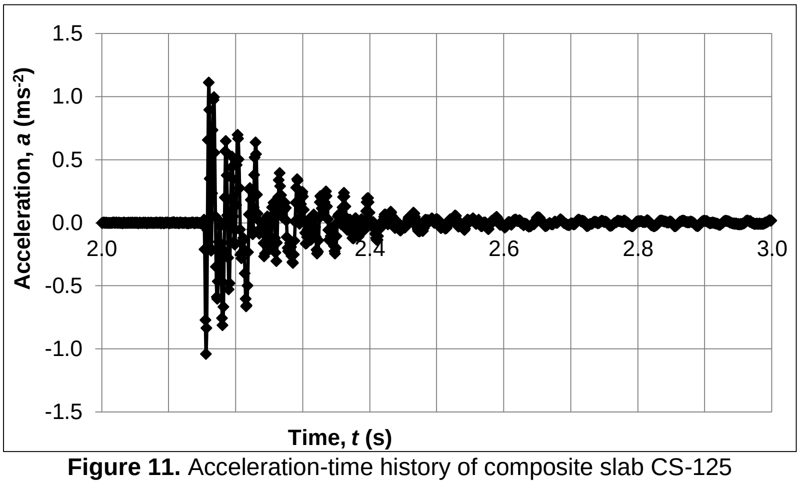


Figure 11. Acceleration-time history of composite slab CS-125

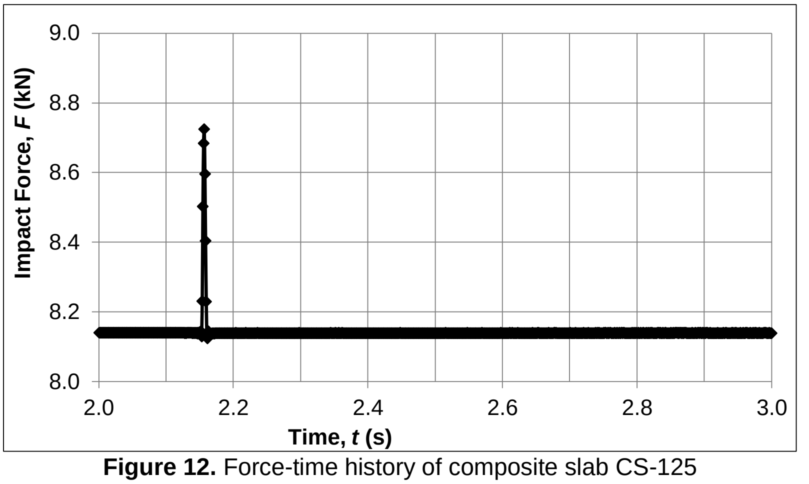


Figure 12. Force-time history of composite slab CS-125

In the design consideration of composite slab, the thickness of concrete topping plays a significant role in governing the serviceability. Basically, increasing the thickness will eventually lead to the better performance and reduce the deflection. In vibration, however, either the structural stiffness or mass of structure that become a dominant parameter in influencing the natural frequency. It can be realized that the natural frequency of foamed concrete composite slab decreases due to the thickness, as displayed in Figure 13. It should be emphasized here that the natural frequency in this experimental study refers to the first mode shape of composite slab. According to Khan et al. (2013), increasing the thickness lead to the higher mass of structure that subsequently decrease the natural frequency. Composite slab CS-075 recorded the highest natural frequency at 34.98Hz and the change of value over the thickness is trivially insignificant.

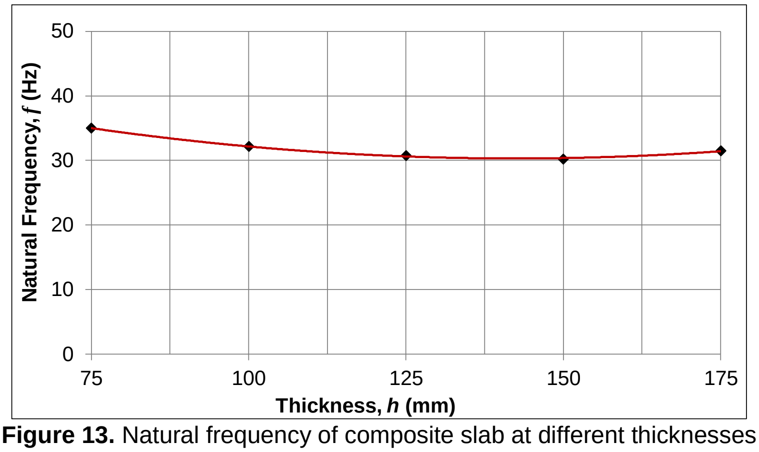


Figure 13. Natural frequency of composite slab at different thicknesses

However, the natural frequency is slightly increased for the composite slab CS-175 in which a similar behaviour was recorded by Kansinally & Tsavdaridis (2015). Therefore, the structural stiffness was affirmed as the foremost factor in governing the natural frequency while the mass of structure is limited within the certain array. A comparison of natural frequency using the empirical approach, as summarized in Table 2, revealed a contradict situation with experimental study. By using the empirical approach, the natural frequency was analysed in the elastic state with no crack effects taken into the account. Further verification using MEScope shows a very close set of results with a similar pattern of the natural frequency as obtained through experimental study. In the empirical approach and MEScope, the highest natural frequency was experienced by the composite slab CS-175.

Table 2. A comparison of natural frequency between experimental, empirical and MEScope

Composite Slab		Natural Frequency, *f* (Hz)		
Code	Thickness, h (mm)	Experimental	Empirical	MEScope
CS-075	75	34.98	24.91	32.60
CS-100	100	32.18	33.77	31.70
CS-125	125	30.77	36.54	30.20
CS-150	150	30.17	39.29	33.50
CS-175	175	31.50	41.98	34.10

Meanwhile, the relationship between the natural frequency with the thickness of composite slab can be translated as Equation 8:

$$f = 0.0011h^2 - 0.295h + 51.306 \quad (8)$$

However, this correlation is merely controlled by the mass of structure based on the volume of foamed concrete and not reflected by other specifications. In vibration responses, the serviceability must also be connected with the structural stiffness and span of composite slab. Moreover, foamed concrete is controlled by its density in favour of strength and hence must not be ignored. When the natural frequency is plotted against the ratio of span-to-thickness, the natural frequency can be presented by the best fitted of polynomial form:

$$f = \alpha_\rho \left[0.033\left(\frac{S}{h}\right)^2 - 0.842\left(\frac{S}{h}\right) + 36.145 \right] \quad (9)$$

where h is the thickness of composite slab, S is the span and α_ρ is the multiplying factor related to the density. Since Equation (9) was developed for foamed concrete with density 1800kg/m^3, hence the multiplying factor can be considered as 1.

Structural Damping

Basically, the structural damping of composite slab fall within the range of 2.0% to 3.0%. However, composite slab made of foamed concrete has the structural damping around 1.5% to 5.0% in which it can be considered quite high. Figure 14 shows the structural damping at different thicknesses of composite slab. It can be observed that the structural damping increases as the thickness become larger. However, such behaviour only pertinent to the composite slab with thickness not bigger than 125mm and beyond that the structural damping is gradually declined.

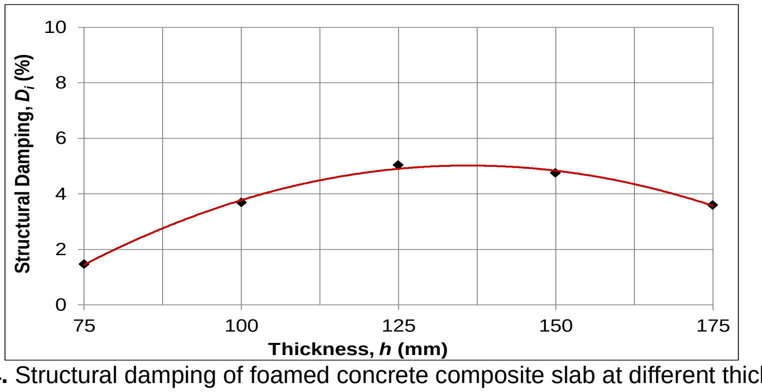


Figure 14. Structural damping of foamed concrete composite slab at different thicknesses

Although foamed concrete is categorized as lightweight material, but the structural damping reveals that any dynamic conditions will oscillate or ring the composite slab within the brief period only. This is an indication that the vibration amplitudes can decay faster and concurrently make the composite slab achieves a good comfortability. The structural damping was identified as a dynamic property of the composite slab relevant to the floor response and hence must not be overlooked. In certain situation where the natural frequency is acceptably high, the composite slab may experience much greater excitation due to the low structural damping that less than 1%.

The relationship between the structural damping and thickness of composite slab can be simplified to the following Equation 10:

$$D_i = 0.0011h^2 - 0.261h + 12.711 \quad (10)$$

Ta et al. (2006) suggested that the structural damping has a linear relationship with the natural frequency. Moreover, Varela & Battista (2011) clarified that the structural damping increases in parallel with the natural frequency, as an example for composite slab with the natural frequency of 7.57Hz and 12.21 Hz, the structural damping is 0.25% and 0.64% respectively. In this experimental study, the relationship between the structural damping and natural frequency is in linear inverse state as can be expressed by the following Equation 11:

$$D_i = -0.725f + 26.854 \quad (11)$$

However, there is a potential that the structural damping become proportional to the natural frequency for the composite slab with thickness bigger than 175mm.

Energy dissipation

Figure 15 shows the energy dissipation of composite slab at different thicknesses. It can be observed that the energy dissipation ranges between 0.10J to 0.55J and the value increase linearly as correspond to the thickness. However, the change in the energy dissipation due to the thickness of composite slab is relatively small, around 35%, except for the composite slab CS-125. When the thickness of composite slab getting bigger from 120mm to 125mm, the energy dissipation increases as high as 115% from 0.169J to 0.364J.

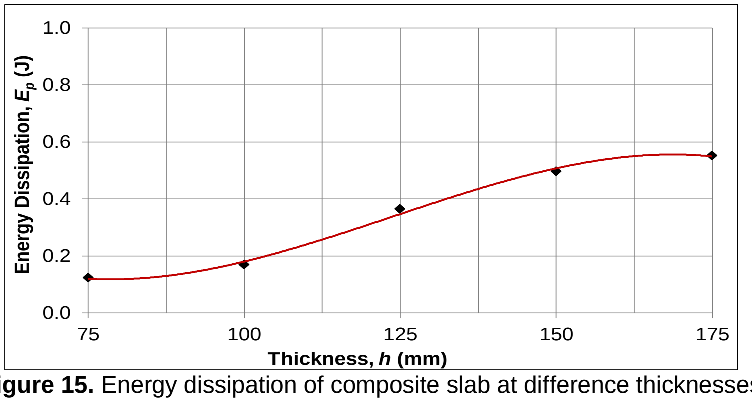


Figure 15. Energy dissipation of composite slab at difference thicknesses

According to Brownjohn (2001), the energy dissipation of lightweight precast slab is around 0.1J to 0.2J. Therefore, the energy dissipation obtained in this experimental study can be considered convincing. Since the energy dissipation was triggered from the first mode shape, it is the lowest value that can be activated by the composite slab. The relationship between the energy dissipation and thickness of composite slab can be depicted as Equation 12:

$$E_p = -0.5e^{-6}h^2 + 0.006h - 0.323 \quad (12)$$

In general, the energy dissipation of composite slab is mainly depended on the natural frequency, structural damping and kinetic energy. Foamed concrete itself is well recognized a good material in the energy absorbing. However, the structural damping was identified as the crucial factor that associated with the energy dissipation. The structural damping is the process through which the vibration of composite slab dismisses in amplitude due to the loss in mechanical energy. The relationship between the energy dissipation and structural damping can be alleged as a linear, though in actual it can be complicated. The simplified form of energy dissipation correspond to the structural damping is expressed as the following Equation 13:

$$E_p = 0.063 D_i + 0.05 \tag{13}$$

CONCLUSIONS

An experimental study on the vibration responses and energy dissipation of foamed concrete composite slabs was conducted using hammer-impact test. In addition, the compression and splitting-tensile tests were also performed to determine the compressive and tensile strengths of foamed concrete containing 40% RHA and 9kg/m^3 PMM. The compressive and tensile strength achieve 35.03MP and 2.01MPa respectively in which exceeded the expectation and requirement by the code of practise. The presence of high silica content in RHA and confinement effects from PMM that contribute to the satisfactory of compressive and tensile strengths. Therefore, foamed concrete can be confidently applied in composite slab. Meanwhile, the acceleration-time history measured from hammer-impact test was analysed to determine the natural frequency, structural damping and energy dissipation of composite slab. The results indicate that the natural frequency decreases throughout the thickness, but slightly increase when the thickness bigger than 150mm. The structural damping, however, shows a contradict behaviour in which the value increases for the thickness up to 125mm before gradually decay. On the other hand, the energy dissipation increases correspond to the thickness of composite slab. In this experimental study, it can be observed that the thickness plays a crucial role in governing the vibration responses and energy dissipation.

In conjunction of that, polynomial equations were proposed for the natural frequency, structural damping and energy dissipation. Besides the thickness that prompting the structural stiffness and mass of structure, the span of composite slab and density of foamed concrete are important parameters that must be taken into the consideration. Therefore, a suitable and simply equation was proposed to determine the natural frequency that depend on the ratio of span-to-thickness and multiplying factor of density. Moreover, linear equations were introduced in correlation between the structural damping ratio with natural frequency and the energy dissipation with structural damping. However, these equations are limited to the certain extend and require further verification.

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STATE OF THE ART ON PRE-TENSIONING STEEL STRAPS CONFINEMENT: LITERATURE REVIEW

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Abstract

This paper presents the state-of-the-art of the current research development on pre-tensioning steel straps confinement. Critical review was conducted based on careful selection of references to provide in-depth overview in the development of this confining technique. The paper was started with highlighting the features of such confining technique, then an extensive description of the connection clip systems was presented. The experimental and numerical investigations performed on steel-strapped concrete structures were discussed at length in the paper. The potential usage of pre-tensioning steel straps confinement in increasing the bond properties of confined concrete and as a repairing technique of damaged concrete were also covered in this paper.

Keywords: *Steel straps; Confinement; High-Strength Concrete; Fibre-reinforced polymer; pre-tensioned*

INTRODUCTION

The steel-strapping tensioning technique (SSTT) is a method by using recycled steel straps to confine concrete structures and secured the strap ends using end-clips (Ma et al., 2014a, 2014b; Moghaddam et al., 2010). With such confining pressure, the strength and ductility of concrete can be significantly enhanced (Ma et al., 2017a). High-strength concrete (HSC) which is brittle in nature can be ductile as a result of restraining action of HSC's lateral dilation through this confining technique. The properties especially stress-strain behavior of concrete can be essentially changed due to such restraining action (Awang, 2013).

The main aim of this review is to provide an up-to-date information of SSTT that covers different aspects such as the confining technique, the influence of concrete properties, design approaches and numerical modeling. This paper also provides general insight into the history and recent development of SSTT. In this review, the literature review process is done according to a systematic approach as illustrated in Figure 1.

For this confining technique to be efficient, three fundamental design criteria must be satisfied: (i) the pre-tensioning force exerted by steel straps should be adequate to ensure the concrete is efficiently confined without snapping of the steel straps during strapping process; (ii) the connection clip/ end clip must be strong and stiff enough to transfer the pre-tensioned force; and (iii) extra precaution must be taken to ensure no loss of pre-tensioning force during the securing of strap ends. The awareness of the behavior and design approaches of SSTT-confined concrete is also important for this type of construction to penetrate into the construction and building industry.

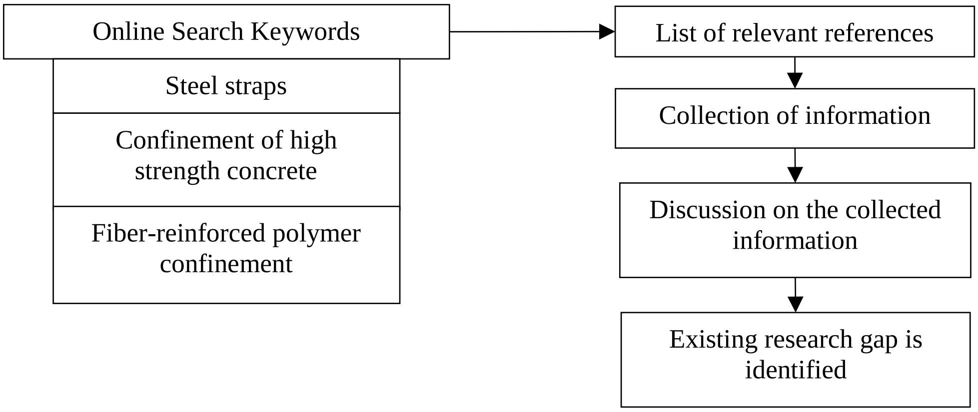

Figure 1. Process implemented for literature review

Before the introduction of SSTT confinement, concrete confining technique mainly used Fiber-reinforced polymer (FRP) as a confining material (Ma et al., 2015a; 2015b; 2015c). This includes FRP-wrapping of existing columns and concrete encased in FRP tubes for new column construction. However, as what has been demonstrated during the repair and rehabilitation works of Kuala Lumpur Middle Ring Road 2 (MRR2) in Malaysia, the construction cost involving FRP is extremely high. Subsequently, recycled steel straps, which are commonly used in packaging industry were proposed to replace FRP. Its primary application was in confining HSC. Its effectiveness, although is lower compared to the FRP confinement, it provides a low-cost alternative for confining technique.

The steel strap used for the confinement is as shown in Figure 2. It is made of low-cost recycled steel. Although various sizes of straps are available in the market, the most commonly used type in confining concrete is the one with nominal thickness and width of 0.5 mm and 15 mm, respectively. Figure 3 shows the typical stress-strain curves of steel straps and the associated mechanical properties are as listed in Table 1.

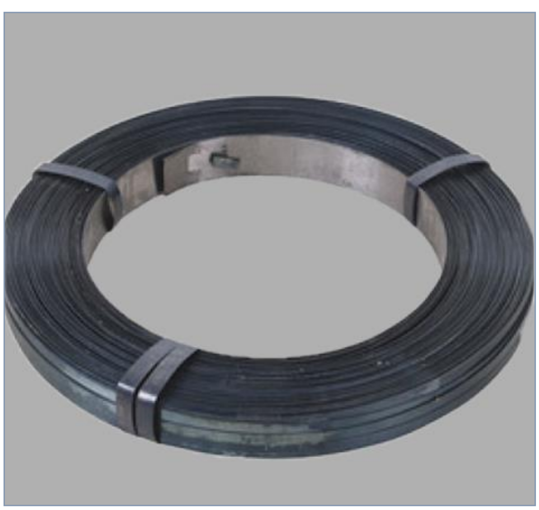

Figure 2. Low-cost recycled steel strap (Ma et al. 2017d)

CONNECTION CLIPS

In SSTT, clips are utilized to secure the ends of steel straps. Type of clips used will affect efficiency of the confinement. Type and operational condition of the sealing machine used also affect the clip efficiency. A manually operated sealing machine tool was used to secure the straps by punching notches into the clip as shown in Figure 4(a). The lifting of the sealing tool will result in the reduction of approximately 40 – 50 % of the initial pre-stressing force. This reduction reduces the effectiveness of confinement. In addition, notches punched into the clip reduced the effective dimension and ultimate strength of the strap (Garcia et al., 2014).

Table 1. Average mechanical properties of steel straps (Ma et al. 2014a)

Property	***Average value***
Thickness	0.512 mm
Width	15.06 mm
Yield strength, f_{sy}	1,003.3 MPa
Ultimate strength, f_u	1,079.4 MPa
Elastic modulus, E_s	202.7 GPa
Ultimate strain, ε_u	0.096 %

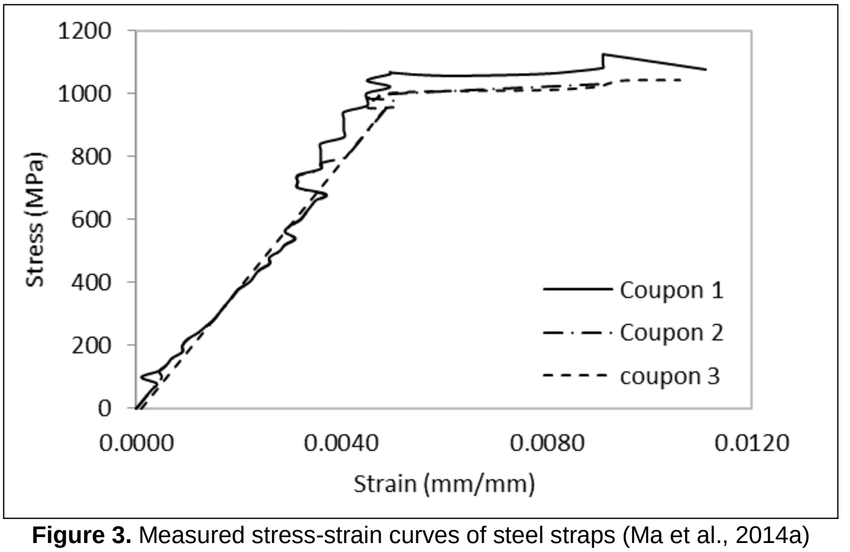

Figure 3. Measured stress-strain curves of steel straps (Ma et al., 2014a)

Recently, new clips and connection were introduced by Awang (2013), which were made by cutting a steel strap into small piece and bent at both ends (Figure 4(b)). The clips allow self-regulation between the strapping layers. Preliminary tests indicated that connections with a single clip were not sufficient and hence double clips as shown in Figure 3(c) were recommended. The clip made from thicker steel strap and placed in double was used to provide more bearing and twisting bending resistance to the confining strap. The width of the clip had a length equal to the width of the steel strap. The height of the clip was designed based on the thickness of the steel strap and on the number of layers being used (Figure 3(d)).

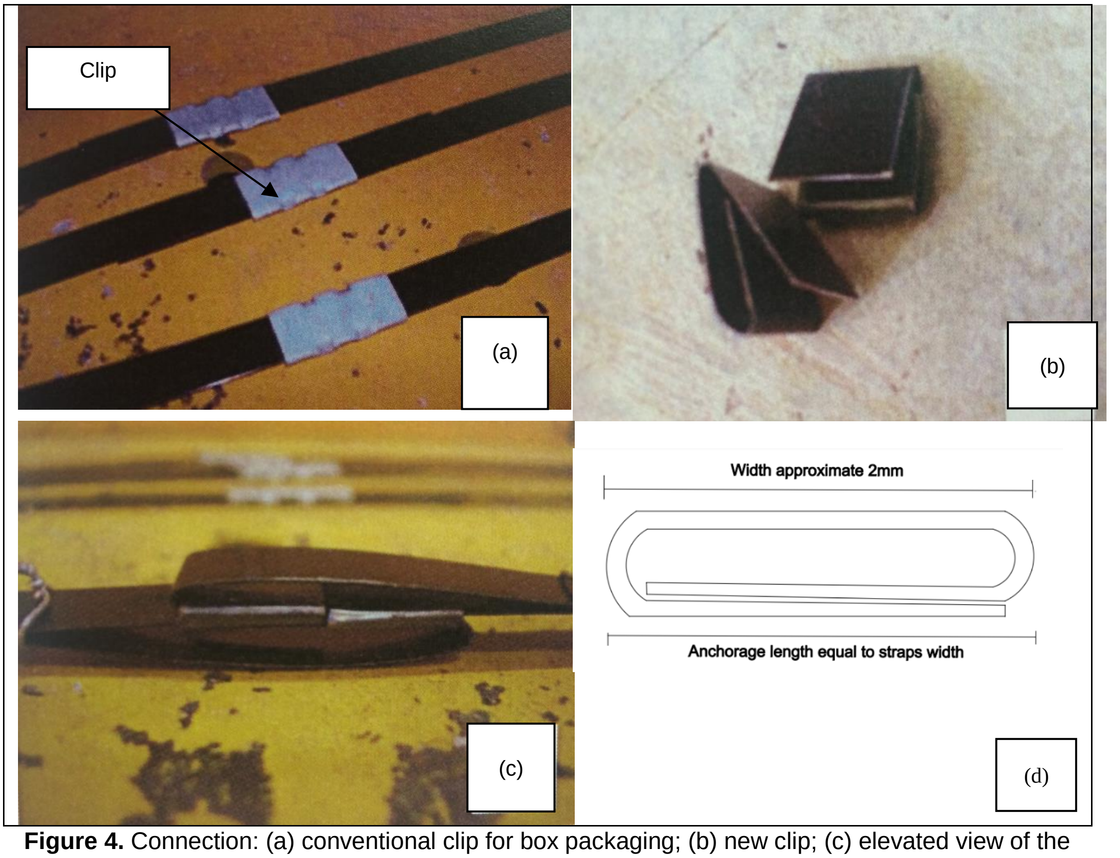

Figure 4. Connection: (a) conventional clip for box packaging; (b) new clip; (c) elevated view of the connection; (d) schematic diagram of the clip connection

AXIAL COMPRESSIVE BEHAVIOR

Monotonic behavior

SSTT as a confining technique was first attempted on concrete cylinders to confirm its effectiveness. Awang (2013) performed a series of experimental investigations of effectiveness using SSTT in confining HSC. The experimental work was performed by testing cylindrical specimens (100 mm × 200 mm). Concrete compressive strength were fixed at 50, 60 and 80 MPa. Parameters studied including different properties such as spacing of straps, number of strapping layers and different levels of pre-tensioning stresses. The confined specimens were tested in compression until failure under monotonic and cyclic loading conditions. His work confirmed the ability of the technique to improve the ductility and strength of the concrete, especially HSC, by effectively utilizing the confining material. It was found that the pre-tensioned steel straps confinement delayed the onset of volumetric expansion when the concrete is loaded and hence increase the ductility. Subsequently, a stress-strain model was proposed as Equation 1, 2, and 3:

$$f'_{cc} = f_{co} \cdot 2.62(\rho_v)^{0.4} \tag{1}$$

$$\varepsilon'_{cc} = \varepsilon_{co} \cdot 11.60(\rho_v) \tag{2}$$

$$\varepsilon'_{cu} = \varepsilon_{co} \cdot (8.9\rho_v + 0.51) \tag{3}$$

where f'_{cc} is confined concrete strength; ε'_{cc} is the strain at peak confined concrete strength; ε'_{cc} is the ultimate confined concrete strain; ρ_v is the volumetric confinement ratio of steel straps ($\rho_v=V_s f_y/V_c f_{co}$, where V_s and V_c are the volumes of straps and confined concrete, respectively, and f_y is the yield strength of the straps); ε_{co} is the ultimate strain of unconfined HSC (assumed equal to 0.004); and ε'_{cu} is the ultimate strain of confined HSC. The constitutive relationship defined by the above equations was calibrated using data from HSC cylinders confined with SSTT confinement ratios ranging from 0.076 to 1.50. Note also that the above equations assume that a minimum amount of straps (at least ρ_v=0.076) always exist around the element. The stress-strain relationship was proposed to refer to Popovics (1973), the concrete stress f_{ci} at a given strain ε_{ci} is defined by Equation 4:

$$f_{ci} = \frac{f'_{cc} x r}{r-1+x^r} \tag{4}$$

where $x=\varepsilon_{cc}/\varepsilon'_{cc}$; ε_{cc} is the axial compressive strain of concrete; ε'_{cc} and f'_{cc} are the strain and concrete strength of confined concrete, respectively; whereas f_{co} is the unconfined concrete compressive strength. In the above equation, $r=E_c/(E_c-E'_{sec})$, where E_c is the tangent modulus of elasticity of concrete and E'_{sec} is the secant modulus of elasticity of the confined concrete at peak stress.

The effects of lateral confining stresses were investigated by Lee et al. (2014) on 12 HSC cylinder specimens (100 mm × 200 mm) wrapped with two different confining materials of different mechanical properties and lateral pre-tensioning stresses. The pre-tensioning stresses ranging from 1.4 kN to 3.2 kN were applied on these specimens. Results showed that the specimens with higher confining level performed better as it delayed the strength losses at post-peak region, resulting in a slower but safer concrete failure. However, the optimum confining level to the confined HSC was not suggested in this study. It was suggested that when certain confining level has been exceeded, the performance of such confinement will either halt or reduce.

The effects of various parameters on strength and ductility of confined concrete were studied including compressive strength of concrete, mechanical volumetric ratio of confining straps, post-tensioning force in the strap, number of strapping layers wrapped around the specimens and details of strap joint were investigated by Moghaddam et al. (2010). Test results showed significant increase in the strength and ductility of specimens due to active confinement by steel straps. Ductility of confining material was reported to affect the confined concrete ductility. This improvement is greatly dependent to the effective mechanical volumetric ratio of confining straps. It was also observed that the SSTT results in stiffer confined concrete than the conventional passive confinement, as opposed to Ma et al (2014a, 2015a). It was observed that the strength and ductility of concrete are also dependent on the shape of specimen cross section. For a constant amount of confining straps, specimens with circular sections gained more strength and ductility enhancement than those with square sections due to stress concentration at the corners of square section. The radius of corners strongly influences the effectiveness of confinement. The application of higher post-tensioning force to the confining straps increases the concrete strength considerably and provides stiffer pre-peak behavior, but speeds up the post-peak degradation of concrete capacity. Applying more post-tensioning force to the confining straps will reduce the ductility or ultimate strain. The main reason of higher capacity in SSTT-confined concrete is that whilst the conventional confinement is initiated by the concrete dilation, the SSTT has perfectly confine the concrete even prior to the load application. Higher the post-tensioning forces in the straps will result in higher concrete strength. The poorer post-peak behavior of actively confined specimens is because some portions of the constant toughness of straps have been consumed in pre-stressing the straps, while the passively confined specimens use this capacity after the peak point.

Cyclic Behavior

Most recently, Lee et al. (2014) tested 21 HSC specimens (150 mm × 300 mm), confined with steel straps in different confining ratios, and tested to failure under uniaxial cyclic loadings (Figure 5). Results indicated that the technique is beneficial in strengthening concrete subjected to cyclic loading. Findings also shown that the stress deterioration ratio is independent of confining ratio and loading patterns. The cumulative effects due to repeated unloading/reloading cycles was observed can cause permanent strain and stress deterioration of confined HSC.

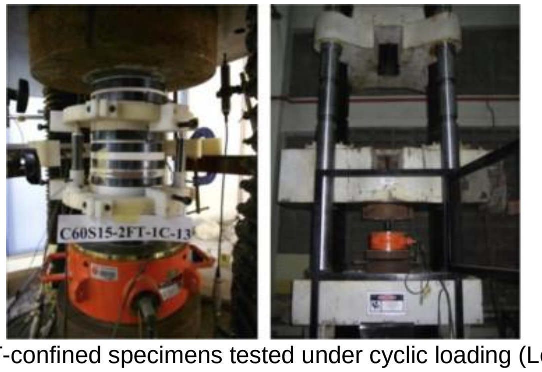

Figure 5. SSTT-confined specimens tested under cyclic loading (Lee et al. 2014)

TESTS TO FAILURE OF SSTT-CONFINED STRUCTURE ELEMENTS

Reinforced Concrete Columns

Previous experimental tests showed that SSTT can double the ductility of HSC than unconfined HSC. However, most of the tests performed on SSTT-confinement have focused on concentrically loaded short specimens. To cover this gap, experiments were carried out to investigate the slenderness effect of SSTT-confined HSC columns subjected to eccentric loads (Ma et al. 2014a; 2014b). It was confirmed that SSTT increases both the strength and deformability of slender HSC columns, although the confining effects are reduced proportionally with an increase in slenderness ratio.

For slender NSC columns, they was observed to have failed in a more ductile manner with an increase in slenderness ratio. However, HSC columns were observed to fail in a brittle and sudden manner but with an increased slenderness ratio and with the use of SSTT, they failed in a more ductile manner. The beneficial effects of SSTT confinement appear to be greater for a higher confinement ratio but is less effective in increasing the ultimate load capacity of slender HSC columns due to its ineffectiveness in improving flexural rigidity/stiffness.

Later, the same researchers (Ma et al., 2016) extended the experimental tests to rectangular shape rectangular columns. A total of nine square HSC specimens (six confined and three unconfined columns) were investigated. It was observed that SSTT is also applicable to rectangular shape column as significant increase in maximum load was obtained when the columns were confined. However, the increment in maximum load of HSC columns was found to decrease with the increase in confinement level, showing no proportional increment in maximum load with confinement level. A similar observation has been reported by Jiang and Teng (2012), Ma et al. (2014a) and Johansson (2002), indicating that there is a threshold strength improvement due to external confinement. In columns with a large eccentricity, which means a large bending moment, the presence of steel straps produced higher ductility with the increase in confinement level.

From these, it can be concluded that higher confinement ratio of SSTT can increases both the strength and ductility of reinforced concrete columns. Higher slenderness ratio of SSTT-confined HSC columns will result in lower strength improvement by the confinement. The strength improvement also decreases with the increase of confinement level.

Reinforced Concrete Beams

The applicability of SSTT confinement in strengthening flexural member such as beam was further performed by Ma et al. (2016b). Twelve over-reinforced HSC beams (f_{co} = 50 or 80 MPa) were designed to fail prematurely by concrete crushing at mid-span as shown in Figure 6. The mid-span region of eight such beams was confined externally using the SSTT with different steel strap confinement ratios, the aim of which was to delay concrete crushing. Results showed that although the unconfined beams failed in a brittle manner with no post-peak deflection, the steel straps were very effective at enhancing the post-peak deformation of the SSTT-confined beams by up to 126 %. Moreover, for the beams tested in this study, the use of the SSTT led to failures after the yielding of the tensile reinforcement.

Figure 6. Final view of a typical HSC beam after strapping using the SSTT (Ma et al. 2016a)

DESIGN APPROACHES

SSTT confinement was found resulted in 'short' member to behave like 'slender' member and more susceptible to instability (Ma et al. 2014c; Jiang and Teng, 2012). This instability is particularly significant in HSC member as it inherent the brittle nature of the material (Galano and Vignoli, 2008). Hence, an analytical study was carried out by Ma et al. (2014d) in the view of developing a simple equation in order to determine the slenderness limit for SSTT-confined columns. Based on the analysis, slenderness limit for SSTT-confined HSC columns, λ_{lim} were proposed as Equation 5:

$$\lambda_{lim} = \frac{50\frac{e_s}{D}\left(1-\frac{e_i}{e_s}\right)+24\left(1.2-\frac{e_s}{D}\right)^4}{6\rho_v-5.7\rho_v{}^2+0.6} \tag{5}$$

where e_i and e_s is the first and second eccentricity, respectively.

In designing the SSTT-confined columns, safe and economic are two important aspects. The designed member should not be too expensive but guarantee minimum safety. Generally, the effectiveness of confinement became less pronounced as the slenderness of column increased. This is mainly due to the flexure-dominated behaviour of slender column and the non-contribution of the confinement to the flexural stiffness of the concrete. Hence, a numerical analysis was conducted by Ma et al. (2014d) to establish the upper-bound slenderness limit of SSTT-confined HSC columns. The utilisation of SSTT beyond this limit is not recommended. The slender SSTT-confined HSC column is considered to reach its upper-bound slenderness limit when the ductility enhancement is less than 5%. Results from the numerical modelling on SSTT-confined HSC columns with slenderness ratios ranging from 10 to 50 showed that the slenderness ratio increased from 10 to 50, the ductility of HSC columns dropped rapidly from about 380% of equivalent unconfined column to < 5%. When the slenderness of column reached 48, even higher confinement ratio has insignificant effect.

Once the so-called lower-bound and upper-bound slenderness limits of SSTT-confined columns were established, a systematic design procedure based on nominal curvature approach was proposed by Ma et al. (2016c). The proposed design approach is based on results from segmental analyses of slender SSTT-confined circular columns subjected to eccentric loads. The results obtained from the analyses are used to determine the variables governing the design of such columns. The use of the proposed design parameters predicts conservatively the capacity of small-scale slender HSC circular columns confined using the SSTT, and can be thus used in the practical design of reinforced concrete (RC) structures. Existing design codes for RC members use an 'equivalent stress block' with uniform compressive stresses to represent the compressive stress profile of concrete at ultimate condition. Such equivalent stress block is usually defined by the magnitude of stresses and by the depth of the stress block. To maintain force balance, the resulting equivalent stress block and the original stress profile have to resist the same axial force and bending moment. Due to the steel strap confinement, the equivalent stress block proposed by codes is inappropriate to assess the ultimate capacity of SSTT-confined HSC columns. Therefore, a parametric study is carried out to develop an equivalent stress block of SSTT-confined HSC sections. The equivalent stress block is defined by: 1) A mean stress factor (α1), defined as the ratio of the uniform stress over the stress block to the compressive strength of SSTT-confined HSC, and 2) A block depth factor, β_1, defined as the ratio of the depth of the stress block to that of the neutral axis. A constant value $\beta_1 = 0.90$ was suggested for practical design whilst the mean stress factor α_1 (Equation 6) is proposed for design:

$$\alpha_1 = 0.195\rho_v + 0.85 \tag{6}$$

The ultimate flexural strength, N_u, and moment, M_u are proposed as Equation 7 and 8 :

$$N_u = \alpha_1\beta_1 f'_{cc}A + \sigma_{sc}A_{sc} - \sigma_{st}A_{st} \tag{7}$$

$$M_u = \alpha_1\beta_1 f'_{cc}A\left(\frac{D}{2}-\frac{\beta_1 x_n}{2}\right) + (\sigma_{sc}A_{sc} - \sigma_{st}A_{st})\left(\frac{D}{2}-d\right) \tag{8}$$

NUMERICAL MODELING

Alongside with the experimental development in SSTT-confined structures, A theoretical model was developed by Ma et al. (2014b) to analyse slender SSTT-confined HSC columns. The developed model simulates the second-order distribution experienced by slender SSTT-confined HSC column by axial load-moment-curvature path. Similar numerical procedures were adopted previously by other researchers (Park and Paulay, 1975; Kong and Evan, 1987; Jiang and Teng, 2012). Ma et al. (2014b) modified the method by incorporating the new stress-strain model for SSTT-confined concrete. The proposed numerical procedure has reported to yield close agreement with experimental results from Ma et al. (2014a) and Lei (2012).

OTHER POTENTIAL APPLICATIONS OF SSTT

Another potential application of SSTT confinement is to repair damaged concrete columns. In a series of tests, Ma et al. (2017a; 2017b; 2017c) confirmed that the SSTT is a reliable method in restoring the capacity loss in damaged concrete cylinders. The restorability can be as high as twice the residual capacity of damaged concrete columns, provided the confining volumetric ratio is high enough. However, the effectiveness is highly dependent on the damage level of column. Besides, SSTT-confinement was attempted by Sulaiman et al. (2017) in increasing the bond properties of confined concrete. A series of pull-out tests were conducted and confirm that the confinement can increase at least 50% of bond strength compared to unconfined counterparts.

CONCLUSION

In this paper, the state-of-the art on the research of SSTT confinement has been presented. Several aspects, such as the features of the confining system, the types of end clips developed the experimental and numerical investigation performed on SSTT-confined concrete structures have been discussed. From the review, it can be concluded that SSTT has good potential to be used to strengthen concrete structure, at the same time, increase the ductility of HSC, which however is greatly hindered by the lack of awareness among potential industrial users. Although SSTT has been investigated in-depth and adequate knowledge has been acquired, several issues such as durability, long-term pre-stressing loss and capacity loss due to exposure to the fire, are yet to be discovered. In addition, the behaviour of such confined structures under cyclic loading is considered limited and the design procedure for seismically loaded confined structures is not yet introduced. These research data will be needed for the implementation of SSTT in practical.

ACKNOWLEDGMENT

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FINITE ELEMENT PROCEDURES DEVELOPMENT FOR NONLINEAR STEEL FRAME ANALYSIS WITH QUASI-STATIC SEMI-RIGID CONNECTION

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Abstract
As the semi-rigid behaviour becomes essential in current design consideration, this paper presents a study on the development of a nonlinear numerical analysis that is capable of simulating the quasi-static semi-rigid behaviour of structural steel frames under variable loadings for reliable design. The beam element shape functions with semi-rigid connections are formulated and the overall stiffness matrix is derived. The theory to solve the nonlinear analysis is also presented. The developed procedures also revealed the complex finite element calculations in order to identify the structural performance of nonlinearity for steel structures. The major modifications that include the revised solution procedure, the variable load pattern capability, the quasi-static connection model, the modification of input data to cater for initial imperfections and the inelastic constitutive model for steel are presented. Validation process has been performed for the developed finite element procedures with loading and unloading behaviour for semi-rigid connections and full-scale frame behaviour. From the results, the modified procedures are capable to predict the behaviour of the structural steel frames under varying loading conditions with various connections configuration and material properties.

Keywords: *Nonlinear, Inelastic, Finite element analysis, Semi-rigid, Quasi-static, Steel frame*

INTRODUCTION

Performance based design for steel frames has been studied for nonlinear plasticity (Banihashemi et al., 2015). Stepping into a performance based design, consideration of the nonlinearities and rigidity of connection became important to maintain reliable structural behaviour. Finite element analysis is capable to solve the complex problems in masonry structures (Lourenço et al., 2007), stainless steel (Ashraf et al., 2006), composite structures (Spacone and El-Tawil, 2004), and connections (Díaz et al., 2011). The finite element computer programs are based on a series of successively more complex enhancements of a program originally developed by Jones (Jones, 1980). The original version could only investigate the behaviour of an isolated 2-dimensional beam-column restrained by semi-rigid connections, which were modelled using cubic B-Splines and attached to infinitely stiff restraints.

The program was modified by Rifai (1987) to include the effect of beams framing into the column. The beam-column connections were considered semi-rigid whilst on the other end of the beams were taken as rigid in nature. SERIFA was then developed by Ahmed (1992) modifying Rifai’s program to handle the elastic and inelastic analysis of planar steel frames with semi-rigid connections. The full load-deflection response up to failure is traceable.

Traceably, there are well-developed commercialised computer aided engineering packages like ABAQUS (Dassault Systèmes, 2014), ANSYS (ANSYS Inc., 2013), NATRAN (MSC Nastran, 2014) and etc. which have their own benefits in application of structural analysis. Research investigations with nonlinearities of geometry (Lee et al. 2015a; 2015b), material and boundary condition, as well as connection flexibility behaviour (Lee et al. 2015c; 2017) have been actively conducted for steel structures, especially cold-formed structures. With suggested solutions by these packages, engineers may not fully understand on the methods used. The daily routine works for engineers to analyse structural performance that may need the knowledge on finite element method in order to obtain reliable models in the design stage.

As stepping into a new era of engineering design, performance-based design is a reliable method to be applied in the structural design. Therefore, this paper present a study on the development of finite element analysis for nonlinear steel frames with semi-rigid connections which capable to be a reliable design procedures. The procedures of finite element method have been revealed in order to gain better understanding on the structural analysis. The nonlinearities (material, geometric and boundary) and inelasticity have been included in the analysis for more accurate prediction on structural performance until failure loads. The other features also include variable load pattern capability, the quasi-static connection model, semi-rigid behaviour, the modification of input data to cater for initial imperfections and the inelastic constitutive model for steel. Validation of the developed finite element code is performed with loading and unloading behaviour for semi-rigid connections and full-scale frame behaviour.

FINITE ELEMENT PROCEDURES DEVELOPMENT

A summary is presented which explains the physical mechanics of the model used in the procedures. The beam-column elements are represented by two-noded one-dimensional line elements with three degrees of freedom at each node. This physical beam length is separated into finite elements connected at nodal points.

The displacements at the nodal points are the unknown parameters and are obtained using the equation $\{\Delta\delta\}=[K]^{-1}\{F\}$, where $[K]$ is the stiffness matrix as in Equation 1 and $\{F\}$ is the applied load. Cubic-polynomial interpolation functions are used to define the displacement within the nodal points. As the displacement can be defined at any point on the cross-section within the beam-column element, the state of strain can also be obtained by using the strain equation as in Equation 2. Hence, by applying the constitutive properties of steel, the state of stress along and across the sectional areas of the member can be calculated.

$$[K] = [K_E] + [K_L] + [K_G] \tag{1}$$

$$\varepsilon_N = \left\{ \frac{du_o}{dx} + \frac{1}{2}\left(\frac{d\,v_o}{d\,x}\right)^2 \right\} - y\left(\frac{d^2\,v_o}{d\,x^2}\right) = u_o' + \frac{1}{2}\left(v_o'\right)^2 - y\left(v_o''\right) \tag{2}$$

An approximate method to determine the sectional properties (flexural rigidities, *EI* and axial rigidities, *EA*) and to monitor the spread of yield was suggested by Nethercot (1974). These values are required in determining the stiffness matrix. The stresses at any point in the section can be determined if the strain at that point is known. Therefore the internal force, $\{F_i\}$, i.e. the axial load and moment, $\{M\}$, at each node about an axis passing through the mid-point of the section may be found by integrating over the across the section.

However, due to non-linearity in the geometric, material and boundary conditions, the equilibrium condition is not satisfied at the nodes. Hence, out-of-balance forces, ΔF_i result due to the differences between the external, $\{F\}$ and the internal forces, $\{F_i\}$. Therefore,

$$\{\Delta F_i\} = \{F\} - \{F_\mathrm{i}\} \tag{3}$$

To overcome this problem, an approximate Newton-Raphson's approach discussed later is employed. The numerical solution is obtained if it converges, i.e. if the out-of-balance force, ΔF_i becomes very small. The criterion used to check this convergence is given by Equation 4,

$$\left(\frac{\sqrt{\sum_{1}^{n} \{\Delta F_i\}^2}}{\sqrt{\sum_{1}^{n} \{F\}^2}} \right) \leq \text{Tolerance limit} \tag{4}$$

The second criterion of the numerical solution consists of a check on displacement convergence and is given by Equation 5,

$$\left(\frac{\sqrt{\sum_{1}^{n} \{\Delta \delta_i\}^2}}{\sqrt{\sum_{1}^{n} \left(\{\delta_{i-1}\}^2 + \{\Delta \delta_i\}^2 \right)}} \right) \leq \text{Tolerance limit} \tag{5}$$

where $\Delta\delta$ is the norm vector of the incremental displacements, n is the number of elements in the vector which corresponds to the number of degrees-of-freedom.

The process continues in load steps until the critical load level is reached; this is when the structure becomes unstable and this condition is identified when the diagonal terms of the global stiffness matrix are negative. To ensure that a 'refined failure load' is identified, an automatic step-back procedure is implemented to refine the incremental increase of the applied load.

Modification in Analysis

Stage 1: Initial Stage of The Development and Modification of The Data Input To Cater For Initial Imperfections

In view of the above investigations, a series of procedure developments have been carried out. The development can be categorised into several stages. Modifications of the input data are made by adding text and comment statements. In addition to this, a standard routine to cater for the uniformly distributed load applied along the member is included. This routine is similar to the routine that has been incorporated earlier into the program to deal with point loads along the member.

In order to obtain a satisfactory result at the inelastic region, the computer time increases unnecessarily because a large number of elements are required. It has to be noted that, even with element reduction modifications, a lot of effort is required to modify all the existing routines. With these modifications, it is also capable to cater for the irregular steel-frames and the initial out-of-straightness (geometry) by inputting the data manually. Thus, the problem of initial geometric imperfections faced earlier is solved. The common value for the maximum initial lateral deflection of columns is up to one thousandth of the column's length. A half sine wave is usually assumed for the initial deflections. By referring to Figure 1, the initial deflection δ_o at any point along the column is given by $\delta_o = a_o \sin \pi \frac{x}{L}$, where a_o is the initial central deflection and L is the length of the column.

Stage 2: Development of the Non-Linear Solution Procedure

Figure 2 illustrated schematically the non-linear solution procedure of load-displacement relationships for an arbitrary structure subjected to an applied load $\{F\}$. The incremental displacement vector $\{\Delta\delta_i\}$ is calculated using the standard equation $\{\Delta\delta_i\} = [K_i]^{-1} \{\Delta F_i\}$, where $[K_i]$ is the stiffness matrix of the structure and $\{\Delta F_i\}$ is the out-of-balance force. The numerical solution is obtained when it converges to point A, when the out-of-balance force, ΔF_i becomes acceptably small. The criterion used to check this convergence is given by Equation 4 and Eq 5.

The presence of the out-of-balance force vector, $\{\Delta F_i\}$, it requires the internal forces,$\{F_i\}$ due to the corresponding displacement, Δd_i to be evaluated. A technique that taking the algebraic sum of the shear forces just to the left and just to the right of the applied load is adopted to find the out-of-balance forces due to a concentrated load.

Summing over all the Gauss points gives the internal force vector,$\{F_i\}$ and hence the out-of–balance force, $\{\Delta F_i\}$, can be calculated using Equatin 3. Once this is obtained, convergence checks discussed above are carried out. The same procedure is adopted for the next load step until structural failure occurs.

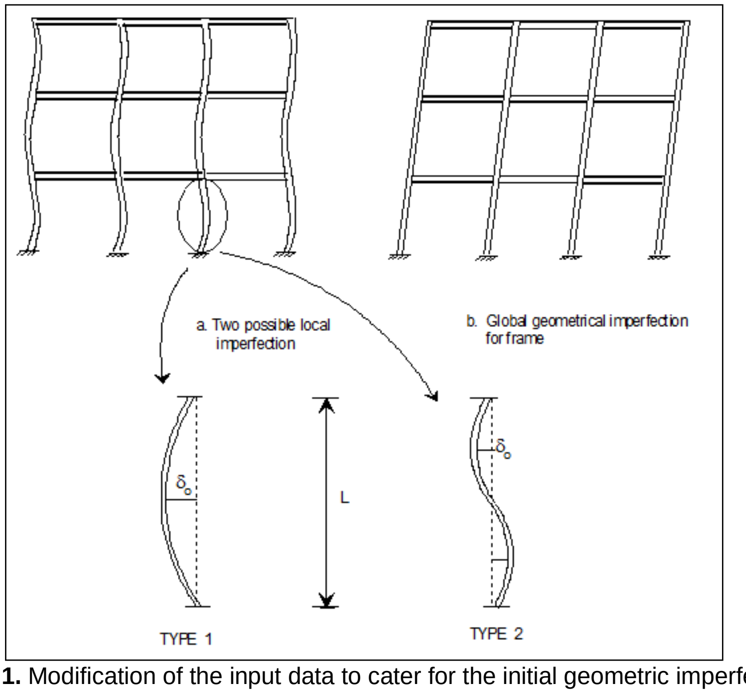

Figure 1. Modification of the input data to cater for the initial geometric imperfection

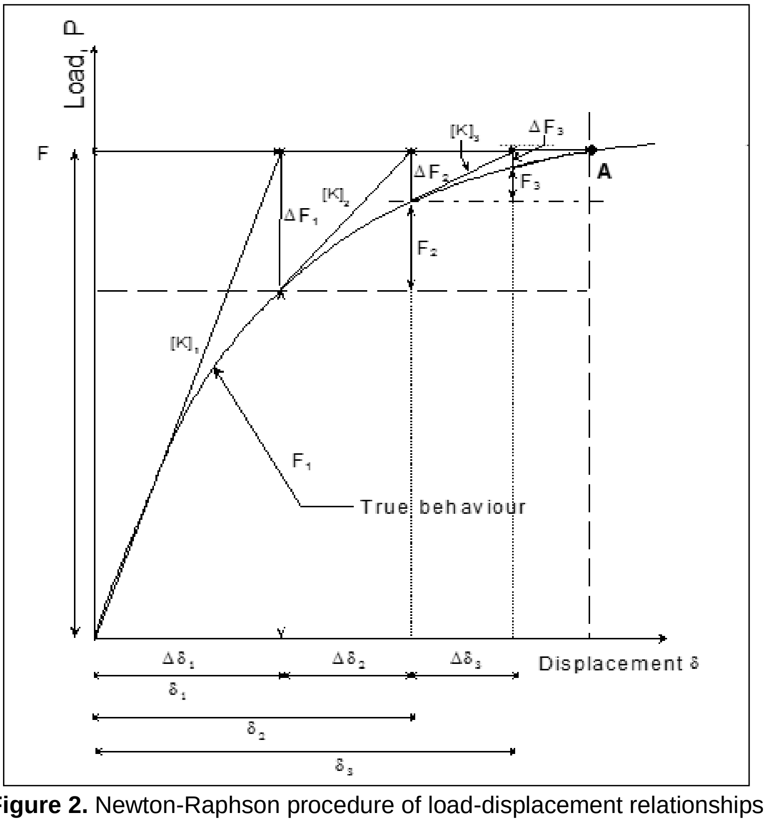

Figure 2. Newton-Raphson procedure of load-displacement relationships

Stage 3: Inclusion of the Variable Load Pattern

The inclusion of the variable loading pattern is of prime importance. Each of these patterns defines the history of the corresponding load i.e whether the load is increasing, decreasing or remains constant. The routine also defines the rate at which the load is increased or decreased. A reference load value, P_n for each load is however required in the input data. The loading process is divided into a number of load stages, (NSTAGE). For each load stage, each load is divided into N_n load increments. At any load stage, any load may be increased (+ve sign), decreased (-ve sign) or maintained as it is. At each load stage, the equilibrium conditions of the non-linear behaviour must be satisfied.

Stage 4: Development of the Connection Model

Over recent years, numerous test data for connections between beams and columns due to monotonic loading for different type of connections have been made available. At present a commonly used approach to describe this connection behaviour is to curve-fit the experimental data with a simple expression. Some of the commonly used models are polynomials, B-Spline models, power models and exponential models. However, all these were developed for monotonic conditions.

For the case of slow cyclic or quasi-static loading, only relatively few test data on the connection behaviour are available. Connections generally exhibit a nonlinear loading and unloading pattern. Full scale frame and assemblage experiments conducted using the semi-rigid connections have shown that whilst connections tend to follow the idealised moment-rotation curve with increasing axial load, the moment at the column head frequently reduces. This phenomenon is known as moment shedding. Unloading of the connections can also occur in the case of sway frames due to the application of lateral loads.

Modelling and incorporating these non-linear loading and unloading connections behaviour into the frame analysis program is difficult. It is also difficult to use curve-fitting techniques to present the constitutive relation. Unloading is assumed to take place elastically with the initial stiffness k_1. This simplification is shown schematically in Figure 3.

The characteristic starts from zero moment with the elastic stiffness k_1 and continues to load at that stiffness up to a moment of M_y. If it starts unloading at any point below M_y, it would unload elastically. If the load is again reversed, the connection will move up the same path with stiffness k_1 until it reaches M_y. If the loading continues, a stiffness of k_2 is assumed, until the moment reaches a value of M_{pl}. If unloading takes place before this point, elastic unloading with stiffness k_1 is assumed until the moments drops by twice the value of M_y. If it goes below this, a stiffness of k_2 is assigned. A stiffness value of k_3 is assumed if unloading continues past the negative value of M_{pl} but if loading commences before this point, a stiffness k_1 is assigned.

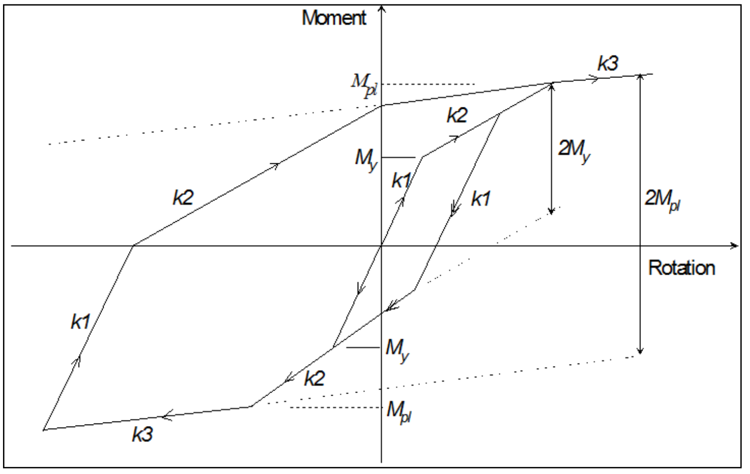

Figure 3. Analytical connection model response to a general load history

Stage 5: Development of The Non-Linear Inelastic Material Properties of Steel

One of the crucial factors that need to be considered in non-linear inelastic analysis of steel frames is the modelling of the material properties. Unlike the geometric non-linearity where it has been incorporated through the formulation of the model described in (Mohammad et al., 2017), material non-linearity enters this formulation via the constitutive model. These properties are usually obtained from a tensile test, in which the steel sample is pulled in a tensile testing machine until it fails. Even if the test procedure is simple, it reveals useful information on the material strength, stiffness and ductility required for the present analysis. BS EN ISO 6892-1 (BSI, 2009) provides the guidance on the tensile testing of structural steel samples.

Analysis Flowchart

An analysis procedure is developed to study the behaviour of frames under variable loading in ambient temperature using finite element method. The main steps of the procedure followed in the analysis are shown in Figure 4.

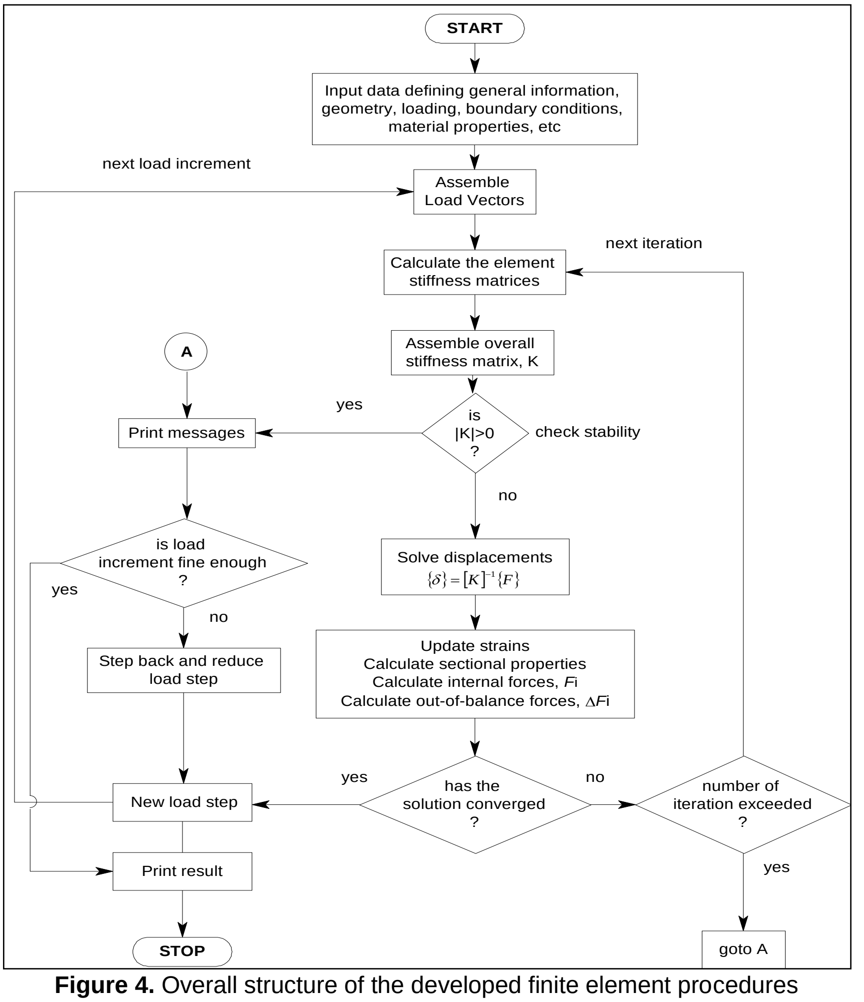

Figure 4. Overall structure of the developed finite element procedures

VALIDATION OF PROPOSED PROCEDURES

It was necessary to verify the validity of the developed procedures with experimental or other analytical results. This is a process to assess whether or not it adequately represents the real behaviour of structures.

Loading and Unloading Behaviour for Semi-Rigid Connections

A two bay and a single storey frame with a slender column as shown in Figure 5, is chosen as a validation example. The 203 × 203 × 86 UC steel columns with an initial geometric imperfection of L/1000 at mid-height were subjected to two types of loading. During the analysis, the beams are loaded up to the design load and kept constant, while the column axial load increased up to failure. In the first analysis, the unloading and the loading connection elastic behaviour is considered to follow the same path. Then a similar analysis is carried out but incorporating the slow cyclic loading-unloading connection behaviour. A tri-linear representation of the moment-rotation connection behaviour is adopted as is shown in Figure 6 so that the manual calculation could be done as a check. A Southwell plot is constructed to estimate the elastic critical load.

The load-deflection relationship at the mid-height of the centre column and their corresponding Southwell plots are shown in Figure 7 and 8 respectively. For the former analysis (non-linear elastic connection behaviour), the elastic critical load is the gradient of the line that is 2208 kN. For the latter analysis (incorporating true loading-unloading behaviour), the elastic critical load is calculated as 2638 kN. For this particular case, an increase of up to 18% is obtained if the loading-unloading connection model is adopted. Figure 9 shows the loading-unloading connection behaviour at the top of the centre column. It could be clearly seen that once the column load is applied, the connection on the right unloaded while the left connection continued to load throughout the loading process. As the right connection underwent relatively smaller rotations, their stiffnesses were generally greater because it corresponded to the path of the initial connection stiffness. On the other hand, if the normal loading connection model is adopted, on the application of the axial load, the right connection unloaded but the stiffness traces back along the loading path of the connection. It is also clear that the tangent stiffness of the loading connection will be significantly less than the stiffness of the unloading connection, which caused the 18% increase in the elastic critical load.

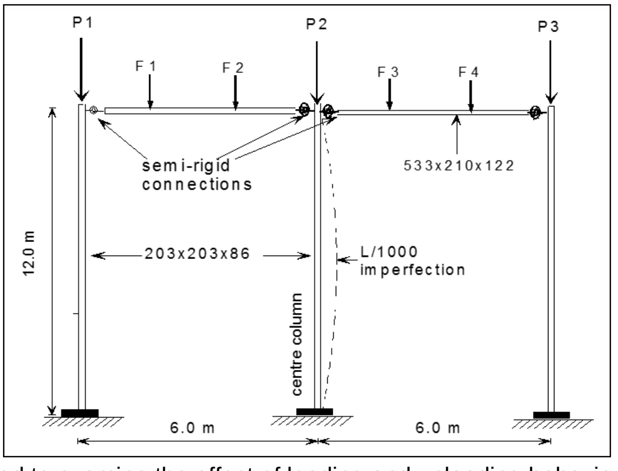

Figure 5. Frame used to examine the effect of loading and unloading behaviour of the connection

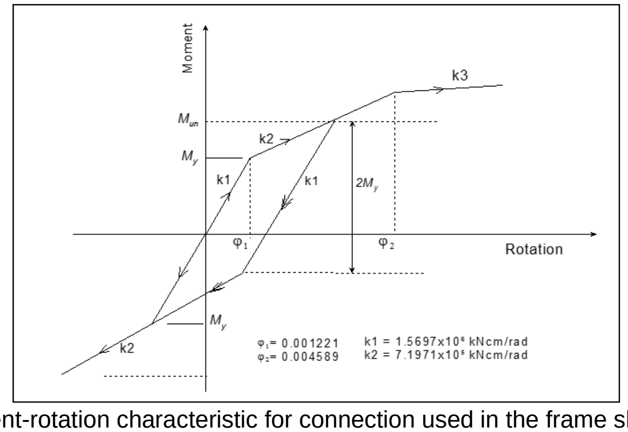

Figure 6. Moment-rotation characteristic for connection used in the frame shown in Figure 6

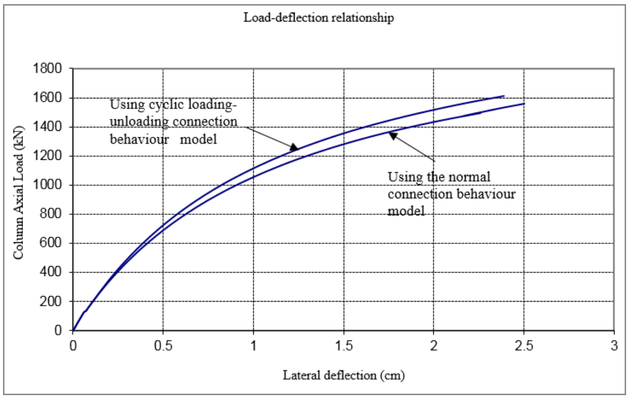

Figure 7. Load-deflection relationship at mid-height of the centre column

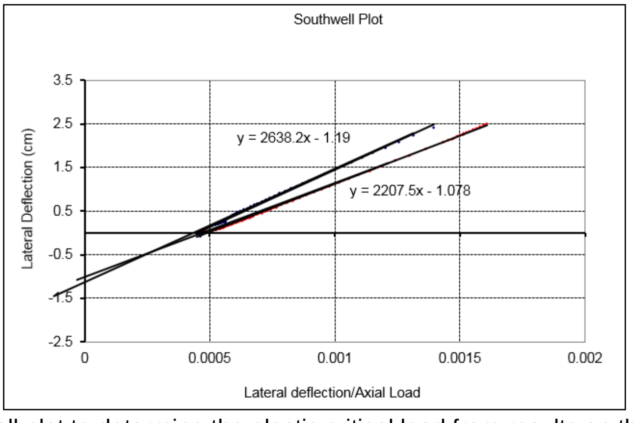

Figure 8. Southwell plot to determine the elastic critical load from results on the frame shown in Figure 6.

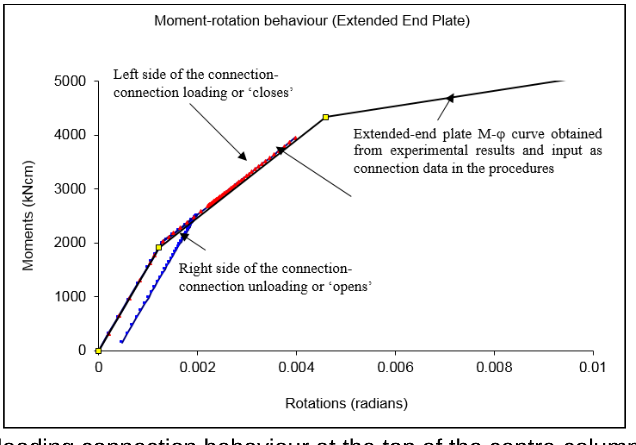

Figure 9. Loading-unloading connection behaviour at the top of the centre column of Figure 19 and 17

In order to validate the results, the same example is analysed using the standard formula for the elastic critical load of a perfect column with one end pinned and the other end restrained. The elastic critical load is given by Equation 8,

$$\tan\left[L\sqrt{\frac{P}{EI}}\right]=L\sqrt{\frac{P}{EI}}\left\{\frac{K}{PL+K}\right\} \tag{8}$$

where, P is the elastic critical load, E is the Young's modulus, I is the second moment of area, L is the column length, and K is the spring stiffness.

Similarly, by solving this equation, the elastic critical load for the latter type of connection model is 2670 kN. The differences between the manual calculations and the Southwell plots for using the cyclic loading-unloading model and the normal model is 2.4% and 1.1% respectively. From these results, it can be seen that the manual calculation and the Sothwell plots compare very well. Furthermore, the behaviour of the connection (as illustrated in Figure 9) is in agreement with the response reported by Gibbons (1990) in his study on the strength of biaxially loaded columns in flexibly connected steel frames.

Comparison with Full-scale Sub-frame Tests – BRE TEST

The frame test was conducted by British Research Establishment (BRE) and used for validation. The frame is represented as in Figure 10.

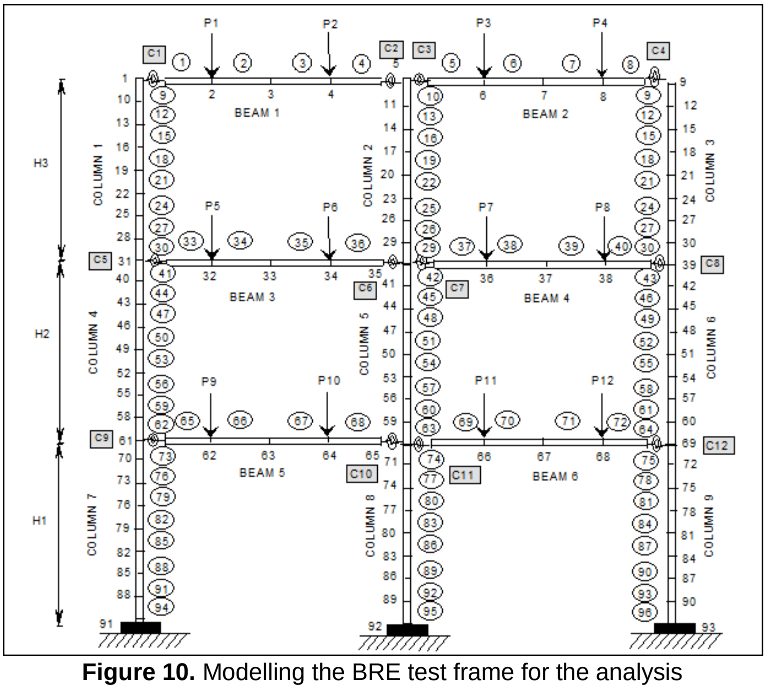

Figure 10. Modelling the BRE test frame for the analysis

Figures 11 and 12 show the load deflection relationships obtained from the test and the analysis respectively. The curves plotted are with the third point load values against the central deflection of the beam. The bending moment distributions on Beam 4 and Beam 6 obtained from developed procedures are compared with the bending moments obtained from the test results. These are shown in Figures 13 and 14. It can be seen that the predicted responses are in good agreement with the experimental results. Thus, it can be seen that the developed procedure is capable of predicting the beam behaviour up to collapse load.

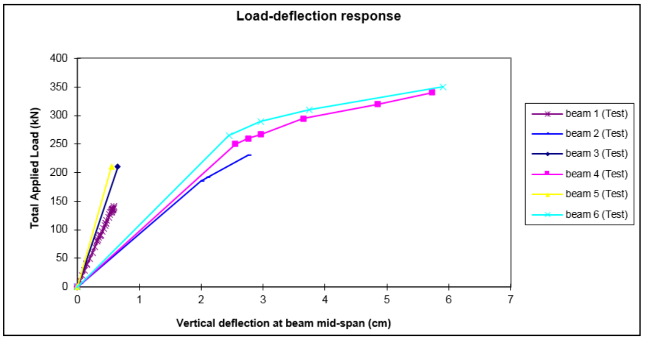

Figure 11. Load-Deflection relationship at the mid-span beam obtained from test results

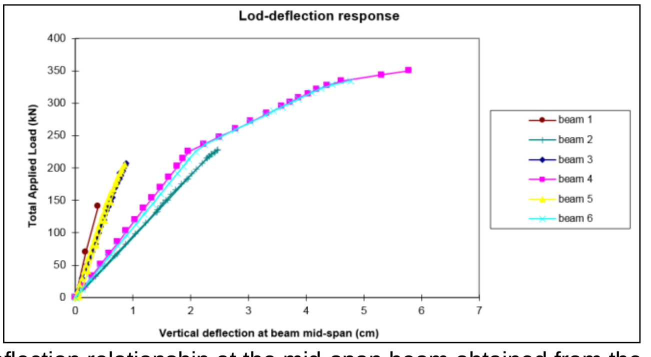

Figure 12. Load-Deflection relationship at the mid-span beam obtained from the developed analysis procedures

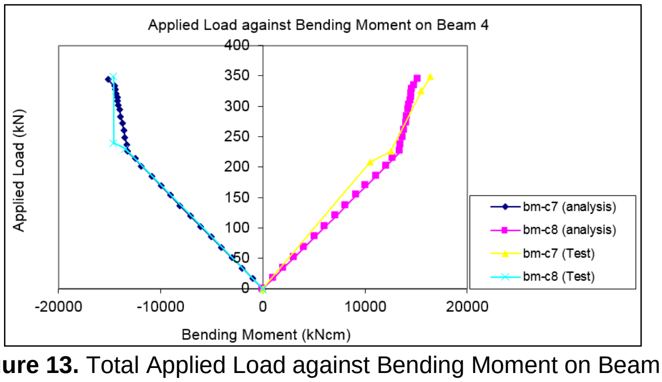

Figure 13. Total Applied Load against Bending Moment on Beam 4

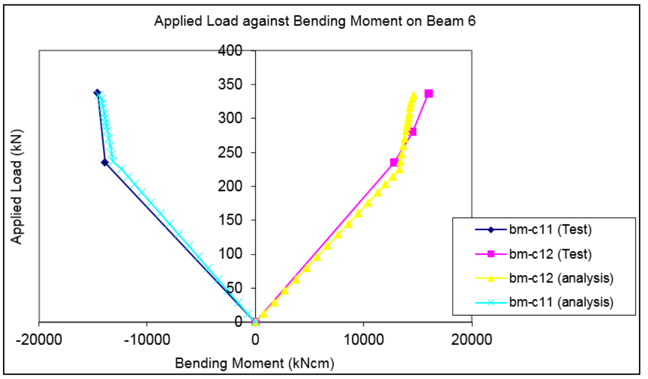

Figure 14. Total Applied Load against Bending Moment on Beam 6

Comparison with Full-Scale Frame Tests -SUF TEST

SUF1 and SUF2 are two-dimensional major and minor axes frames respectively, using section 254 × 102 UB22 beams and columns with 152 × 152 UC3 in sizes. The objectives of these tests were to investigate the overall frame behaviour and, specifically to concentrate on the capacity of the columns under non-sway conditions and to compare the behaviour of columns buckling about their major and minor axes. Figures 15 and 16 show the general arrangement of the frames, SUF1 and SUF2 respectively.

Some of the responses of the beams under the beam loading for the SUF1 tests are given in Figures 17 and 18. Figure 17 (a-c) shows the load-deflection relationships obtained from the analysis and the test plotted with the quarter point load values against the central deflections of beams 2, 3 and 4. Table 1 compares the column test load obtained from strain gauges and the load obtained from the developed procedures. Similarly, the responses of the beams under the beam loading for the SUF2 tests are given in Figures 18(a) to 19(f). Table 2 compares the column test loads with the results from the developed finite element procedures.

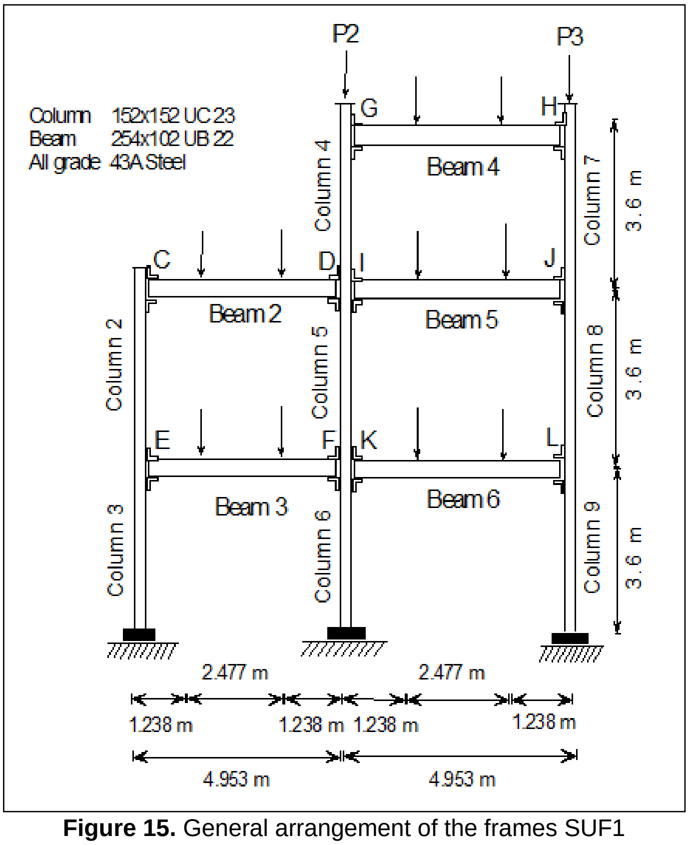

Figure 15. General arrangement of the frames SUF1

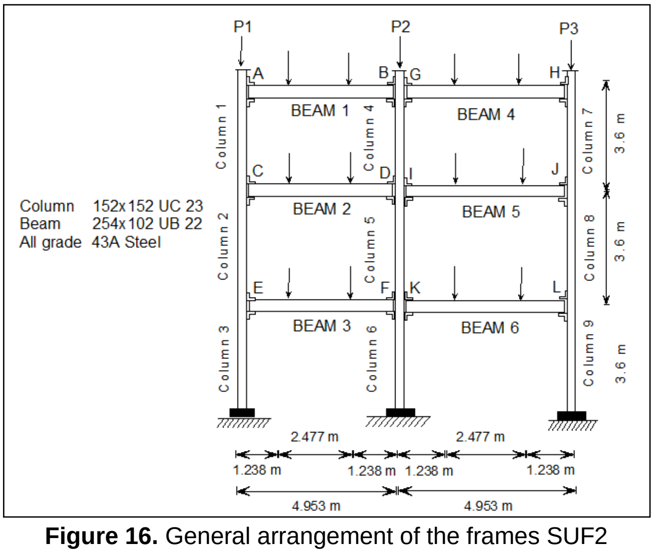

Figure 16. General arrangement of the frames SUF2

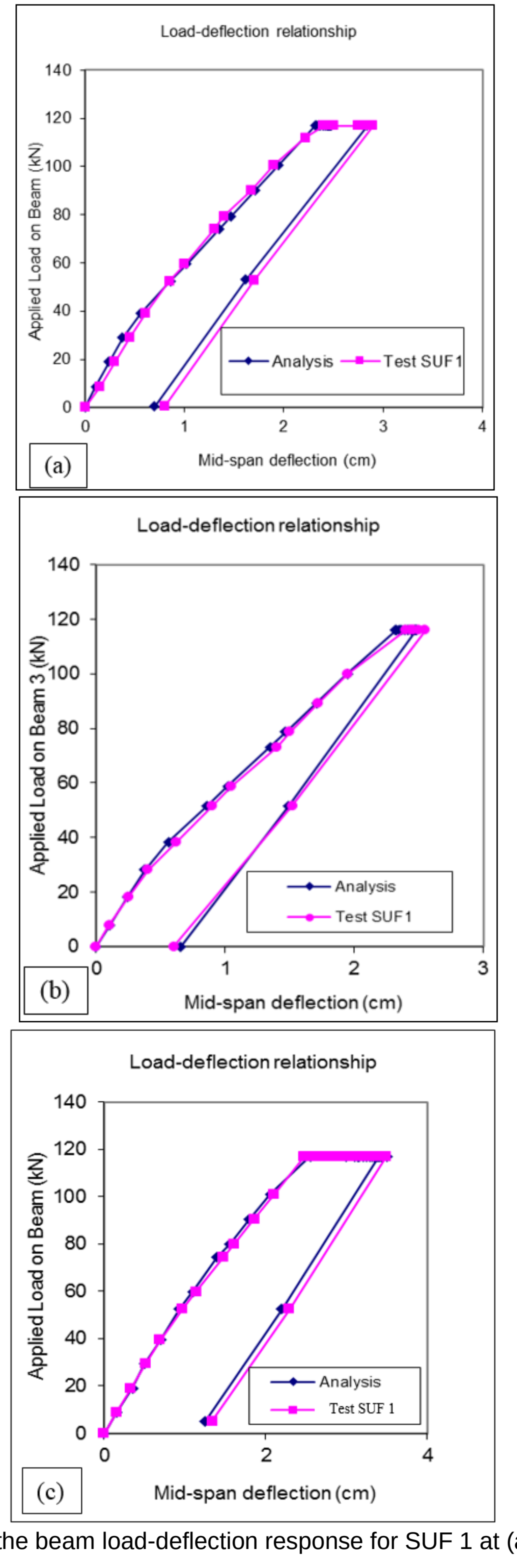

Figure 17. Comparison of the beam load-deflection response for SUF 1 at (a) Beam 2 (b) Beam 3 and (c) Beam 4

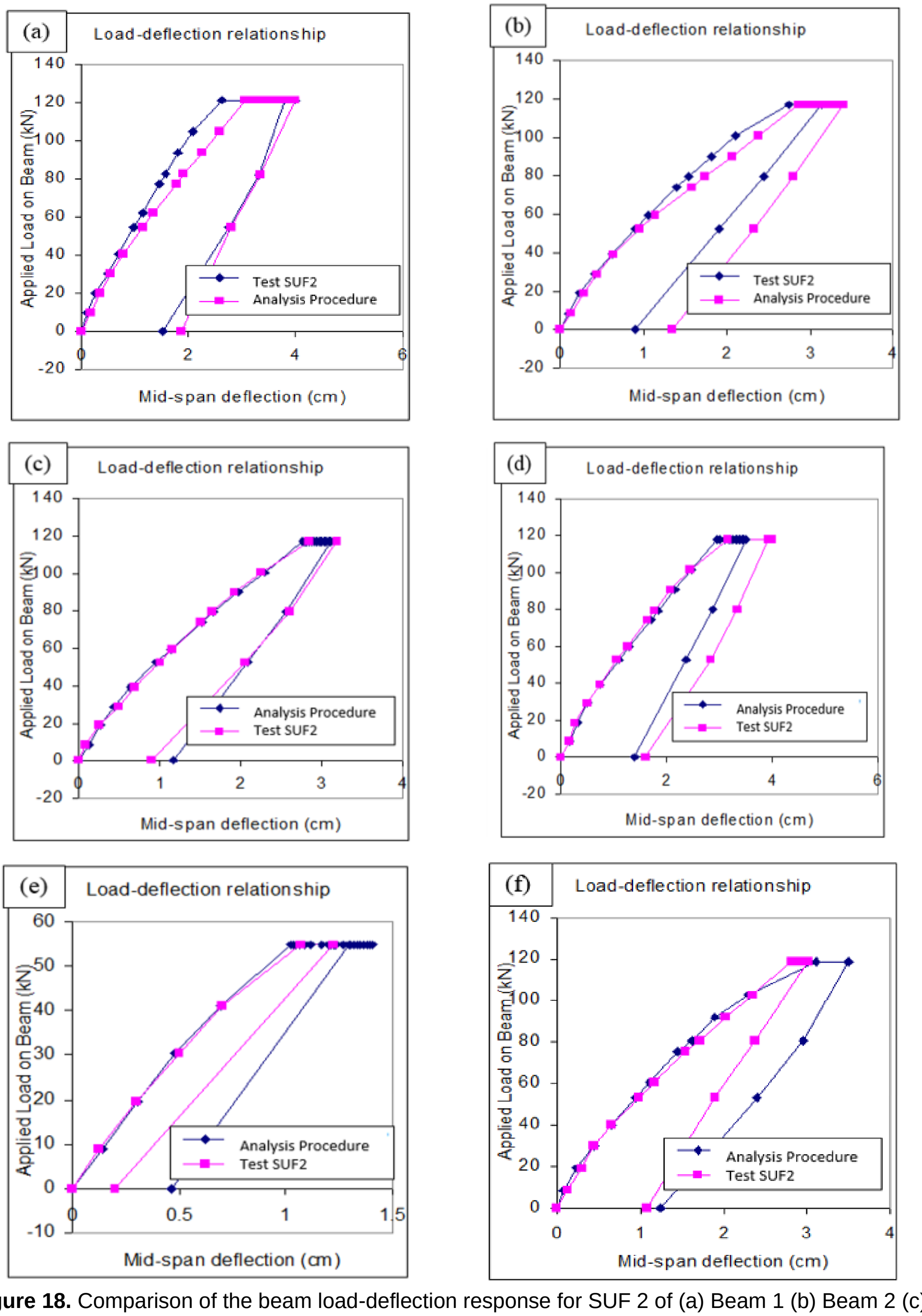

Figure 18. Comparison of the beam load-deflection response for SUF 2 of (a) Beam 1 (b) Beam 2 (c) Beam 3 (d) Beam 4 (e) Beam 5 (f) Beam 6

Table 1. Comparison of test and analysis loads for columns at failure in test SUF1

Column	Column Axial Loads at failure (kN)		% Difference
	Test Results	Analysis	
4	535.2	539.8	0.8%
5	639.8	624.3	2.4%
6	783.8	741.6	5.3%
7	728.2	680.8	6.5%
8	764.8	707.7	7.5%
9	834.7	766.5	8.0%

Table 2. Comparison of test and analysis loads for columns at failure in test SUF2

Column	Column Axial Loads at failure (kN)		% Difference
	Test Results	Analysis	
1	501.6	508.1	1.2%
2	546.1	566.8	3.6%
3	626.5	607.0	3.1%
4	521.1	533.7	2.4%
5	615.8	619.6	0.6%
6	813.4	738.8	9.2%

It can be seen that the predicted responses are in good agreement with the experimental results. The results also show that the developed procedure is also capable of predicting the frame behaviour under loading and unloading process. As shown in the figures, the differences in the permanent deflections after being unloaded is marginally small, indicating the capability of the developed procedures to simulate the inelastic behaviour of the frame. Both tests also demonstrate the capability of the developed procedures of predicting the column axial load behaviour especially up to 90% of the column failure load.

CONCLUSION

This paper presents an overall picture of the work carried out to develop the nonlinear finite element analysis with quasi-static semi-rigid connection model. Moreover, it is also have highlighted several experimental and analytical tests available as a mean to validate the proposed finite element procedures.

The results of the comparisons have been examined and it can be concluded that the present analysis procedures are able to predict the behaviour of the frames under varying loading conditions, connections, materials properties and mode of buckling. Thus, it is suitable for structural analysis for steel frames with semi-rigid connections.

ACKNOWLEDGEMENT

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EFFECTIVENESS OF REPAIR METHOD USING HYBRID FIBER REINFORCED POLYMER FABRIC ON CONCRETE-FILLED DOUBLE SKIN STEEL TUBULAR COLUMNS EXPOSED TO FIRE

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Abstract
Concrete-filled double skin steel tubular (CFDST) columns were often used in outdoor construction where fire is not a main concern. Therefore, this series of research deals with behaviour of CFDST columns after fire exposure, residual strength and method of repairing fire-damaged columns. This particular paper focused on the effectiveness of Hybrid Fiber Reinforced Polymer (FRP) repairing method. Total of 42 specimens were casted and 36 of the specimens were exposed to ASTM E-119 until the temperature of 600˚C. After that, the temperature was kept constant for 60 and 90 minutes. Out of 36 specimens that were exposed to fire, 24 of the specimens were repaired with FRP using hand lay-up method. All of the specimens (control, heated unrepaired and heated repaired) were subjected to concentric axial loading. It was found that by using Hybrid FRP, the ultimate strength at failure of repaired specimens greatly increased when compared to fire-damaged specimens to the extent of exceeding the control specimens. In addition, FRP is also effectively confined thinner outer steel tube than thicker outer steel tube. However, the effectiveness of Hybrid FRP repair method depends on several factor such as thickness of outer steel tube and maximum exposure time.

Keywords: *Hybrid fiber reinforced polymer, Concrete-filled double skin steel tubular columns, Hybrid repair, Carbon fiber reinforced polymer, ASTM E-119 time-temperature curve*

INTRODUCTION

Concrete-filled double skin steel tubular (CFDST) columns were first introduced by Shakir-Khalil (1991) in 1990’. After that, CFDST columns were widely used as described in (Han et al., 2014). However, CFDST columns were mainly used in outdoor construction, such as bridge pier (Zhao and Grzebieta, 2002) and an electrical pole in China (Han et al. 2014). Engineers are reluctant to use CFDST columns in an indoor construction due to limited research on performance of CFDST columns during fire. Unlike CFDST columns, the performance of concrete-filled steel tubular (CFST) columns during fire were already established and in depth review on performance of CFST columns during fire can be found in (Jayaprakash et al. 2012).

Another main concern when dealing with fire is residual strength of CFDST columns. Residual strength of damaged columns needs to be determined in order to predict the approximate strength gained after retrofitting the damaged columns. For structural steel, the main concerns were deformation and distortions of the steel structural member. As for concrete, it’s mechanical properties depends on several factors such as maximum temperature of the concrete during fire and fire exposure time (Han et al. 2005). Both steel and concrete acted differently in CFDST columns after fire. Thus, these need to be studied first before fire-damaged CFDST columns can be repaired effectively with Fiber Reinforced Polymer (FRP).

To author knowledge, limited research can be found on repairing of CFDST columns with FRP. However, research on repairing CFST columns using Carbon FRP have been conducted by Tao and Han (2007); Tao et al. (2007; 2008). All the aforementioned research used either one layer or multiple layer of CFRP. According to Hu et al. (2011), the effect of galvanic corrosion occurred when steel and carbon were in contact for a long period of time. Due to this reason, Glass FRP (GFRP) was used as first layer in Hybrid FRP repairing scheme. In addition, GFRP possess high ultimate tensile strain that proved to be beneficial during failure as reported in Shahidan et al. (2016).

As a part of this research study, a total of 102 specimens were casted, heated and repaired with single layer and Hybrid layer of FRP. Details of this research studies can be found in Mohd Zuki et al. (2017); Shahidan et al. (2016); Zuki et al. (2015a; 2015b). This paper will focused on the effectiveness of using Hybrid FRP as a repair method on fire-damaged CFDST columns.

MATERIALS PREPARATION

Total of 42 specimens were casted in an upright position. Prior to filling in the concrete, steel tubes were wire brushed and air blown in order to cleanse the tubes from dust and debris. The concrete was filled in between two steel tube by layers and were vibrated using poker vibrator. All the specimens were left to cure at Concrete Laboratory at room temperature until the day of heating and testing. Six concrete cubes of 100 × 100 × 100 mm were casted and cured in similar condition as CFDST specimens. The cubes were tested after 7 and 28 days. The compressive strength of concrete after 7 and 28 days is shown in Table 4. The preparation and testing of compressive strength of concrete cube is carried out in accordance with BS EN 12390 Part 1 (British Standards Institution BSI, 2012), BS EN 12390 Part 2 (British Standards Institution BSI, 2009) and BS EN 12390 Part 3 (British Standard, 2009). Three steel coupons were cut from steel tubes and tested for tensile strength and tensile strain at failure. The test was conducted according to ASTM E8/E8M-11 (ASTM, 2011). The result is shown in Table 5.

Table 4. Compressive strength of concrete

Batches	7 days	28 days
C1	38	41
C2	34	40
C3	34	41
C4	40	44
C5	41	43
C6	40	41
C7	29	35

Table 5. Parameter and material properties of steel

Diameter (mm)	Thickness (mm)	Ultimate Tensile Strength (N/mm^2)	Tensile Strain (%)
101.6	2	566	16.1
152.4	3	409	14.8
152.4	4	430	14.8

The properties of FRP were obtained through series of tensile test on FRP coupons. The nominal thickness of FRP is 0.17 mm. These coupons were cut out from the same FRP sheets that were used to wrap the specimens. The coupons were cut into desired length, wetted with epoxy and rolled to make sure there are no air bubbles in between layers, similar to the condition when it was used to wrap specimens. These so called witness panels are practiced in the field particularly during repair or rehabilitation program of structures using FRP. Witness panels for Hybrid FRP, GFRP and CFRP is shown in Figure 8. Witness panels were used as indicator of quality when hand lay-up method was used. The coupons were cured for a minimum of seven days before testing. Balaguru et al. (2009) suggested that witness panel or/and repaired structures needed to be cured for at least 3 days before it can be fully loaded. The data obtained from these tests are average tensile strength and average tensile strain at failure of six FRP coupons. The result of FRP tensile test is summarized in Table 6. The test is carried out in accordance with ASTM D3039/D3039M – 08 (ASTM, 2014).

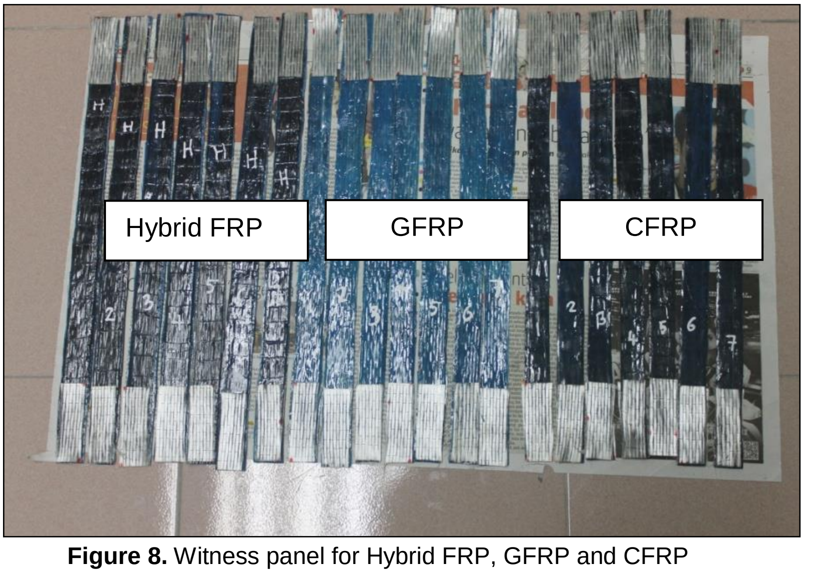

Figure 8. Witness panel for Hybrid FRP, GFRP and CFRP

Table 6. Result of tensile test for CFRP, GFRP and Hybrid FRP

	Carbon Fiber Reinforced Polymer (CFRP)	**Glass Fiber Reinforced Polymer (GFRP)**	**Hybrid Fiber Reinforced Polymer (Hybrid FRP)**
Tensile Stress [MPa]	2924	2499	4269
Tensile Strain [%]	1.41	2.06	1.53

The specimens were divided into two categories based on the thickness of outer steel tube, t_o as shown in Table 7. Apart from t_o, exposure time and condition of heated specimens were varied. Exposure time were varied into 60 and 90 minutes based on previous research on CFST columns where maximum exposure time were 180 minutes. Since CFDST columns possess void in the middle of the columns, the maximum exposure time were reduced to 60 and 90 minutes. As for the repairing scheme, two repairing schemes were adopted which is one layer of CFRP and Hybrid layer of FRP. Hybrid FRP is the combination of two FRP, GFRP as the first layer and CFRP as second layer. Diameter of outer steel tube (152.4 mm), diameter of inner steel tube (101.6 mm), thickness of inner steel tube (2 mm) and length (600 mm) of the specimens were kept constant.

Table 7. Test specimens

Diameter of outer tube [mm]	Thickness of outer tube [mm]	Diameter of inner tube [mm]	Thickness of inner tube [mm]	Exposure time [min]	Condition	No. of specimens
152.4	4	101.6	2	0	Control	3
				60	Un-repair	3
					Repair CFRP	3
					Repair Hybrid	3
				90	Un-Repair	3
					Repair CFRP	3
					Repair Hybrid	3
	3			0	Control	3
				60	Un-repair	3
					Repair CFRP	3
					Repair Hybrid	3
				90	Un-Repair	3
					Repair CFRP	3
					Repair Hybrid	3
Total						42

FIRE TEST

All of the specimens, except six specimens that were labels control, were heated in accordance with ASTM E119 – 11 (ASTM, 2010). The specimens were heated in fire furnace available at Universiti Sains Malaysia (USM) Concrete Laboratory. The time-temperature curve is shown in Figure 9. ASTM E-119 time-temperature curve was chosen over ISO-834 time-temperature curve due to ASTM-E119 curve gave more severe result to the exposed specimens for the first 1.5 hours of exposure compared to ISO-834: Fire Resistance Test Elements of Building Construction (Harmathy et al., 1987). After 1.5 hour, both curves gave similar severity to each other thus, it can be neglected.

The specimens were heated following ASTM E-119 curve until the temperature reached 600˚C. Afterwards, the temperature was kept constant until the end of the test where the furnace will automatically shut down when reaching pre-program exposure time i.e., 60 minutes and 90 minutes. According to ASTM E-119, the critical temperature for structural steel is 538˚C. If the structural steel was exposed to this temperature for less than 20 minutes, the strength of the structural steel is unaffected. Hence, 600˚C was chosen in this research study. During casting process, thermocouple was inserted into the concrete up to 300 mm from the top of the specimens. The thermocouple used was Type-K Thermocouple with 1000 mm length and 3 mm in diameter. At the end of thermocouple, there was a 1000 mm cable which is attached to data logger for the temperature recording during the fire test.

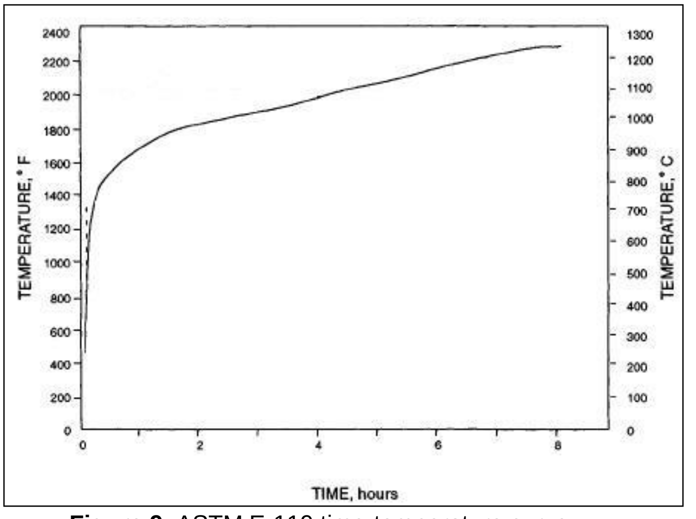

Figure 9. ASTM E-119 time-temperature curve

The specimen was put upright during heating process as shown in Figure 10. In addition, ceramic wool was stuffed inside the specimen void in order to minimize the heat transfer by means of convection and radiation. This is also to portray the real situation where the CFDST column was protected against heat transfer through connection to other structural members, i.e., beams. Only one specimen was heated per session. Four ceramic thermocouples were attached to the wall of the furnace to measure furnace temperature during heating process. The furnace was connected to control panel where the data was recorded. Using thermocouples type K, the temperatures of inner steel tubes as well as in the concrete core was recorded for the entire fire exposure duration. After the targeted exposure time was achieved, the furnace was shut down. Then the furnace lid was opened and the specimens were left to cool down inside the furnace naturally.

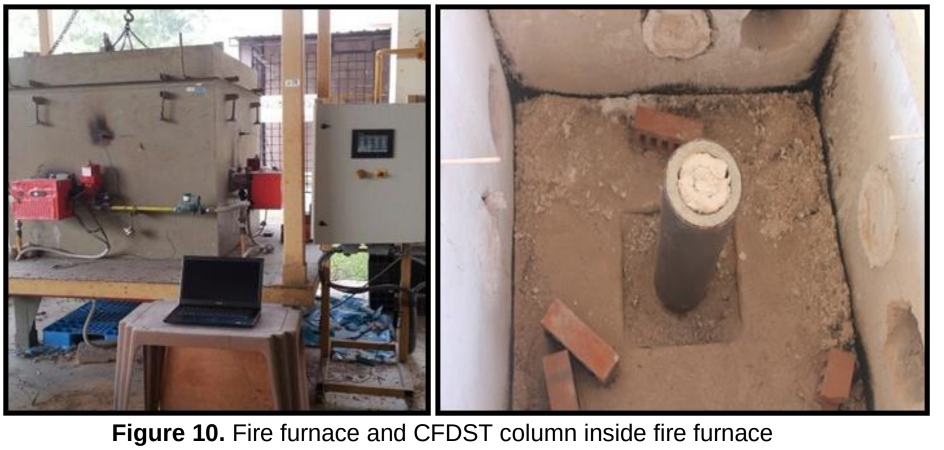

Figure 10. Fire furnace and CFDST column inside fire furnace

REPAIR SCHEME

The specimens were left to cool down to ambient temperature before repairing process started. There were two types of repairing scheme in this research study. The first one was by using only one type of FRP which was CFRP. This type of repair was addressed as one layer of CFRP repairing scheme. Second repairing scheme was called Hybrid FRP repairing scheme. Hybrid FRP consisted of two layer of FRP and used of different type of FRP i.e., GFRP as the first layer and CFRP as a second layer. GFRP and CFRP was cut out to it desired length with additional 100 mm. This extra length is to allow for the overlapping in the wrapping process. The FRP were wrapped using hand lay-up method. For Hybrid FRP, the second layers were done while the epoxy was still wet and the lapping between layers were carefully chosen to make sure it was perpendicular to each other. This was done in order to make sure the stress was evenly distributed throughout the column. Prior to the wrapping process, the columns were wire brushed and air blown to make sure the contact surface is clear from any dust and debris. The repaired columns are shown in Figure 11. The epoxy used in this wrapping process is two part MBrace Saturant. Its materials properties as provided by manufactures is shown in Table 8. The epoxy was hardened after 24 hour of curing. However, as suggested by (Balaguru et al., 2009), the curing process is at least 72 hour before any structures or members can be fully loaded. In this research study, the specimens were cured for a minimum of 7 days before testing.

Figure 11. CFDST columns after repair

Table 8. Material properties of epoxy (as provided by manufacturer - MBrace)

	MBrace Primer	MBrace Saturant
Compressive strength [MPa]	73	50
Tensile strength [MPa]	35	39
Flexural strength [MPa]	55	62
Tensile modulus [MPa]	2097	2400
Compressive modulus [MPa]	2320	2006

RESULT AND DISCUSSION

All specimens, (control, heated unrepair and heated repair) were subjected to monotonic axial compression test. The test was conducted at Construction Research Institute of Malaysia (CREAM), Kuala Lumpur, Malaysia. The loading rate used was 1 mm/minute. The load was applied until the load dropped to about 80 percent of maximum load. Four 10 mm electrical strain gauges were used to measure longitudinal and lateral displacement of the specimens. Two set of strain gauges were placed at mid-height of the specimen opposite of each other. Vertical displacement of the specimens was recorded by Universal Testing Machine (UTM).

Table 6 shows the result of tested CFDST columns. The values in the table were the average of three specimens. The notation S3-C4/C3 under specimens name indicate series that the specimens belongs to and thickness of outer steel tube, t_o, respectively. The number 3 refers to $t_o = 3\,mm$ and number 4 refers to $t_o = 4\,mm$. Control, 60 and 90 indicates the condition of the specimens, where control refers to unheated specimens. Control specimens were tested as a reference specimens in order to compare the before and after fire exposure. 60 and 90 refers to maximum exposure time, 60 minutes and 90 minutes of fire exposure time, respectively. Finally, R and RH refers to repair with one layer of FRP and repair with Hybrid FRP, respectively.

Table 9. Result of tested concrete-filled double skin steel tubular columns

Diameter of Outer Steel Tube [mm]	Specimens	Ultimate Load [kN]	Displacement at Ultimate Load [mm]	Residual Strength Index [%]	Strength Enhancement Index [%]	Secant Stiffness [kN/mm]	Ductility Index
152.4	S3-C3-Control	1125	4.48	0	-	251	1.40
	S3-C3-60	951	7.29	15	-	131	1.31
	S3-C3-60-R	1124	10.10	0	18	111	1.14
	S3-C3-60-RH	1157	10.24	-3	22	113	1.13
	S3-C3-90	881	8.64	22	-	102	1.26
	S3-C3-90-R	996	10.28	11	13	97	1.13
	S3-C3-90-RH	1098	10.48	2	25	105	1.20
152.4	S3-C4-Control	1402	6.98	0	-	201	1.49
	S3-C4-60	1292	7.22	8	-	179	1.56
	S3-C4-60-R	1428	11.69	-2	11	122	1.17
	S3-C4-60-RH	1494	11.57	-7	16	129	1.18
	S3-C4-90	1199	9.26	15	-	129	1.80
	S3-C4-90-R	1401	12.39	0	17	113	1.27
	S3-C4-90-RH	1386	10.90	1	16	127	1.18

Details discussion on maximum temperature of concrete and inner steel tube after fire as well as discussion on failure pattern, ultimate load, Residual Strength Index (RSI), secant stiffness and Ductility Index (DI) after fire can be found in Mohd Zuki et al. (2017), Shahidan et al. (2016) and Zuki et al. (2015a; 2015b). This paper is only focusing on effectiveness of single and Hybrid FRP repair method.

Figure 5 and Figure 6 shows the ultimate load at failure and Strength Enhancement Index (RSI) of fire-damaged CFDST columns after being repaired with single and Hybrid FRP. Ultimate load and SEI increased as layer of FRP increased from single to Hybrid FRP. The SEI were as low as 11% and as high as 25% for S3-C4-60 and S3-C3-90, respectively. Overall, wrapping fire damaged specimens with single and Hybrid FRP was proven to be able to restore the ultimate load. The enhancement in both ultimate load and SEI is higher for specimen repaired with Hybrid FRP than those repaired with single FRP due to double confinement provided by Hybrid FRP.

From Figure 13, the highest SEI for specimen repair with Hybrid FRP was observed in specimens with t_o = 3 mm for both 60 and 90 minutes of fire exposure time. Therefore, Hybrid FRP is also proven to be able to effectively confine thinner t_0 than thicker t_0. For t_o = 3 mm, the increment of SEI for 60 minutes and 90 minutes of fire exposure time were 4% and 12%, respectively from single to Hybrid layer of FRP; whereas for t_o = 4 mm, the increment in SEI from single to Hybrid FRP for 60 minutes of fire exposure time was 5%. However, for 90 minutes of fire exposure time, there is no significant difference when the fire-damaged specimens are repaired with single and Hybrid FRP. This proves that the effectiveness of repair using FRP is affected by the t_0.

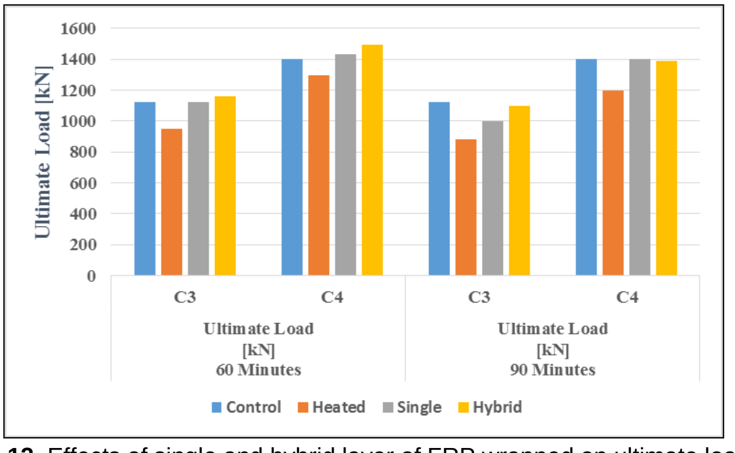

Figure 12. Effects of single and hybrid layer of FRP wrapped on ultimate load

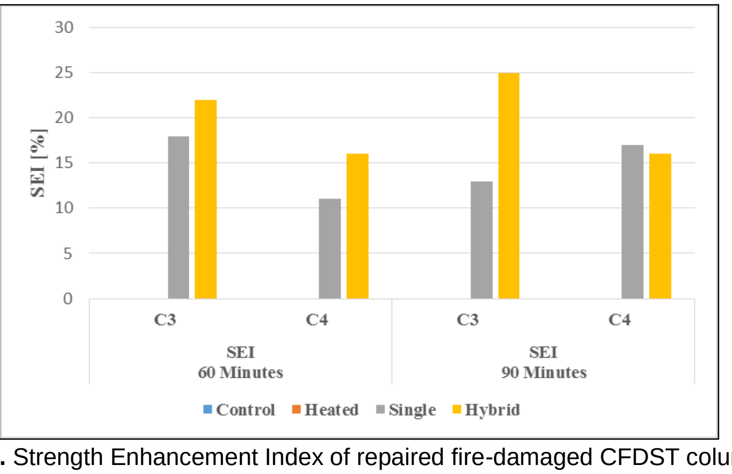

Figure 13. Strength Enhancement Index of repaired fire-damaged CFDST columns

The secant stiffness of specimen repaired with FRP increased with increase in layer of FRP as shown in Figure 14. The increment in secant stiffness was observed in both cases of t_0 ($t_0 = 3mm$ and $t_0 = 4\ mm$) and fire exposure time (60 and 90 minutes). However, the increment in secant stiffness is still not sufficient to fully restore the secant stiffness up to secant stiffness of control specimen. Nevertheless, it is expected that increasing in the number of FRP layers will increase the confinement, thus increases the secant stiffness of the fire-damaged specimens.

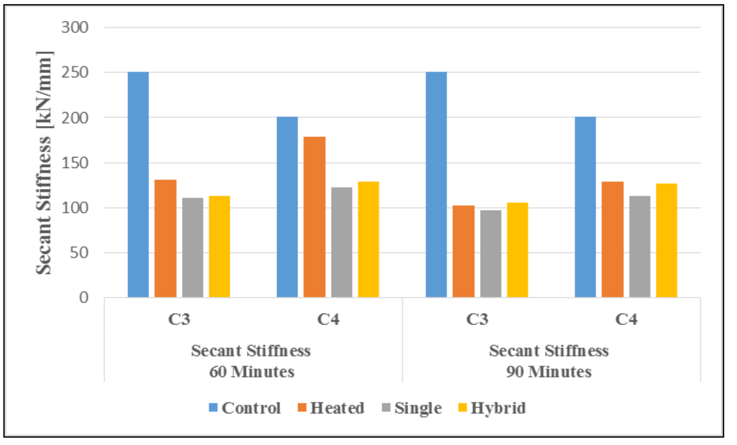

Figure 14. Secant stiffness of repaired fire-damaged CFDST tubular columns with Hybrid FRP

DI of the repaired CFDST columns with single and Hybrid FRP are shown together in Figure 15. The DI of specimen repaired with Hybrid FRP remains almost constant in comparison with specimens repaired with single layer of CFRP which could be attributed to failure mode (sudden rupture of FRP) of the FRP. The FRP is unable to provide lateral restraint to the deformation of the specimen once it failed. The more layers of FRP resulted in more sudden the nature of failure leading to almost constant DI. Sudden rupture of multilayers of FRP was also observed by other researchers such as Tao et al. (2007) and Che et al. (2012). However, both researchers used only one type of FRP in their research study, namely CFRP. Therefore, repairing fire damaged specimen with Hybrid FRP yielded similar result in DI as that using single layer FRP.

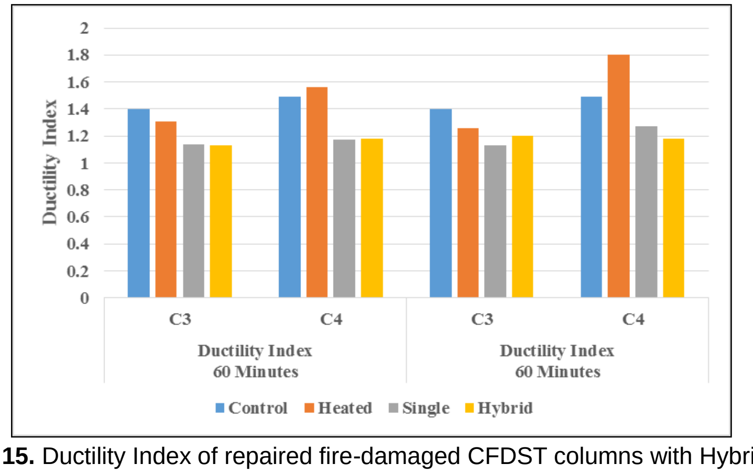

Figure 15. Ductility Index of repaired fire-damaged CFDST columns with Hybrid FRP

Figure 9 shows axial load versus axial-hoop strain curves for specimens with $t_o = 4\ mm$. Initially, it can be seen that the axial load-hoop strain curves for all specimens follow similar trend. There was a small expansion in hoop direction until the specimen reaches unconfined strength of the specimen. This expansion occurred in control and heated specimens. The expansion is due to the expansion of steel and concrete under increased axial load. Soon after this point, there is significant increase in hoop strain. With increased hoop strain, both control and heated specimen experienced gradual decrement or in some cases constant axial load. This can be attributed to the occurrence of local buckling of the specimens. This is also associated with the steel tube losing its confinement ability due to expansion of concrete.

Axial load-hoop strain curves of repaired specimens have no expansion in hoop direction as in control and heated specimens as illustrated in Figure 17. After the first portion of graph, both axial load and hoop strain increased steadily. During this stage, the CFRP effectively confined the steel tube. The presence of FRP delayed the occurrence of local buckling due to the confinement provided by the FRP wrapped. In addition, the specimens are able to carry more loads until failure of FRP. The failure of FRP is associated with sudden drop in axial load. During this stage, the additional strength provided by FRP confinement is no longer available thus resulting in sudden drop in load.

Based on the observation and from the axial load-hoop strain curves, wrapping with single layer of FRP on fire-damaged specimen is proven to be able to delay the occurrence of local buckling. Besides, the confinement provided by FRP further improved the load carrying capacity of fire-damaged specimens.

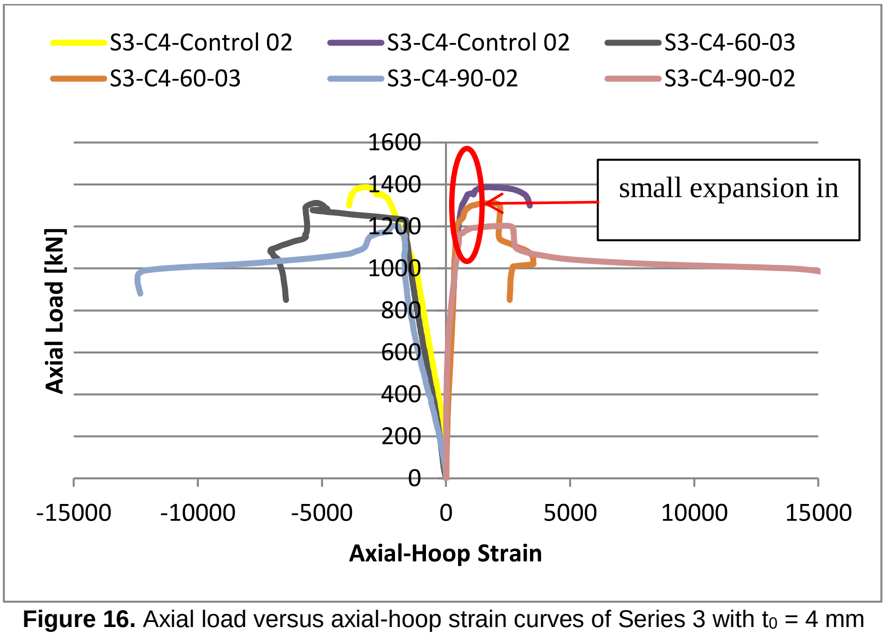

Figure 16. Axial load versus axial-hoop strain curves of Series 3 with t_0 = 4 mm

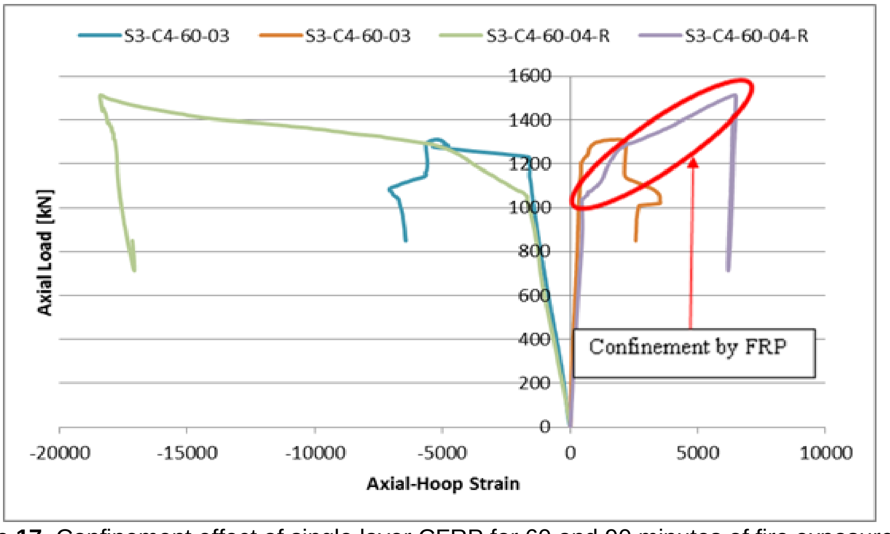

Figure 17. Confinement effect of single layer CFRP for 60 and 90 minutes of fire exposure time

CONCLUSION

Repairing of fire-damaged specimens with Hybrid layer of FRP resulted in strength enhancement exceeding the strength of control specimen. However, t_0 as well as fire exposure time of the specimens needs to be considered. If the specimens are exposed to fire for less than 60 minutes and the t_0 is thin, the use of single layer FRP is able to restore the strength of the specimen up to the strength of control specimen. Nevertheless, if the specimens are exposed to more than 60 minutes of fire, Hybrid FRP is the most suitable repairing scheme that should be adopted.

In addition, wrapping CFDST columns with FRP can delayed the occurrence of local buckling based on observation made during experimental test as well as result shown in Figure 9 and Figure 10.

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