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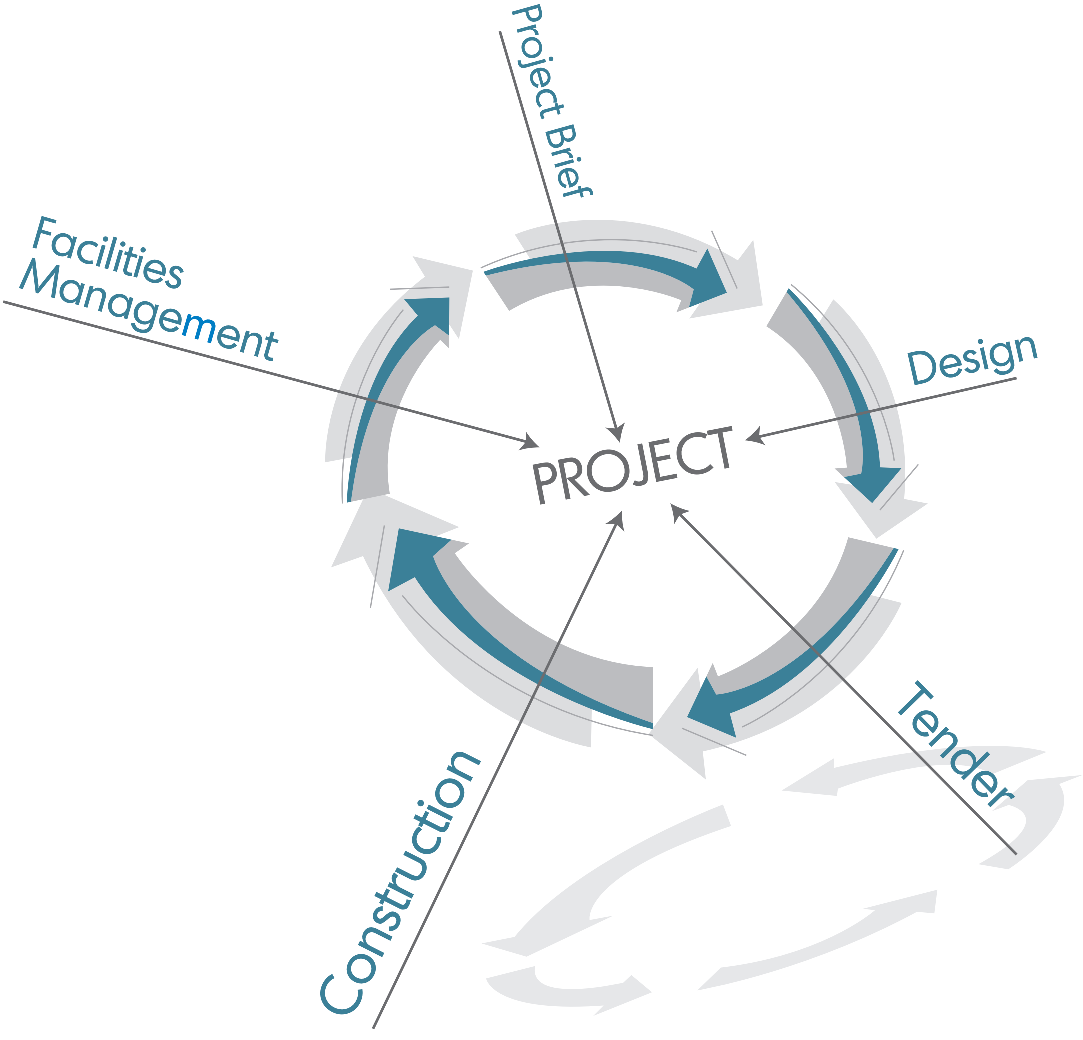

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Contents

Editorial Advisory Board

Editorial

Welcome from the Editors

Welcome to the twenty-eight (28th) issue of Malaysian Construction Research Journal (MCRJ). In this issue, we are pleased to include eight papers that cover a wide range of research areas in construction industry. The editorial team would like to express our sincere gratitude to all contributing authors and reviewers for their contributions, continuous support and comments.

In this issue:

Oh Chai Lian et al., investigated the usage of eggshells as an additive in concrete to reduce the environmental problems. Eggshells at certain level is studied for the fresh and hardened properties of self-compacting concrete (SCC). The results showed that the increment usage of eggshells in SCC mixes reduces the workability of SCC. However, it will increase the compressive strength for up to 29% as compared to the control mix.

Nurul Izzatul Lydia Zaba et al., evaluated the tensile strength for three species of Malaysian tropical hardwood which are Kempas, Keruing and Light Red Meranti. This study also compares the strength between small clear and structural specimens. The specimens were tested accordance to BS373:1957 for small clear specimens and BS EN 408:2010 for structural size specimens. The results show that the structural specimens have lower tensile strength as compared to the small clear specimens. It also shows that the higher strength group will give a higher tensile strength ratio.

Anis Azmi et al., determined the compressive strength properties for Malaysian hardwood tropical timber which are Keruing (KRG) and Light Red Meranti (LRM) parallel to the grain. The sampling was collected from four different regions and the 5th percentile of characteristics strength was determined. The results were measured through failure characteristic, compressive strength properties which shows that KRG employs higher compressive strength and modulus of elasticity (MOE) as compared to the LRM since KRG has higher density.

Mohd Fadzil Arshad et al., studied the effect of fine aggregate to heat of hydration value in cement mortar by comparing the heat of hydration of cement paste without aggregate with cement mortar containing river sand, quarry dust and kenaf core. The result shows that the inclusion of aggregate has reduced the heat of hydration of cement.

Kartika Puspa Negara et al., explored the public Client Project Manager (CPM) competencies in Indonesia. This paper clarifies the definition of CPM, review of project manager competency, review client project manager duties and propose standard definition of client project manager. The result shows that further research on the lack of competency of client project managers, specific standards of competency for CPMs in Indonesia and client project competency should be done.

Lorena Peñaherrera Bassantes et al., compared the analysis of the mechanical properties of concrete block masonry used in housing constructions within Argentina and Ecuador. The results show that the block analyzed in Mendoza was 100% met the requirements, while in Ecuador only 45% are accepted. There are great differences in the quality of the building Ecuador and Argentina.

Ali Sadrmomtazi and Oveys Ghodousian evaluated the effect of paste volume, water to cementitious materials ratio and fiber dosages on in-situ strength of fiber-reinforced self-compacting concrete as a repair layer. The result shows that the in-situ strength of repair layer increase considering the influence of shrinkage and tensile strength with reduce in paste volume and cementitious material and also increase in fiber dosages.

Md Yusof Hamid et al., examined the importance of space occupancy and utilization audit of a Malaysia Public University for future planning. The audit was conducted within three months through fieldwork activities, documentation review and retrieval including archival record review. The findings show that the space occupancy and space utilization rates play an important role in determining the future space required by a faculty or a university.

Editorial Committee

FRESH PROPERTIES AND COMPRESSIVE STRENGTH OF SELF-COMPACTING CONCRETE CONTAINING EGGSHELLS

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Abstract
Continuously increase in the eggshell waste in recent years has caused environmental and disposal problems. Therefore, the reuse of eggshell waste as an additive in concrete is a potential solution to reduce the environmental problems. Utilization of eggshells as a cement replacement in concrete is feasible owing to its great characteristic as a good accelerator for cement bound material. This study investigates the fresh and hardened properties of self-compacting concrete (SCC) containing eggshells at replacement levels of 0%, 5%, 10%, 15%. Water to cement and fine to coarse aggregate ratio are 0.35 and 0.55, respectively, were chosen in the design mix. The workability properties of SCC such as filling ability, passing ability and segregation resistance were evaluated through slump flow, T_{500} slump flow time, L-Box and sieve segregation tests. The compressive strengths of 24 cube specimens with dimensions of 150x150x150 mm were evaluated at 7 days and 28 days. It is found that, the increment of the use of eggshells in SCC mixes reduces the workability of SCC, however, increase the compressive strength for up to 29% as compared to the control mix.

Keywords: *Self-compacting Concrete; Eggshell; Fresh Properties; Workability; Compressive Strength.*

INTRODUCTION

Self-Compacting Concrete (SCC) has continuously gained interest from the practitioners in the construction industry since 1980s. Characteristics of SCC such as excellent flowability, passing ability and high segregation resistance make it suitable for a wide range of civil engineering and structural applications especially large structures and highly reinforced structures. SCC is able to fill and flow through narrow gaps in a formwork without any compaction. Good workability of SCC is achieved by high cementitious materials, content and low, coarse aggregate content in SCC. However, higher amount of cement content can increase the cost production of SCC. To overcome the problem, studies on the supplementary cementitious materials in replacing cement content in SCC and further improve the mechanical properties of the SCC are essential.

The compositions of SCC mixes include substantial proportions of fine-grained inorganic materials which give the possibilities for the utilization of mineral admixtures. Investigation works on SCC containing innovative materials such as SCC containing tyre rubber waste, plastic waste, fly ash, geopolymer and fibre are reviewed by Geeta et al. (2013) and Nuruddin et al. (2013). The use of various agro industrial waste in SCC has shown positive impact on the mechanical properties of SCC (Geeta et al., 2013; Adnan et al., 2017). This also means the workability parameters and the compressive strength of SCC are preferable compared to conventional concrete. Specifically, Fakitsas et al. (2011) compared the performance of SCC with natural rock aggregate and recycled concrete aggregate. It was found that the SCC with recycled coarse aggregates demonstrated superior compressive and frictional characteristics. Khodair and Bommareddy (2017) studied SCC with recycled concrete aggregates and high volume of fly ash, and slag. The findings inclusive of the fresh, hardened, and durability characteristics of a total of twenty mixes were presented. Beycioğlu and Aruntaş (2014) investigated the effect of Low Lime Fly Ash (LLFA), Granulated Blast Furnace Slag (GBFS) and Micronized Calcite (MC) on both workability and mechanical properties of SCC. The study on the mixes showed positive results in the flow-ability, passing ability and viscosity of SCC.

On the other hand, waste products are one of the main sources that contribute to environmental pollution. There are various types of waste disposal such as land filling, open burning, drains clogged up with rubbish and solid waste. The waste management in Malaysia is facing great challenge owing to the increasing production of huge waste. Figure 1 shows the municipal solid waste generated by various sources in Kuala Lumpur, Malaysia. It is noted that the food or organic waste is the highest generated waste compared to paper, plastic and others. Waste recycle is one of many ways to utilize the waste efficiently. Nowadays, there are many types of waste products such as rubber tyre, silica fume, and bottom ash that are used in construction industry to reduce the amount of waste while maximizing the profit. The construction industries were put an effort to search for alternative products that can reduce the construction cost.

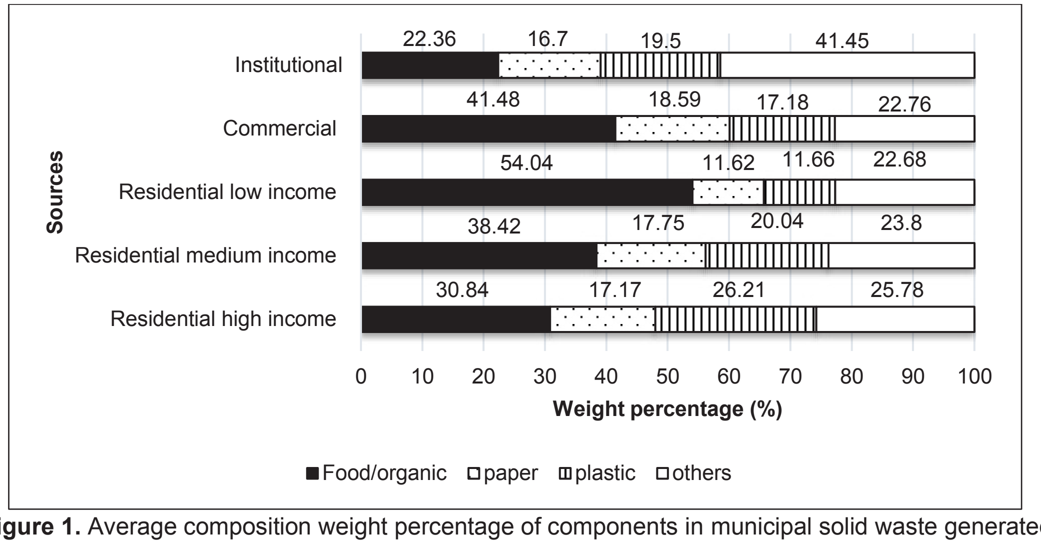

Figure 1. Average composition weight percentage of components in municipal solid waste generated by various sources in Kuala Lumpur. Data reproduced from Kathirvale et al. (2004).

Eggshells generated from reliable sources such as fast food restaurant, bakeries and chick hatcheries are agricultural waste materials. In the ever-increasing efforts to convert waste to wealth, the efficiency of changing eggshells to beneficial use becomes an idea worth embracing. Eggshells are scientifically known that mainly composed of compounds of calcium, which is similar to the cement. Eggshell powder generally contains lime, calcium and protein where it can be applied as an alternative raw material in the production of concrete. Eggshells can also be beneficial to the construction industry, which is able to reduce the construction cost and landfill in addition to give good performance properties in concrete. Therefore, eggshells are suitably used as green material for development in the construction industry such as an additive in the concrete mixes. Especially earlier study of the application of eggshell powder as an additive in concrete production has provided faithful results. Eggshells which come from waste from food industry are known to have good strength characteristics when used in concrete mix.

Recently, the use of eggshells in concrete has been investigated. Nadu (2014) conducted experimental study to investigate the performance of eggshells as partial replacement for sand in concrete. Concrete with 20% eggshells has shown an increase in the compressive and flexural strength compared to conventional concrete. Besides, Gowsika et al. (2014) performed experimental investigation on eggshells powder as partial replacement for cement in concrete. Dhanalakshmi et al. (2015) carried out a comparative study on eggshells as partial replacement of cement with addition of fly ash in the concrete mix. Raji and Samuel (2015) conducted experimental work on eggshells as a fine aggregate in concrete for sustainable construction. However, there is limited study on the use of eggshells in SCC.

The study aims to investigate the effect of the use of eggshells in the fresh properties and compressive strength of SCC. The concrete tests for fresh and hardened properties were conducted with different percentage of eggshells in SCC (i.e. 0%, 5%, 10% and 15%). The control SCC was used as a reference to compare compressive strength in the hardened state at the age of 7- and 28- days.

The remainder of the paper is organized as follows: Section 2 presents the materials for SCC and methodology adopted in the study. Section 3 presents the results and discussions on the fresh properties and compressive strength of SCC containing eggshells. Lastly, conclusions and recommendations are presented in Section 4.

MATERIALS AND METHODOLOGY

Materials

Five main materials for the preparation of SCC concrete mix in the study are as follows:

- Cement: Ordinary Portland Cement (CEM1) by local supplier was used in the experimental work. The cement is manufactured according to Malaysian Standard MS 522: Part 1: 2003 'Specification for Portland Cement (Ordinary and Rapid-Hardening): Part 1: Standard Specification'.
- Eggshells: The eggshells collected from the local restaurants and home supplies were first cleaned with normal water and dried under the sun. The eggshells were crushed into small particles by Ball Mill (Two Tier Jar) machine and were sieved by sieving machine. The ones passing through the sieve pan with a size of 2.36 mm were used in the SCC concrete mix.
- Coarse aggregates: Crushed granite with a maximum size of 10 mm was used as the coarse aggregate.
- Fine aggregate: Natural river sand with 0.3 - 0.8 mm in diameter was used as the fine aggregate.
- Superplasticizer: Superplasticizer MasterGlenium Sky 8784 supplied by Baden Aniline and Soda Factory (BASF) (Malaysia) Sdn. Bhd. was added to the concrete mixture. Table 1 shows the properties of the superplasticizer.
- Water: Drinking water is used for the mixing and curing.

Table 1. Properties of BASF MasterGlenium Sky 8784

Properties	Description
Aspect	Light brown liquid
Relative Density	1.10 at 25°C
Ph	≥ 6
Chloride ion content	< 0.2%

Mix Design and Specimen Preparation

Table 2 shows the proportions of the SCC concrete mixes in the study. Four concrete mixes were designed in order to determine the effects of eggshells in the fresh properties and compressive strength of SCC. The concrete mixes SCC0, SCC5, SCC10 and SCC15 denotes the mixes with 0%, 5%, 10% and 15% eggshells, respectively. Specifically, the cement content of 543 kg/m^3 was replaced with eggshells of 0%, 5%, 10% and 15% by weight in the concrete mixes. All the concrete mixes were designed with a constant water to cement ratio of 0.35 and the fine to coarse aggregate ratio of 0.5. The superplasticizer content was kept to 1% of the cement content by weight.

Table 2. Concrete mixes proportions

Mix	Cement kg/m^3	Water litre/m^3	FA kg/m^3	CA kg/m^3	ES kg/m^3	SP litre/kg/m^3	w/c ratio	FA/CA	SP/ cement %	ES %
SCC0	543.00	190	401.75	803.5	0.00	5.43	0.35	0.5	1	0
SCC5	515.85	190	401.75	803.5	27.15	5.16	0.35	0.5	1	5
SCC10	488.70	190	401.75	803.5	54.30	4.89	0.35	0.5	1	10
SCC15	461.55	190	401.75	803.5	81.45	4.62	0.35	0.5	1	15

Note: FA- Fine aggregate, CA- Coarse Aggregate, ES – Eggshells, SP- Superplasticizer

Figure 2 summarizes the methodology of the study. All the experimental works were conducted in the laboratory of heavy structure, Faculty of Civil Engineering, UiTM Shah Alam, Selangor. Mixing of the concrete materials was conducted in sequence as follows: (i) superplasticizer was added to the mixing water in a pail; (ii) all dry materials (cement, eggshells, coarse and fine aggregates) were placed in the mixer pan and mixed for 2 minutes; (iii) some water containing superplasticizer were slowly added to the mixer pan, for a mixing period of 2 minutes; (iv) the remaining water containing superplasticizer was added to the mixer pan for another 30 seconds.

Once the mixing process completed, the fresh concrete mixes were tested for the properties such as the filling ability, passing ability and segregation resistance. SCC cube specimens from the thick paste of fresh concrete mix were also cast and covered to prevent evaporation for 24 hours. The specimens were then demoulded and the curing process in normal water continued for 7 and 28 days. A total of 24 numbers of 150 x 150 x 150 mm cube specimens was prepared for the compressive strength test.

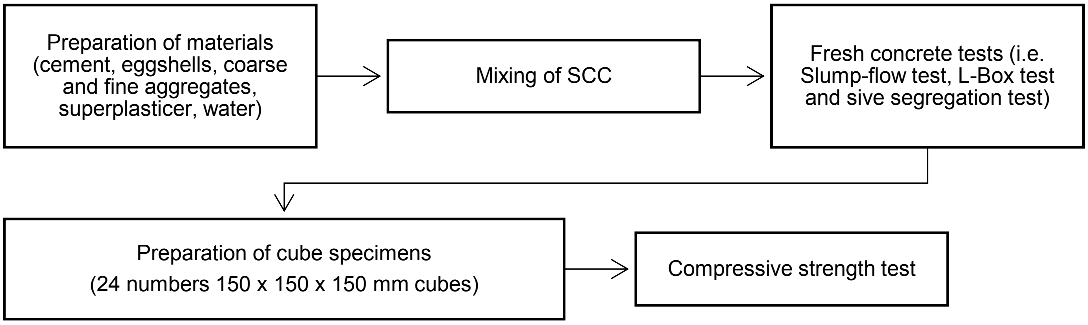

Figure 2. Summary of experimental works

Fresh Properties Tests

This section presents the test procedures for the fresh SCC mixes namely, slump-flow test, L-Box test and sieve segregation test.

A slump-flow test was conducted in accordance to BS EN 12350–8:2010. The test determines the slump-flow and t_{500} time for SCC. The slump-flow and t_{500} time are used to assess the workability (i.e. filling ability) and flow rate (i.e. relative viscosity) of the concrete mixes. In slump-flow test, the fresh concrete mix was placed in a cone without any rodding. After the cone was lifted, without disturbing the base plate or concrete, the largest diameter of the flow was recorded as d_1 and the diameter of the flow spread at right angles to d_1 was recorded as d_2 (see Figure 3a). The flow diameter d_1 and d_2 were measured to the nearest 10 mm. The slump-flow of a mix was taken as the mean of d_1 and d_2. At the same test, the t_{500} time was recorded for the period the cone was lifted by the flow reached a diameter of 500 mm.

On the other hand, L-box test was conducted in accordance to BS EN 12350–10:2010 to measure the passing ability of the concrete mixes. The fresh concrete mix was poured into the L-Box (see Figure 3b) and flowed through the narrow gap provided by the three reinforcing bars. The average heights for H_1 and H_2 were determined, particularly from the heights of three positions, one at the centre and two on each side. The heights H_1 and H_2 (see Figure 3c) are the height of the concrete mix at the top edge and at the end of the box, respectively once the concrete mix stops to flow. The passing ability of the concrete mix is taken as the ratio H_2 / H_1 to the nearest 0.01.

The sieve segregation resistance test was conducted based on BS EN 12350–11:2010. The test assesses the resistance of concrete mix to segregation. The good segregation resistance of the concrete mix allows the concrete to remain consistent during mixing, transport, and placing. Segregation resistance may directly affect the hardened properties such as strength and durability. The test was conducted with a mass of 10kg of fresh concrete placed in a sample container without any disturbance for 15 minutes. Bleeding of water that appeared on the surface of the concrete was checked and recorded. The mass of the material that has passed through the sieve were recorded.

The segregation resistance SR was calculated in percentage as follows:

$$\mathrm{SR} = \frac{\left(m_{\mathrm{ps}} - m_{\mathrm{p}}\right)}{m_{\mathrm{c}}} \times 100\% \tag{1}$$

where m_{ps} is mass of sieve receiver plus passed material, m_p is mass of the sieve receiver and m_c is initial mass of concrete placed onto the sieve.

Compressive Strength Test

Testing on mechanical property specifically the compressive strength of the cube specimens was conducted based on BS EN 12390–3:2009. The cubes were air-dried first about 10 to15 minutes and any obstructions on the surface of the cubes were removed before putting on the bearing plate of the compression testing as shown in Figure 3d. The machine was used to determine the maximum compressive strength of the concrete specimen cubes at 7 and 28 days. Average of compressive strength was taken from three cube specimens for each concrete mix, particularly for concrete at 7 and 28 days.

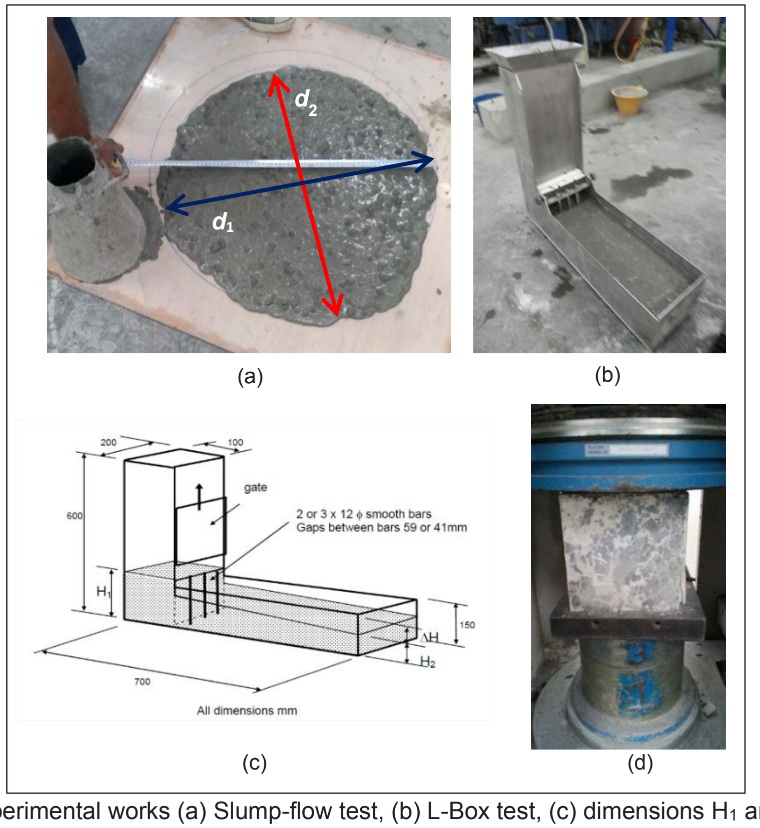

Figure 3. Experimental works (a) Slump-flow test, (b) L-Box test, (c) dimensions H_1 and H_2 and (d) compressive strength test

RESULTS AND DISCUSSIONS

Fresh Properties of SCC

This section discusses the effect of different percentage of eggshells on fresh properties of SCC. Table 3 shows the classification of SCC based on the results of slump-flow test, L-box test and sieve segregation resistance test according to the guidelines from The European Federation of Specialist Construction Chemicals and Concrete Systems (EFNARC).

Table 3. SCC Classes (EFNARC, 2002)

Classes	Name	Range
Slump-flow (mm)	SF1	550 to 650
	SF2	660 to 750
	SF3	760 to 850
Viscosity (s)	VS1	≤ 2
	VS2	> 2
Passing ability (L Box H_2/H_1)	PA1	≥ 0,80 with 2 rebars
	PA2	≥ 0,80 with 3 rebars
Segregation resistance (Sieve segregation %)	SR1	≤ 20
	SR2	≤ 15

Slump Flow

Figure 4a illustrates the variation of slump flow of the concrete mixes with different percentage of eggshells. The slump flow with range of 550 – 640 mm were recorded. Workability of the concrete mixes in the present study was affected by the percentage of the eggshells in the mixes. It is noted from Figure 4a that the slump flow decreases with the increment of the eggshell's percentage. The slump flow of the concrete mix with the use of 15% eggshells shows 14% lesser than the slump flow in the control mix (i.e. 0% eggshells). This is probably due to the larger organic particles in eggshells (i.e. 2.36 mm) compared to the cement used at SCC. Based on the test results, all the concrete mixes can be categorized as slump flow class SF1 (see Table 3), which are suitable for small sections that are not subjected to long horizontal flow as in pile applications (EFNARC, 2002).

T_{500} Time

Figure 4b illustrates the variation of T_{500} time of the concrete mixes with different percentage of eggshells. The T_{500} time of more than 2 seconds are recorded for all the concrete mixes. The concrete mixes with the increment of eggshells percentage generally show increasing trend in T_{500} time. It is clearly seen from Figure 4b that the highest T_{500} time is recorded for the concrete mix with 15% eggshells replacement. The slower flow rate in concrete mixes with greater eggshells could be related to the rougher surface of the eggshells particle compared to the cement. All the concrete mixes can be classified as viscosity class VS2 (see Table 3) which may be helpful in enhancing segregation resistance (EFNARC, 2002).

L-Box Height Ratio

Figure 4c illustrates the variation of L-Box height ratio (H2/H1) of the concrete mixes with different percentage of eggshells. The L-Box height ratios with range of 0.87 - 0.97 were determined for all concrete mixes with different percentage of eggshells. It is noted that the decreasing trend of L-Box height ratio is observed in the concrete mixes when the eggshells percentage increase. All concrete mixes show L-Box height ratio greater than 0.8 and thus can be classified as passing ability class PA2 (see Table 3), specifically for the test with three rebars (EFNARC, 2002). The passing ability specification of greater than 0.8 is given by EFNARC (2002) to avoid the risk of blocking when the fresh concrete mix flow through areas with congested reinforcements.

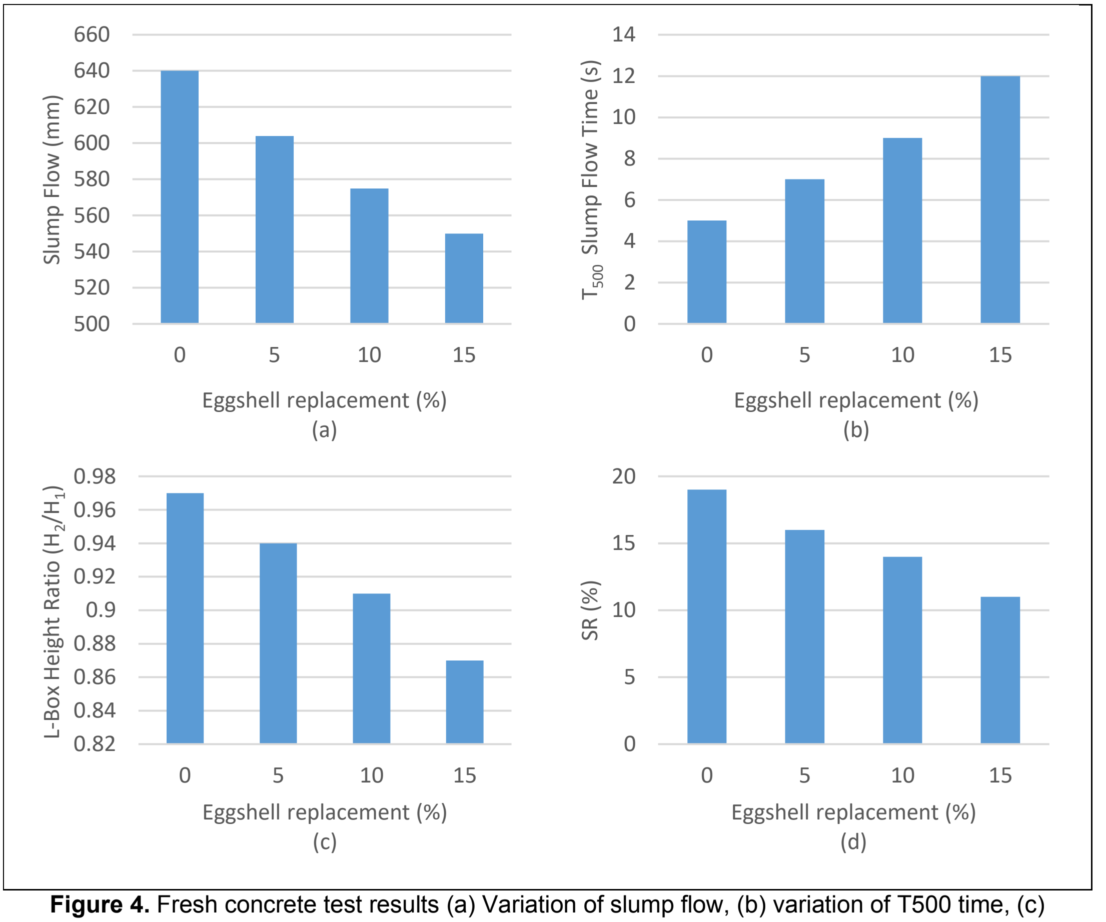

Figure 4. Fresh concrete test results (a) Variation of slump flow, (b) variation of T500 time, (c) variation of L-box height ratio and (d) variation of SR in SCC containing eggshells.

Segregation Index

Figure 4d illustrates the variation of segregation index of the concrete mixes with different percentage of eggshells. Segregation index of 19% and at range of 11-16% were recorded for the control mix and the concrete mix with different percentage of eggshells. The decreasing trend in the segregation index is observed when the percentage of eggshells in the concrete mix increase. The lowest segregation index is found in concrete mix with 15% eggshells replacement. Utilization of eggshells significantly reduces the segregation and therefore reveals the potential of ensuring a high level of homogeneity in the mixes. According to EFNARC (2002), the control mix and the concrete mix with 5% eggshells replacement can be classified as segregation resistance class SR1, whereas the remaining concrete mix with 10% and 15% eggshells replacement as segregation resistance class SR2 (see Table 3).

Compressive Strength

Figure 5 shows the results of compressive strength for the concrete mixes with the use of different percentage of eggshells at 7 days and 28 days. All the concrete mixes have achieved the target compressive strength of 70 MPa at the age of 28 days. It can be observed from Figure 5 that the control concrete after 7 days of curing gained 74% over its 28 days compressive strength. The concrete with eggshells replacement after 7 days of curing gained 74-81% over its 28 days compressive strength.

Additionally, the use of eggshells as cement replacement shows a positive effect on the compressive strength of SCC. It could be seen from Figure 5 that the compressive strength increases for the concrete with a higher percentage of eggshells. Specifically, concrete with the use of 5%, 10% and 15% of eggshells show higher compressive strength compared to the control concrete with the increment of 7%, 12% and 29%, respectively.

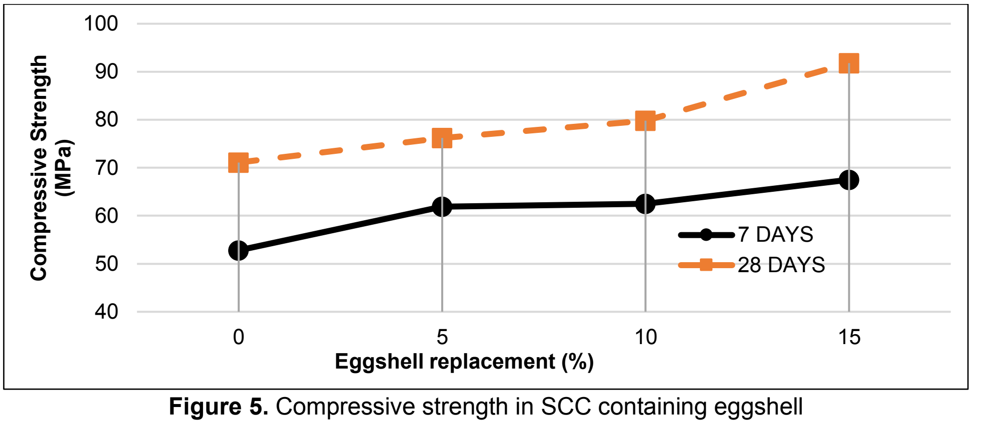

Figure 5. Compressive strength in SCC containing eggshell

CONCLUSIONS

In conclusion, it is possible to use waste material such as eggshells powder as cement replacement at 0%, 5%, 10%, and 15% in producing SCC. The experimental testing on fresh properties such as L-Box, slump flow and sieve segregation test have confirmed all the concrete mix as SCC in the study. The results of the fresh properties are L-Box, PL value more than 0.75, Slump flow, average diameter more than 550mm, and Sieve segregation with SR value less than 20%. The increment of eggshells powder percentage in mix reduces the performance of the workability of the fresh SCC. For hardened properties, compression test has observed at 28 days, the highest value of compression strength is at 15% eggshells powder replacement with 91.78 N/mm2 and the lowest value is at 0% eggshells replacement with 71.11 N/mm2. The addition of eggshell powder increases the strength, performance of SCC up to 29%.

ACKNOWLEDGMENTS

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TENSILE STRENGTH PROPERTIES OF SMALL CLEAR AND STRUCTURAL SIZE SPECIMENS OF KEMPAS, KERUING AND LIGHT RED MERANTI

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Abstract
There is no direct measurement of tensile stress have been determined for Malaysian tropical hardwood. The tensile stress of timber is usually taken as 60% of the bending strength values of bending small clear specimen as found in MS 544: Part 2:2001. This also implemented in BS EN 338:2016 where the tensile stress of hardwood timber is also determined by 60% of bending characteristic value. Therefore, in this study, three species of Malaysian tropical hardwood were selected, namely Kempas (Koompassia Malaccensis spp.), Keruing (Dipterocarpus spp.) and Light Red Meranti (Shorea Siamensis spp.) were used to determine the tensile strength and to compare the strength between small clear and structural specimens. Specimens are prepared and tested accordance to BS373:1957 for small clear specimens and BS EN 408:2010 for structural size specimens. The tensile properties are then presented as tensile strength and modulus of elasticity. Based on this test, it was found that structural specimens have relatively lower tensile strength as compared to small clear specimens. Besides that, tensile strength ratio analysis shows that higher strength group will give a higher tensile strength ratio. In terms of relationship, it is clear that there is a relationship between density, modulus of elasticity (MOE) and tensile strength, however a poor relationship has been discovered between MOE and density in both small clear and structural size specimens.

Keywords: *Hardwood Timber; Small Clear Specimen; Structural Size; Modulus of Elasticity; Malaysian Tropical Timber; Tensile Strength.*

INTRODUCTION

Timber is one of the oldest construction materials. It can be done even without using heavy machines. Malaysia is a country that covered with lush tropical rainforest and there are more than 2500 species of timber. However, only about 100 species that have been investigated for strength properties as listed in MS 544: Part 2. On the other hand, about 10% of these timber species have been identified as suitable for structural elements with unlimited supply if re-plantation is carried out (Yusof, 2001). The density of Malaysian heavy hardwood ranges from 880kg/m^3 to 1200kg/mm^3, for medium hardwood the density ranges from 720kg/mm^3 to 880kg/m^3 and for light hardwood the density is lower than 720kg/m^3 and non-durable unless treated (MS 544: Part 2:2001).

Timbers are used as structural members such as roof trusses, box beam and column where the structural elements can be subjected to tensile, bending and compression. The strength of the timber is particularly important in designing those of structural component system and the strength of timber normally associated with the density, the higher the density of timber generally the greater its mechanical strength. However, this relationship is not an absolute since there are other factors that can alter this relationship, such as the presence of defects and sizes.

The strength data in MS 544 Part 2, is determined based on small clear specimens. Clear specimen means there is no defect in the timber. Technically, stresses obtain from small clear specimen's method are convenient to be used in determining the timber mechanical properties (Wahab et al., 2013). However, it cannot provide accurate values for structural engineering applications due to great biological materials such as growth of defects in the form of knots, zones with compression wood, oblique fibre direction, etc. Such growth characteristics, which were once created to serve the needs of the tree, will usually reduce the strength significantly when the timber is cut and planed. Furthermore, the presence and character of knots and other defects vary from one timber board to another as well as when using different cutting pattern will give different structural properties.

This is why the stresses in small clear specimens need to be modified by reducing certain factor to reflect the structural size strength. Therefore, the difference in size and the amount of defect will affect the performance of tensile strength. In some foreign country such as the United Kingdom, the used of small clear specimen in determining the strength has been abandoned. There is new approach known as limit state design which using structural size specimens to determine the characteristic stress, which the value taken not more than 5% in the normal distribution (Ahmad et al., 2010).

In compression and tension, timber is strongest in the direction of growth and crushes much more readily across the grain. The force generating tension parallel to grain localize a tendency to elongate the wood fibres and to cause them to slip by each other. Resistance to the tension applied strictly parallel to grain is the highest strength property of wood (Wang et al., 1990). This resistance however is substantially reduced when the force is applied at an angle or when the cross-section of the pieces is reduced by knots or holes. Knot and variations in grain orientation are the most significant growth characteristics that affecting the tensile strength (Ahmad, 2010). The effect of tensile parallel to grain and tensile perpendicular to grain of small clear timber can be 40 to 1 (Wellons & Kramer, 1973). In timber design, the lower truss chord or the tension flange of a box beam and an I beam is a very critical design parameter (Chin, 1997; Azizan et al., 2016). There are several reasons why the high tensile strength of timber cannot be utilized in construction, one of the reasons because of the shearing strength along the grain is extremely low (6-10%) in comparisons to the tensile strength along the grain. Thus, the timber is tending to fail at the shear or cleavage at the fastening or joints (Kollman, 1968).

The value of tensile strength is also important for designing glued laminated timber as given in EN 14080. At present, there is no tensile strength value of Malaysian tropical hardwood timber and to use T-class data in Table 2 (EN338) for hardwood timber will be unjustified. There is a need to establish T-class for Malaysian hardwood timber. Therefore, this study investigates the tensile strength properties of Malaysian hardwood timber in structural size and at the same time tensile strength properties of small clear specimens are also determined. This paper just reports the comparison of the tensile strength properties of small clear and structural size specimens of three species.

EXPERIMENTAL PROCEDURE

Three species were selected in this experimental study which is Kempas (Koompassia Malaccensis spp.) from strength group (SG) 2, Keruing (Dipterocarpus spp.) from SG 5 and

Light Red Meranti (Shorea Siamensis spp.) from SG 6. The strength grouping for each timber is based on Malaysian Standard MS 544 Part 2. All of these timbers are visually graded by professional graders from Malaysian Timber Industry Board (MTIB) and it were categorized under standard and better for small clear specimens and Hardwood Structural Grade (HSG) for structural size specimens. Timber were air dried for few months until it reached a moisture content under 19%, which also known as dry condition.

The specification of the specimen used for determining the tensile test for small clear specimens was adapted from BS373:1957 (Method of testing small clear specimens of timber) and illustrated as in Figure 1. The cross-section at the minimum cross-section and weight are measured and recorded for each piece. Overall, there are 60 specimens of each type of timber.

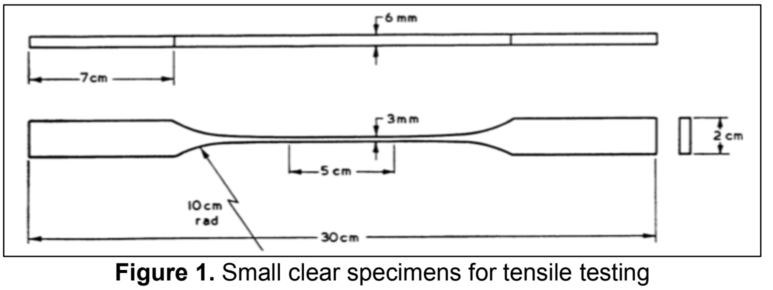

Figure 1. Small clear specimens for tensile testing

Tensile test of small clear specimens was performed by using Shimadzu Precision Universal Tester -Autograph AG-50kNICD- Both ends of the tensile specimens was axially clamp (see Figure 2) and load was applied to the specimen at constant head speed of 1.27mm/min. Strain of the specimens is measured between the gauge length (G.L) of 50mm by using high resolution video extensometer attached to the machine. The gauge length is automatically measured before the beginning of each test and used for strain calculation, thus eliminating errors of manually marking.

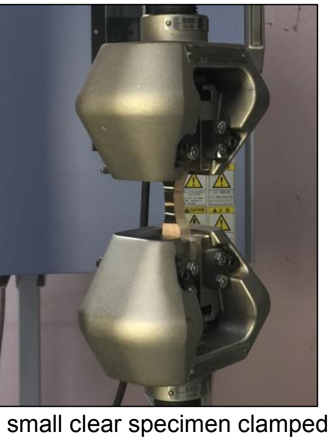

Figure 2. Tensile small clear specimen clamped by the machine

For structural size specimens, BS EN 408:2010 was used as a reference to prepare the specimens. As indicated in the standard, the specimens are in a full structural cross section and the clear length (C.L) of the specimens should at least nine times the larger cross section (see Figure 3). Therefore, the specimens are planning on four sides to the size of 20mm x 80mm x 2670mm. The actual dimension and weight were recorded for each specimen.

Tensile machine 2000kN (see Figure 4) was used to perform a tensile test of structural specimens. The load applied at a constant loading-head movement is adjusted so that the maximum load is reached within 300 (120sec. Two extensometers are used to measure the deformation for determination of MOE. Deformation was measured five times the width of the specimens which in this study the gauge length is 400mm. A total of 60 specimens for each species were tested.

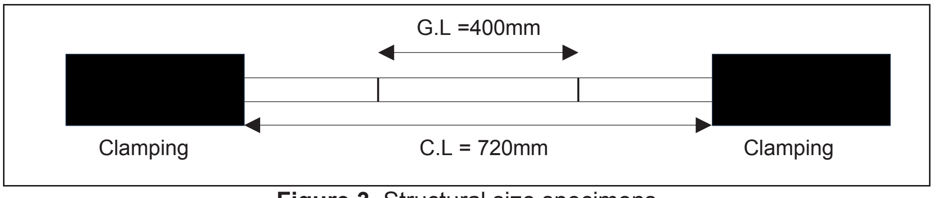

Figure 3. Structural size specimens

Figure 4. Tensile machine 2000kN

A small slice of a full cross section of each specimen with 25mm thickness was cut at a distance not too close to the end of measured moisture content by the oven dry method immediately after testing. The samples were oven-dried at 103 (2 (C until constant weight was obtained. Hence the moisture content is calculated by the ratio of oven dried weight to air dried weight at the time of the test.

RESULTS AND DISCUSSION

Stress Versus Strain

Figure 5 shows typical stress versus strain graph for tensile strength of each species for both small clear and structural size specimens. The graph shows an approximately linear line between strain and stress. The slope of the graph represents MOE. The ultimate tensile strength was determined when the load reached the highest point. It can be seen that after reaching the maximum load, the specimen broke abruptly, which indicates that timber is brittle under tension.

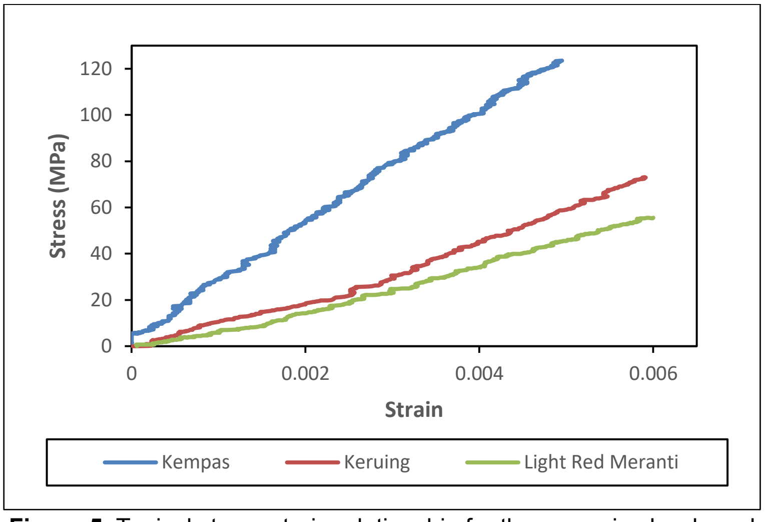

Figure 5. Typical stress-strain relationship for three species hardwood

Tensile Strength Properties of Timber in Small Clear Specimens and Structural Size Specimens

Three Malaysia tropical timber species were tested for tensile tests. The summary of mean tensile test, mean MOE, density and moisture content for each species are shown in Table 1 for small clear specimens and Table 2 for structural size. It can be seen that the tensile strength of small clear specimens is much higher than the tensile strength in structural size. This is because, small clear specimens are assumed to be entirely free from defects such as decay, knots, wanes and others. However, for structural size, defects and irregularity in growth considerably will reduce the tensile strength. An analysis of variance (ANOVA) was performed to determine if there are differences in mean tensile strength values among small clear specimens. F-test indicated at 5% significance level, there are significant different (p-value = 0.000) in the mean tensile strength properties of all three species in small clear specimens. Similar trends have been recorded for structural size specimens.

Table 1. Summary of tensile strength properties of small clear specimens

Species	Mean Tensile strength (MPa)	Mean MOE (GPa)	Density (kg/m^3)	Moisture Content (%)
Kempas	132.8±33.9	24.4±6.6	1076	11.5
Keruing	105.4±22.8	15.2±5.0	794	12.1
Light Red Meranti	73.6±15.1	12.1±2.7	493	11.8

Table 2. Summary of tensile strength properties of structural size specimens

Species	Mean Tensile strength (MPa)	Mean MOE (GPa)	Density (kg/m^3)	Moisture Content (%)
Kempas	94.6±26.5	23.2±5.8	920	15.4
Keruing	67.4±19.9	15.9±4.6	772	17.3
Light Red Meranti	39.1±14.1	10.2±2.6	490	16.5

For small clear and structural size specimens, the tensile strength and Modulus of Elasticity of Kempas is higher than Keruing and Light Red Meranti.

One of the best ways to express the relationship of small clear and structural size specimens are by strength ratio (Alik & Badrul). The actual tensile strength ratio (TS_{ratio}) was calculated based on (Doyle & Markwardt, 1967) which the tensile strength of structural size dividing by the tensile strength of small clear specimens.

$$Ts_{ratio} = \frac{Ts_{structural}}{Ts_{smallclear}} \qquad \text{Equation 1}$$

The mean values of the actual tensile strength ratio (TS_{ratio}) were found to be 71% for Kempas, 64% for Keruing and 53% for Light Red Meranti. Based on that, the tensile strength ratios being lower at the lower strength group. It also found by Mark et al. (2016) where the flexural strength ratio between small clear specimens and structural specimens in static bending test were decreasing as the flexural strength decrease.

The MOE ratios (Moe_{ratio}) were calculated by dividing MOEs of structural size specimens by MOE of small clear specimens.

$$Moe_{ratio} = \frac{Moe_{structural}}{Moe_{smallclear}} \qquad \text{Equation 2}$$

The average values of MOE_{ratio} were found to be 95% for Kempas, 105% for Keruing and 84% for Light Red Meranti. The MOE_{ratio} are in different trend from TS_{ratio} which Keruing seems to give higher MOE_{ratio} compared to Kempas. It is because the MOE of the structural size specimens for Keruing is much higher than the MOE value for Keruing small clear specimens. However, F-test indicated a p-values = 0.47 at the 5 % significance level that indicates no significant difference of Kempas and Keruing between MOE of structural size and MOE of small clear specimens. Yet, there is a significant difference in Light Red Meranti (p-values = 0.000).

Based on the result of the strength ratio, it can be concluded that there are poor to moderate relationship obtained from small clear specimens for predicting the structure size tensile strength. The poor relationship may occur by numerous factors which including the amount of variability of each specimen and also a different failure mode for both small clear and structural specimens.

Relationship of Density, MOE and Tensile Stress Between Small Clear and Structural Size Specimens

There is a lot of research shown that increased density may lead to increasing of stiffness, strength and MOE (Wahab et al., 2013). The relationship between these mechanical properties were assumed linear and positive. Figure 6 and Figure 7 show the relationships between air dry density, MOE and tensile stress. It can be seen that generally the tensile strength increases as the density increases thus increasing in MOE. The regression line in the graph indicates a significant correlation if the R^2 values are approaching 1. It can be seen that there is good correlation between density and tensile strength of structural size specimens ($R^2 = 0.6$) and moderately correlated for small clear specimens ($R^2 = 0.4$). In the case of MOE and tensile strength, there is good correlation observed between MOE and tensile strength which is $R^2 = 0.43$ for small clear and $R^2 = 0.54$ in structural size. Overall, it can be seen that structural size specimens give a better correlation compared to small clear specimens. Correlations in this finding indicate that controlling the density can give positive effects on tensile strength.

However, there are weak correlations observed between density and MOE for both small clear ($R^2 = 0.02$) and structural size ($R^2 = 0.14$) as shown in Figure 8. The correlation of the density with strength and MOE are usually very strong. However, there are few researches have also found a weak relationship between MOE to specific gravity in some coniferous species such as *Abies Fabri* and *Pinus massoniana* (Zhang, 1997) and in fast growing red pine (Deresse, 1998). There has a research that reported specimens having similar specific gravity can also exhibit significantly different strength values due to factors that may be associated to other factors to which specific gravity is less sensitive (Deresse, 1998 & Zhang, 1997).

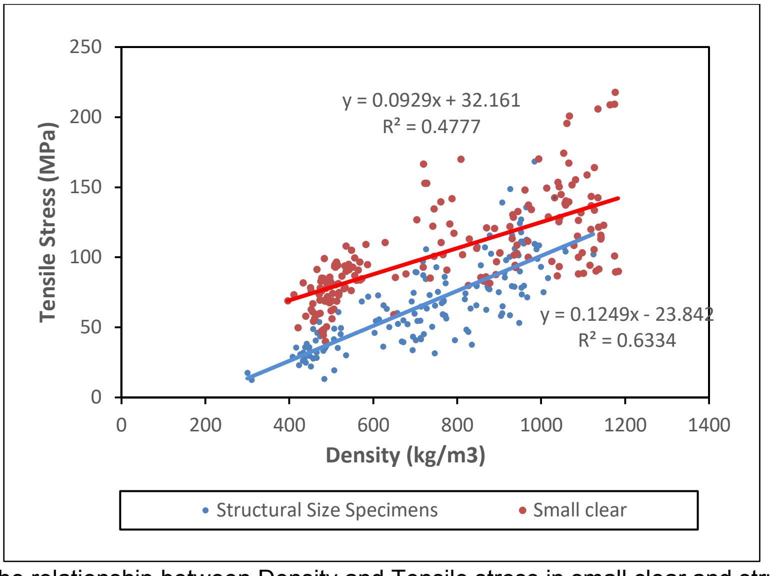

Figure 6. The relationship between Density and Tensile stress in small clear and structural size specimens among three species

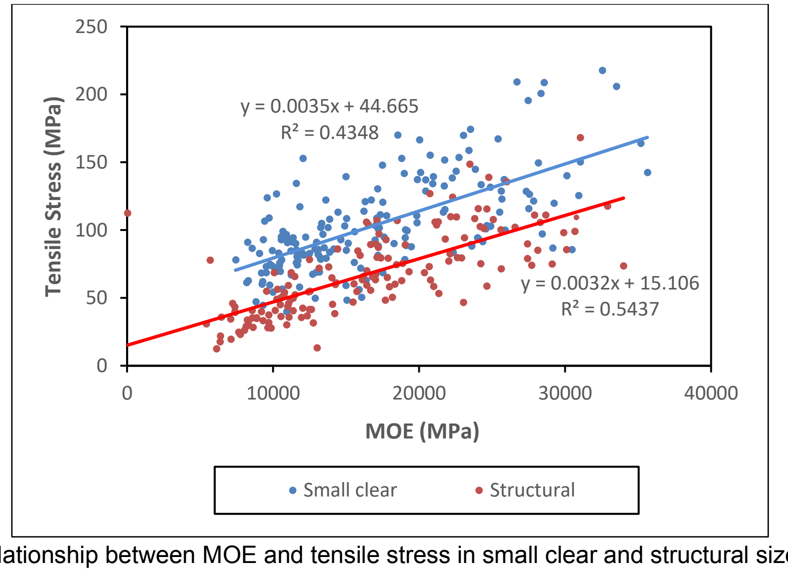

Figure 7. Relationship between MOE and tensile stress in small clear and structural size specimens among three species.

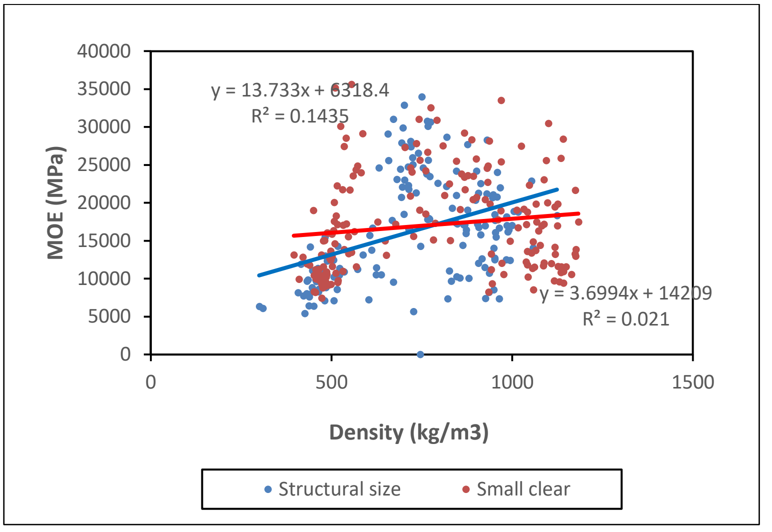

Figure 8. Relationship between MOE and density in small clear and structural size specimens among three species.

CONCLUSION

From this study, the tensile strength properties of Kempas (*Koompassia Malaccensis spp.*) from strength group (SG) 2, Keruing (*Dipterocarpus spp.*) from SG 5 and Light Red Meranti (*Shorea Siamensis spp.*) from SG 6 were investigated. It can be concluded that:

1. Tensile strength properties (tensile strength and MOE) for Kempas are higher than Keruing and higher than Light Red Meranti for both small clear specimens and large size specimens.
2. The tensile stress of small clear specimens is relatively higher that in structural size. However, there are no significant differences in MOE between small clear and structural size of the species.
3. There is a weak correlation between MOE and density of both specimen's size.
4. Strength ratio is different among the species. Thus, poor relationship to predicting structural size tensile strength from small clear specimens.

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COMPRESSIVE STRENGTH PROPERTIES OF STRUCTURAL SIZE MALAYSIAN TROPICAL HARDWOOD TIMBER

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Abstract
Two different species of Malaysian hardwood tropical timber namely Keruing (KRG) and Light Red Meranti (LRM) in structural size were used to determine compressive strength properties parallel to the grain. The experimental work was carried out according to EN 408:2012 for structural size specimen consists of 100 samples for each species that were collected from four different regions. Then, the 5th percentile of characteristic strength for both species was determined in accordance with EN 384:2016. It is important to determine the compressive strength using structural size specimens as the data available for Malaysian hardwood tropical timber is limited. Currently, structural design of timber structure in Malaysia is still based on permissible stress design method which the strength data for Malaysian tropical timber currently used is derived from small clear specimens. Therefore, there is a need for established strength data using structural size specimens in order to have more reliable data.

Keywords: *Compressive Strength; Characteristic Strength; Structural Size; Tropical Hardwood*

INTRODUCTION

Keruing (*Dipterocarpus* spp.) is occurring gregariously throughout South East Asia, Burma, India, The Andamans, Sri Lanka, Thailand, South Vietnam, Cambodia and the Philippines. Classified as medium hardwood with a density range from 690-945 kg/m^3 air dry in Peninsular Malaysia, and with a density of 595-865 kg/m^3 air dry for the species found in Sabah (Choo et al., 2001), KRG is suitable for heavy construction, posts, beams, joists, rafters, ship and boat building. If treated, it is suitable to be used as railway sleepers, harbour works, bridges, power-line poles and telegraph poles. Light Red Meranti (*Shorea* spp.) which fall under the category of light hardwood timber generally can be found in peat swamp forest (Choo et al., 2001; Choo at el., 1998). KRG is grouped in strength group SG5 according to Malaysia Standard MS544: Part 2 (2001), while Light Red Meranti (LRM) is in strength group SG6 which in timber industry, it is suitable for uses of joinery, light-duty flooring, panelling and interior partition (Choo et al., 1998).

Traditionally, most timber codes that were used throughout the world until the 1980s were in an allowable stress format where the strength data of timber were derived from properties of clear wood. As the world rapidly changes and due to the development of knowledge, the introduction of the limit states code principles of Eurocode 5 presents opportunities for engineers to optimize the design of timber structures and get the best out of the material. Therefore, in order to permit the European Conformity (CE) marking, all sawn wood for structural timber requires data for mechanical and physical properties based on full size specimen since strength data from small clear specimen still imposed inaccuracies as it does not represent the actual strength of the structural size of timber components.

It is difficult to represent the actual superior strength in structural member of timber if it is only based on small clear specimen because of natural growth defects such as knots, slope of grain, oblique fibre, zone of compression and many others that will usually reduce the strength significantly when the timber is sawn. The need for precise design criteria with accurate strength properties of structural timber is important to produce an economical and effective design as well as full utilization of timber.

The structural strength of wood is a measure of its ability to resist outside forces, such as compression, tension and shear. The density of wood is a reliable indicator of many of its structural and mechanical properties. There is a particularly strong relationship between density and compressive strength, bending strength and hardness, and a fairly reliable relationship between density and stiffness. This paper investigates the compressive strength properties of timber in structural size specimen from two selected Malaysian tropical timbers.

MATERIALS AND METHOD

Materials

KRG and LRM were chosen for this study. Full size specimens parallel to the grain were collected from four different regions of Malaysia, namely A1, A2, A3 and A4 represent Kelantan, Pahang, Johor and Sarawak respectively. The timber was cut into specified size according to the EN 408: 2012 (2012) for structural size as presented in Table 1. Then, the samples were undergone air dried process and moisture content was kept under 19% (dry condition). All of the specimens were visually graded and fulfilled the HS grade according to the BS 5756: 2011(2011). Prior to testing, the specimens were conditioned under room temperature in the laboratory.

Table 1. Number of tested samples

Species	Size	A1	A2	A3	A4
Keruing (KRG)	100x150x600	30	25	15	30
Light Red Meranti (LRM)	100x150x600	30	25	15	30

Method

The experimental work was carried out according to EN 408:2012 (2012) for structural size specimen. Preliminary test was conducted for structural size specimen in order to determine the loading head movement so that the maximum load, F_{max} is reached within 300s ± 120s. For KRG, the loading head speed used was 0.022 mm/s while for LRM was 0.02 mm/s and tested using Universal Testing Machine 2500 kN. Two Linear Variable Displacement Transducers (LVDT) were placed at the both sides of larger cross section to measure the displacement. The properties of structural size timber were determined according to the following procedures:

a) Determination of modulus of elasticity (MOE) compression parallel to the grain according to the clause 14 in EN 408: 2012.
b) Determination of compression strength parallel to the grain according to the clause 15 in EN 408:2012.
c) Determination of density by dividing the weight of the volume of the test piece.

d) Determination of a moisture content (MC) using the oven dry method according to the EN 13183-1: 2002. Samples used for moisture content were defect-free, i.e. knot and resin pocket. The specimens were weighed before dried in the oven at temperature of 103±2°C until the difference in mass between two successive weighing is less than 0.1%.

Then, the series of test results for structural size obtained from the experimental work were analyzed according to the EN 384:2016 in order to determine the characteristic value. The data were adjusted piece by piece with appropriate adjustment factor to achieve standard reference condition. Compression parallel to the grain is much affected by moisture content compared to the bending and tension. Hence, the test values for compression parallel to the grain, modulus of elasticity parallel to the grain and density, which not tested on reference moisture content (12%) and have a moisture content between 8% to 18% were adjusted to achieve the specific moisture content of 12% using Equations (1), (2) and (3) (EN 384, 2016). Then, the characteristic strength value of 5th percentile was evaluated using the corrected values at 12% MC according to the standard requirements.

$$fc_{,0} = f_{c,0}\ (u)\ (1+0.03\ (u-u_{ref})) \tag{1}$$
$$E_0 = E_0\ (u)\ (1+0.01\ (u-u_{ref})) \tag{2}$$
$$\rho = \rho\ (u)\ (1-0.005\ (u-u_{ref})) \tag{3}$$

Where, u is the moisture content at testing ($8\% \leq u \leq 18\%$) and u_{ref} is the reference MC, normally 12%.

RESULTS

Failure Characteristic

Figure 1a and Figure 1b shows some common failure mode found for Light Red Meranti and KRG respectively. For LRM, 69% of specimens failed in crushing and 20% of specimens failed in shearing. Other specimens were failed in wedge splitting and crushing with splitting. According to Bodig and Jayne (1982), specimens that fail in crushing will give higher compressive strength compared to the specimens fail in shear. Most of KRG specimens failed in shear and 40% failed in crushing. It was also seen that the density of the timber gives major influence on failure characteristic where the less dense timber has larger cell structure and thinner cell wall compared to the high-density timber (Ahmad et al., 2010).

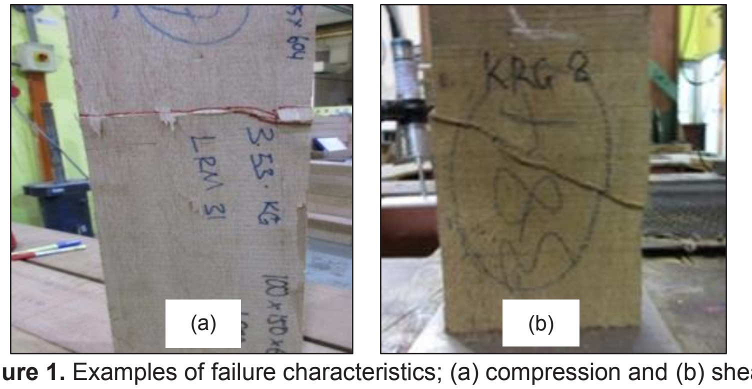

Figure 1. Examples of failure characteristics; (a) compression and (b) shear

Generally, higher density timber tends to fail in shear as compared to the less dense timber as KRG is harder than LRM. Surface of KRG is resinous and sticky when handled and there was resin coming out during testing as shown in Figure 2a. Figure 2b shows the presence of knot in structural size specimens of KRG and this defect is allowed with the appropriate percentages as given in BS 5756: 2011. In compression strength testing, it is important to have a proper cutting end as it will affect the failure mode of specimen.

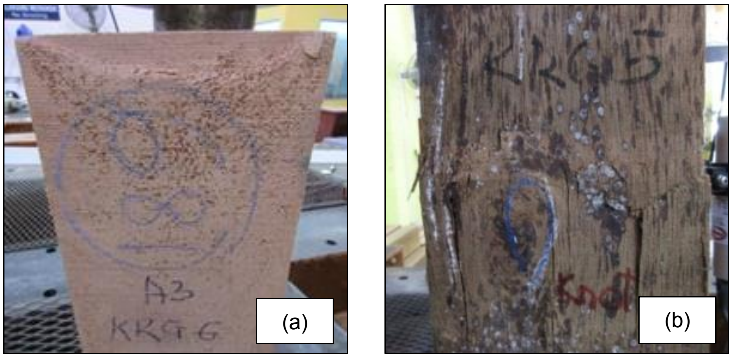

Figure 2. Examples of failure characteristics for KRG, (a) resinous surface and (b) presence of knot

Compressive Strength Properties

Figure 3 shows a typical graph of the stress - strain relationship for KRG and LRM. The graphs show similar behaviour where the graphs are approximately linear until reaches the highest point and then the load decreases when the specimens start to fail. The slope of the graph is representing the MOE of species which KRG shows a steeper slope, thus higher MOE compared to the LRM. This slope represents the stiffness of material which associated with the ability of a material to resist deformation under load where the steeper the slope, the stiffer material is. Therefore, from the graph it shows KRG is stiffer than LRM and has higher compressive stress. There are three factors that govern compressive strength parallel to the grain, which is homogeneity of material, the even distribution of load and straightness of the column. It is believed that homogeneity of material is responsible to affect the compressive strength parallel to the grain.

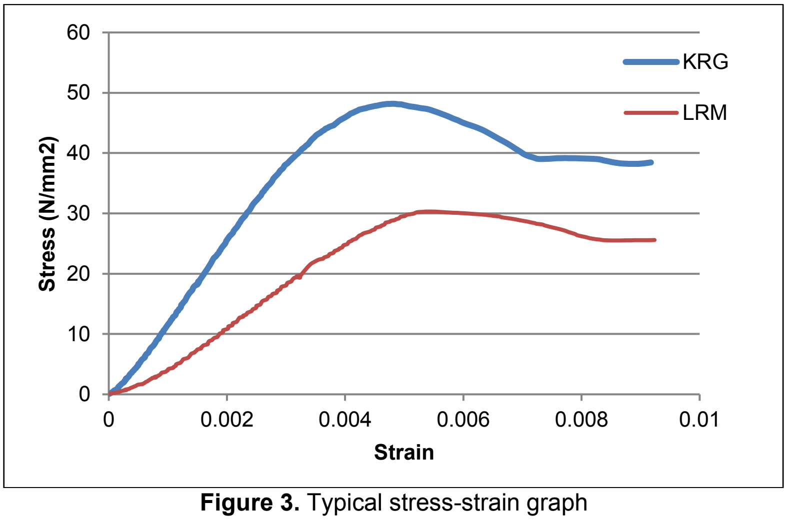

Figure 3. Typical stress-strain graph

Table 2 gives an overview of the test results for KRG and LRM. The mean compressive strength of KRG is 47.89 ± 8.2 with the minimum value and maximum value are 30.88 N/mm^2 and 71.55 N/mm^2 respectively. Whereas LRM has mean compressive strength 32.9 ± 5.6 with minimum value is 19.48 N/mm^2 and maximum value is 48.21 N/mm^2. The Grade stresses for each species were computed from this result based on 1-percentile in order to compare with the small grade stresses in MS 544: Part 2 using equation (4) below. It was also assumed that the compressive stress distribution is normal.

$$T_{(1\%)} = \frac{T_{mean} - 2.33\ (\sigma_{N-1})}{Factor\ of\ safety} \quad (4)$$

Where, T= the average strength means, 2.33 = 1st percentile coefficient, (σ_{N-1}) = Standard deviation and Factor of Safety = 1.5.

The present result calculated from 1-percentile grade stress from this experiment shows that the value is higher than the grade stress in MS 544: Part 2 (2001) and MS 544: Part 3 (2001). This result is in agreement with previous researches (Ahmad et al., 2010; Hassan et al., 2004) also found that the value of grade stress compressive strength and tensile strength for structural size is higher than the grade stress in MS 544: Part 2. It is shown that the grade stress value in MS 544: Part 2 could be over-designed when used in structural design. Therefore, it is important to have value of compressive strength based on structural size timber in order to produce precise and economical design as the value of grade stress based on small clear specimen will lead to bigger section and higher cost.

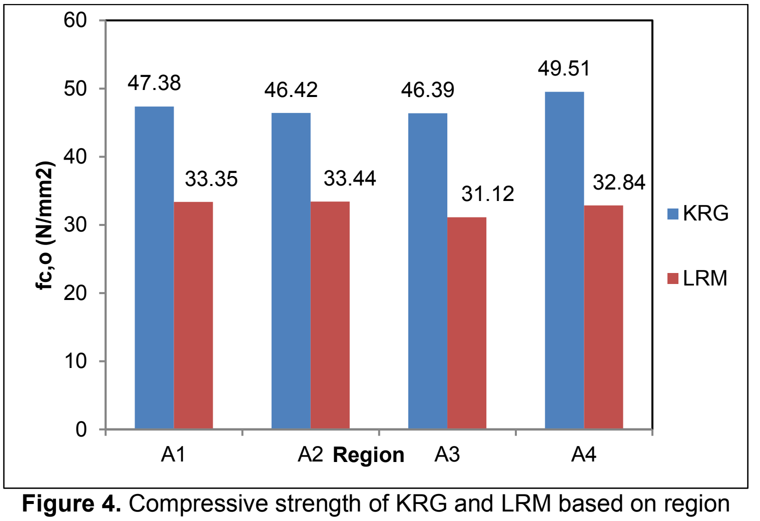

Figure 4. Compressive strength of KRG and LRM based on region

Figure 4 shows the comparison of compressive strength for KRG and LRM based on region of samples collected. The result shows that region A4 and A2 give higher compressive strength for KRG and LRM respectively. Generally, the value of compressive strength for every region does not differ too much from one another.

Table 2. Compression properties of LRM and KRG

Compressive strength								
Species	**Compressive strength (N/mm^2)**			**1-percentile Grade stresses ^{*1} (N/mm^2)**	**Select Grade stresses ^{*2} (N/mm^2)**		**MOE (N/mm^2)**	**ρ (kg/m^3)**
	Mean	Maximum	Minimum		**MS 544: Part 2**	**MS 544: Part 3**		
KRG	47.89 ± 8.2	71.55	30.88	19.2	15.4	16.0	11300	870
LRM	32.9 ± 5.6	48.21	19.48	13.3	11.4	9.9	7600	490

Analysis of Distribution of Compressive Strength

In order to express the data in characteristic strength, the 5th percentile compressive strength value using parametric method was calculated assuming the data is in normal distribution. Figure 5 shows the cumulative distribution function of compressive strength for each species. These graphs are in the S-curve, which indicated that the samples are normally distributed. From the strength distribution graph, it shows clearly that compressive strength of LRM is lower than compressive strength of KRG.

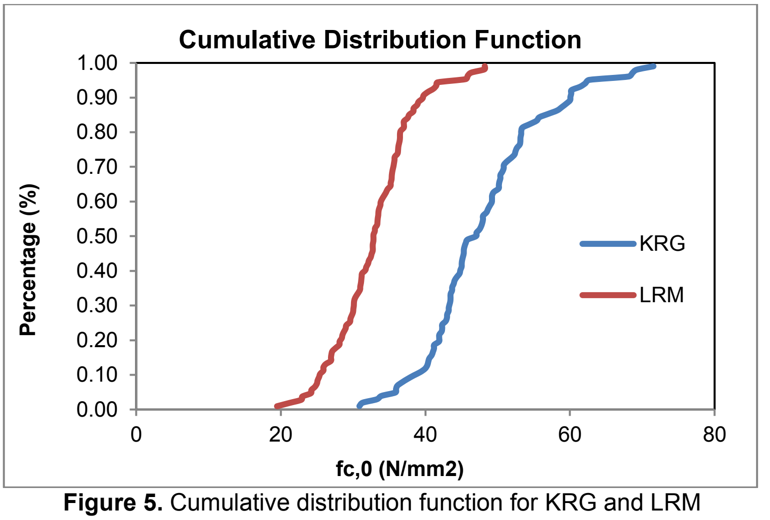

Figure 5. Cumulative distribution function for KRG and LRM

Table 3 shows the characteristic value of compressive strength for KRG and LRM. The values are the mean of all specimens from region A1, A2, A3, and A4 after moisture content correction has been made. From the characteristic value, KRG have higher value in terms of characteristic compressive strength, MOE and density compared to the LRM.

Table 3. Characteristic value of compressive strength properties

Species	n	Characteristic value of compressive strength (N/mm^2)	Characteristic MOE (N/mm^2)	Characteristic density, ρk (kg/m^3)
KRG	100	31.23	11723	670
LRM	100	21.42	7433	356

Regression Analysis between Compressive Strength and Density

Density of timber has been a focus in most research and has been emphasized as an important factor in determination of timber strength quality (Kamak et al., 2014; Yahya and Ismaili, 2015). One of the reasons is the density of timber is a good indicator to determine timber strength. However, in recent years the emphasization of stiffness as a better indicator of wood strength has been increased as it characteristic governs the design of timber structures. Yet, the relationship between stiffness and density shows a poor indicator (Teracy et al., 2001). Figure 6 shows the correlation between compressive strength and air-dry density for KRG and LRM. Moisture content of samples is 12%. The two measurements were linearly correlated to a degree of R^2 = 0.10 for KRG and R^2 = 0.53 for LRM. KRG shows weak relationship of compressive strength and density compared to the LRM where the compressive strength does not increase with the increasing of density.

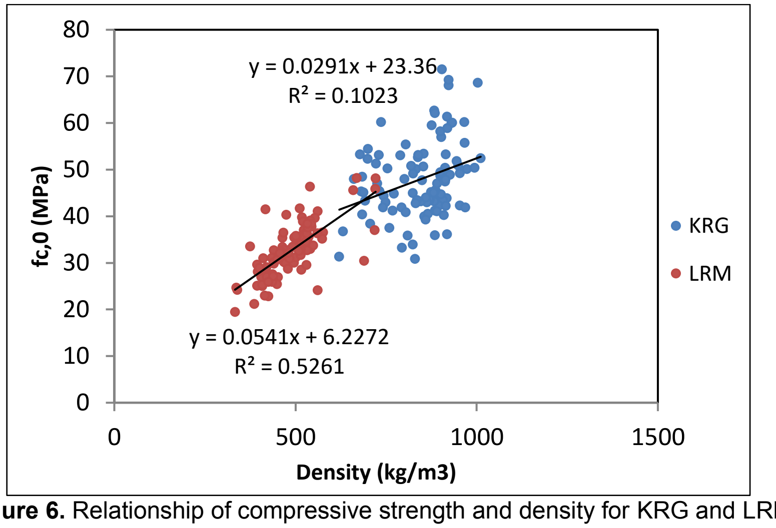

Figure 6. Relationship of compressive strength and density for KRG and LRM

Hassan et al. (2004) found that for compressive strength parallel to the grain, the compressive strength does not increase with an increase of density in contradicts with compressive strength perpendicular to the grain that the strength will increase with increasing density. Research on density by Senft and Bendtsen, (1985); Cown, (1992); Walker and Butterfield, (1996) as mentioned in Teracy et al. (2001) suggested a weak correlation between density in mature wood and density in juvenile wood. The density of timber increase from pith to bark thus increased in modulus of rupture (MOR) and modulus of elasticity (MOE). It has superior mechanical properties such as stiffness and strength than juvenile wood (Missanjo and Matsumua, 2016). Increasing in wood density from pith to bark is resulted from the age of the cambium (Akachuku, 1984). According to Brazier (1986), though the density of the timber gives some significance in timber strength, but it is not important as some other factors which lowered performance of timber in use. Density of wood is governed by the thickness of the fibre cell wall which higher wood density indicates a greater amount of cell wall material. Generally, latewood tissue has higher density, which comprised of thick wall fibres with small diameter cell and small lumen whereas early wood is softer and lighter in weight with thin wall fibres (Teracy et al., 2001). According to Wiedenhoeft (2010), species with thin-walled fibres have low density compared to the species with thick-walled fibres.

CONCLUSION

The compressive strength properties of KRG (*Dipterocarpus* spp.) and LRM (*Shor*ea spp.) from SG 5 and SG 6 according to MS 544: Part 2 was investigated. The following conclusions were derived:

1. KRG exhibit higher compressive strength and MOE compared to the LRM due to the fact that KRG govern higher density.
2. The grade stresses of structural size timber are relatively higher than the grade stresses based on small clear specimen in MS 544: Part 2 and MS 544: Part 3.
3. The value of compressive strength based on different region does not differ too much from one another.
4. Characteristic value of compressive strength properties of KRG is higher than value of LRM.
5. There are weak correlations between compressive strength and density for KRG as compressive strength parallel to the grain does not increase with the increase in density.

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EFFECT DIFFERENT TYPE OF FINE AGGREGATE TO HEAT OF HYDRATION IN CEMENT MORTAR

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Abstract
Heat of hydration is a heat generated from the reaction of Calcium oxide (CaO) in cement with water during the hydration process. This paper presents on the effect of fine aggregate to heat of hydration value in cement mortar. A study was carried out by comparing the heat of hydration of cement paste without aggregate (M0) with cement mortar containing river sand (M1), quarry dust (M2) and kenaf core (M3). The kinetics of hydration of mixes up to 72 hours were studied using isothermal heat flow calorimeter and ToniCAL apparatus. This study investigates the effect of different type of fine aggregate on the rate of heat evolution during hydration, and the heat of hydration. Results show that the inclusion of aggregate has reduce a heat of hydration of cement. It is also found that the utilization of kenaf core as fine aggregate in cement mortar mixes has improved the heat evolution of cement mortar as compare to cement mortar which contain river sand and quarry dust.

Keywords: *Heat Hydration; Fine Aggregate; Sand; Quarry Dust; Kenaf Core*

INTRODUCTION

Heat of hydration is a heat release by cement powder as a reaction with water during the hydration process. The hydration of cementitious materials in concrete reported to be exothermic while thermal conductivity of concrete is relatively low, but cement hydration can result in a large temperature rise (David et al., 2008). The quantity of heat generated by cement in the concrete hardening process is depending on several factors such as chemical composition of cement and concrete matrix, quantity of cementitious and concrete materials, type of cement used, water quality and type of aggregates. Many studies were carried out to determine the effect of cementitious material to the heat of hydration (David et al., 2008; Mostafa and Brown, 2005; Rashad, 2016; Winter, 1997; Ramli, 2008; Don, 2009). However, there are very few studies carried out to determine on the effect of aggregate type to the quantity of hydration heat of the overall concrete matrix. The aim of the utilization of by-products is to give the concrete new properties suitable for certain purposes (Mostafa and Brown, 2005).

Pure Portland Cement (PC) is slowly decreasing in the favour of substituted cements and composite cements. David et al. (2008) studied on the heat of hydration produce by Ordinary Portland Cement (OPC) and compare with two (2) different blended cement containing metakaolin (MK) and fly ash (FA). From their studies, it was reported that OPC has the highest value of heat during the hydration process as a result of the higher CaO content. Blended cement containing MK and FA produced lower heat due to lack of CaO content as MK and FA used as cement replacement material of OPC and their two materials has low CaO content. Mostofa and Brown (2005) also found almost a similar finding in their study to determine the heat of hydration at different high pozzolanic blended cement.

Reuse of by-products as a partial or full replacement of natural fine aggregate in construction activities not only reduces the demand for extraction of natural raw materials, but also saves landfill space and reduce the consumption of natural resources. Although several researches have been conducted to determine the potential of natural resources either by-product materials or waste materials to be used as aggregate replacement material, the acceptance of this product is still very limited.

Milon et al. (2012) studied on the effect of aggregate types on thermal properties of concrete concluded that thermal diffusivity of concrete is increasing with the increasing density of tested concrete that directly proportionate with density of aggregate used. Tomas and Ganiron (2014) studied the potential of sawdust as fine aggregate in a concrete report that sawdust particles within the concrete that pre-absorbed water during mixing helped hydrate the centre part causing it to reach early high compressive strength at such early time period.

The kenaf core is by-product from kenaf processing during the separation process to produce kenaf fibre. Very little research was carried out to utilize kenaf core as aggregate replacement material in a concrete matrix. The effects of kenaf core as aggregate material to concrete properties were also not discovered. Meanwhile, quarry dust is by-product of the crushing process during quarrying activities and considered as quarry waste that create a dumping problem to aggregate industries. The potential of this product was very limited which discovered by research community, especially in determining the effect of quarry dust to concrete heat of hydration.

This paper present on the effect of three different types of aggregate such as river sand, quarry dust and kenaf core which used in cement mortar to the quantity of heat generated during the hydration process. Knowledge of the heat-producing properties of different cementitious materials is required in order to choose a suitable formulation in this study for green and eco-friendly construction product.

RESEARCH METHODOLOGY

This study was carried out through laboratory work at Faculty of Civil Engineering, Universiti Teknologi Mara, Shah Alam, Selangor. Three (3) mixes labelled as M1, M2 and M3 of cement mortar containing different type of fine aggregate were prepared and M0 is the control sample. This study focuses on their heat evolution and heat of hydration during the hydration process. The heat evolution recorded in (dQ/dt in J/gh) and heat of hydration recorded in (Q(t) in J/g) were then compared with heat evolution of cement paste (M0). A water/cement ratio of 0.6 was used to prepare all the samples. Testing on heat evolution was carried out using ToniCal Trio (Toni Technick) apparatus as per describe in ASTM C186-15a. Table 1 presents the mixes and proportions of materials used in this investigation.

Table 1. Mixes proportion of samples

Mixes	Proportion	Cement (g)	Sand (g)	Quarry Dust (g)	Kenaf Core (g)
M0	Cement	5.19	-	-	-
M1	Cement + Sand	5.19	31.14	-	-
M2	Cement + Quarry Dust	5.19	-	31.14	-
M3	Cement + Kenaf Core	5.19	-	-	2.4

Materials

OPC refers to type (CEM 1 42, R) was used as cement in this study. The cement used is confirmed to satisfy BS EN 197-1:2000 requirement. River sand, quarry dust and clipping Kenaf Core was used in this study was classified as fine aggregate. River sand refers to sand that satisfied the standard requirement of fine aggregate for concreting as stated in ASTM C33/C33M-16e1 supplied by local supplier in Selangor area. Quarry dust is a by-product of granite quarry activity. It was collected from the Rawang granite quarry and stored in airtight, contained under the sheltered area in Concrete Laboratory, Faculty of Civil Engineering, Universiti Teknologi MARA, Shah Alam, Selangor. Kenaf core was collected from Kenaf Fibre Separation Plant, Tebu Hitam, Rompin, Pahang. Kenaf core was putting in shredded machine to a size of 3mm ± 2mm. After shredded, it was store in air thigh container under sheltered area to keep in good condition. Table 2 presents the properties of cement used and Table 3 presents aggregate properties used in this investigation.

Table 2. Properties of Cement

Properties	Value
Compressive Strength at 28 days (MPa)	29MPa
Standard Consistency (%)	28%
Setting Time (mm)	Initial =110min Final =235min
Fineness (%) of No 200 Sieve Residue	2.0%

Table 3. Properties of aggregate

Properties	River sand	Quarry dust	Kenaf core
Maximum size (mm)	5	5	5
Moisture Content (%)	3-5	3-5	3-6
Unit weight (kg/m^3)	1600	1800	180
F.M. Value	2.85	2	1.5
Absorption Capacity	2.4	3	120

Thermal Measurements

The hydration test was conducted using ToniCal Trio (Toni Technick). This equipment allocated in Non- destructive Testing (NDT) Laboratory, Faculty of Civil Engineering UiTM Shah Alam. Figure 1 presents the schematic diagram of the apparatus for testing hydration process. The apparatus was computer controlled isothermal heat flow calorimeter with three measuring cells that remarks as Cell 1, Cell 2 and Cell 3. The temperature was set at 27^{o}C to maintain the equilibrium of three cells.

For this study, the first stage peak of heat evolution did not identify since the inclusion of water to cement paste was carried out outside the apparatus. The second and third peaks of heat evolutions were analyzed in order to know the effect of cement with other materials. Before preparing samples for this testing, the process of mixture between the dry sample and water is done outside of the equipment.

The heat evolution versus time in the second stage was identified which is dormancy where all the materials in sample started to react and increase the strength. Time taken to complete for this testing is 72 hours. The rates of heat evolution (dQ/dt in J/gh) during hydration, and the heat of hydration, (Q(t) in J/g) were measured and recorded using a computer data acquisition system.

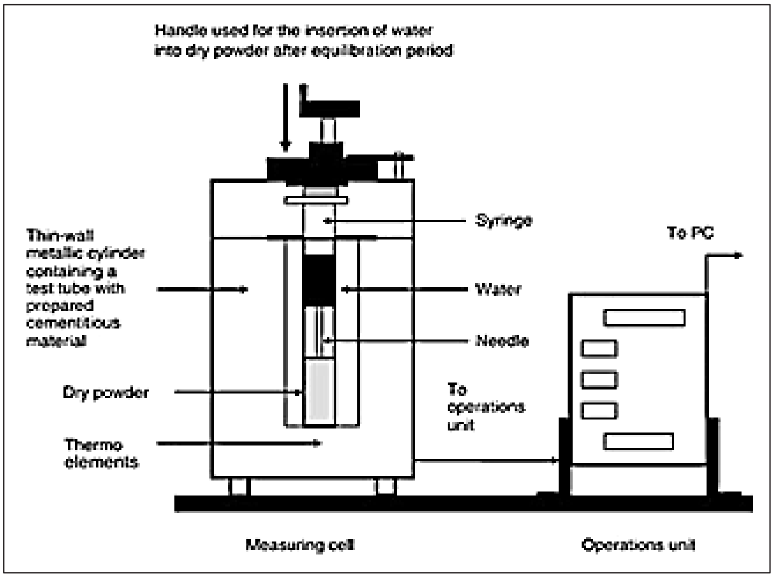

Figure 1. Schematic diagram of ToniCal Heat Flow Differential Calorimeter

Results and Discussion

Figure 2 and Figure 3 present the heat evolution (dQ/dt) and heat of hydration (Q/t) respectively. Figure 2 and Figure 3 represents a cement mortar contain three (3) different type of aggregates compared to sample contain cement paste only. From Figure 2 and Table 4, it found that heat evolution of cement mortar containing kenaf core is the highest as compared to other cement mortar at the first 20 minutes and sustained high at the first 2 hours. Maximum heat of evolution of mix M3 is 11.66 J/gh while for M0, M1 and M2 at the same hours is only around 5, 1 and 0.5 J/gh, respectively. The maximum heat evolution recorded along the experiment conducted is 8.74J/gh for M0, 1.12 J/gh for M1, 1.17J/gh for M2 and 11.66J/gh for M3.

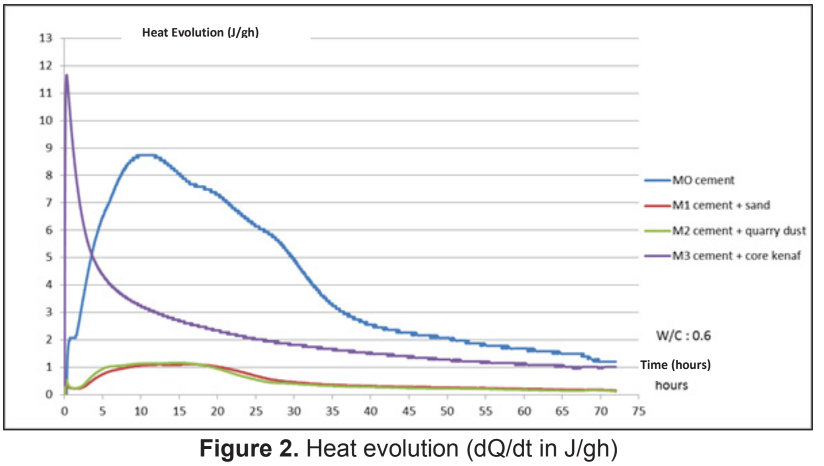

Figure 2. Heat evolution (dQ/dt in J/gh)

Figure 3 and Table 4 present the cumulative heat of hydration value of all samples tested in this investigation. From Figure 4, it is revealed that sample without aggregate (M0) produce the highest cumulative heat of hydration with a value of 290 J/g. Sample M3, which content kenaf core as aggregate recorded as the second highest value of heat hydrated during the hydration process in 72 hours. Heat of hydration value of sample M3 recorded as 155.29 J/g. This value follows by sample M1 and M2 where their cumulative value is 35.73 and 34.33 J/g, respectively.

From the result obtained, it is also portraits that the heat of hydration of mortar containing river sand and quarry dust is comparable but reduce for about 88% from M0 and the utilization of kenaf core as fine aggregate has reduced the heat of hydration of mortar for about 46% from M0. Sample M3 also found to have a heat of hydration four (4) times higher than sample M1 and M2.

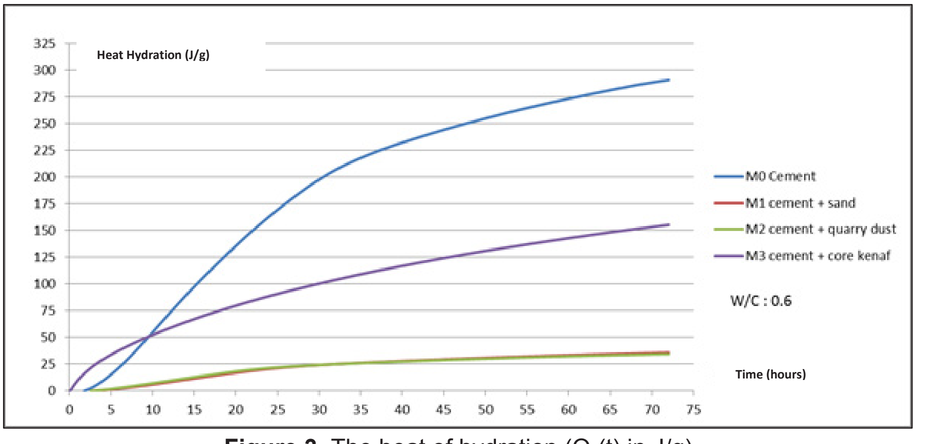

Figure 3. The heat of hydration (Q (t) in J/g)

Table 4. Value Heat of Hydration and Heat Evolution

Mixes	Maximize the rate of heat evolution (J/gh)	Maximizing the heat of hydration (J/g)
M0	8.74	290.68
M1	1.12	35.73
M2	1.17	34.33
M3	11.66	155.29

The rates of heat evolution (dQ/dt in J/gh) and the heat of hydration (Q(t) in J/g) for pure Portland Cement is very high due to the result of exothermic chemical reaction between cement and water, which raises the temperature of concrete. High volume content of CaO in OPC is responsible on the high heat of hydration produce (Don et al., 2009; PCA, 1997; Metha et al., 2006). The utilization of kenaf core found to increase the heat of hydration at an early period of time is may due to the water absorption properties of kenaf core as the same reason stated by finding on the effect of pre absorbed effectively by saw dust to the hydration of concrete (Milon et al., 2012).

Table 5 presents the time where the sample heat the maximum heat evolution value. It is found that M3 is having the highest heat for 20 minutes after sample M3 placed inside the calorimeter. M0 is having the maximum heat after 12 hours, followed by M2 and M1 at 15 and 16 hours, respectively. Analysis of the rates of heat evolution was focused on stage II to stage III showed that the rate of hydration increases fairly slowly (except for the sample containing kenaf core as aggregate) and the products of hydration will contact one another.

The heat evolution reaches a second peak typically at 12 hours until 17 hours. In stage III, the rate of hydration slows down over a long period.

Table 5. Time of sample having maximum heat of evolution

Mixes	Maximize the rate of heat evolution (J/gh)	On Time (hours)
M0	8.74	12.00
M1	1.12	16.46
M2	1.17	15.48
M3	11.66	00.20

Results obtained from this research also revealed that the heat of hydration may affected by the diffusivity rate of aggregate used. The high heat of the sample containing kenaf core (M3) may due to the diffusivity rate of kenaf core. Kenaf core reported to have a high porosity volume that directly reflects to diffusivity properties (Mohamad Jani Saad and Izran Kamal, 2012). Instead of that, the water absorption and water binding level of carnage core may also responsible for the high heat evolution produced where the cooling effect of water was restored by the pounding effect.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions were drawn based on the experimental result as follows:

1. The present of aggregate has reduced the heat evolution and heat of hydration value of cement paste.
2. As compared to three types of aggregate, kenaf core as aggregate found to produce the highest heat of hydration value followed by sand and quarry dust.
3. Kenaf core can be used as the replacement of aggregate in the construction industries.

As recommended for further studies, a study on the utilization of kenaf core as an aggregate and its effect to mechanical properties is essential. The study will be more exclusive to be supported with XRF and XRD analysis to represent the hydration product level. A study on porosity, water absorption, diffusivity and its correlation to density and hydration level is also recommended.

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EXPLORING PUBLIC CLIENT PROJECT MANAGER COMPETENCIES IN INDONESIA

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Abstract
Managing public infrastructure projects can be more difficult compared to handling private projects due to challenges that project managers encounter. A competent client project manager (CPM) is expected to overcome challenges and contribute to achieving public infrastructure project success. However, little research has been carried out examining public client project manager competency in the infrastructure sector in Indonesia. To address this gap, this paper aims to provide an understanding on current project manager competency, with a focus on client project managers in the Indonesian public infrastructure sector. The authors review project management articles pertinent to project manager competency published over the last two decades and complement them with non-project management journal articles related to the definition of competency. This paper clarifies the definition of CPM, reviews and identifies project manager competency, reviews client project manager duties and proposes a standard definition of client project manager. This paper also provides an overview of the challenges the public sector is currently facing, examines the CPM competency standard in Indonesia and presents an overview of the unit and officials involved in public projects in Indonesia. The review indicates that there are limited studies on CPM competency in public infrastructure projects, there is a lack of competency in CPM in Indonesia, and there is a lack of specific standards of CPM competency in Indonesia. This paper focuses on Indonesian public infrastructure sector only, however, it is expected that the findings of the review provide insights for further research on identifying CPM competency and enriches knowledge regarding public client project manager competency in other developing countries.

Keywords: *Competency, Client Project Manager, Construction, Public Project*

INTRODUCTION

It is widely known that competency plays an important role in work environments. Competency leads to high performance and enables organizations to achieve goals. Moreover, competency is a standard for recruitment and selection; a measure for performance assessment; a reference for conducting training and development needs; a basis for staff development processes and a springboard to integrate organizational strategies and goals (Fletcher, 1992; Soderquist et al., 2010; Sparrow, 1995).

The importance of competency has also been explored in the project management literature. Some studies indicated there is a significant correlation between project manager competency and project success, for example Geoghegan and Dulewicz (2008). The studies also found some people become project managers unintentionally or due to demand, which results in a lack of competency and affects the achievement of project success (read Vanessa et al., 2010; Pinto and Kharbanda, 1995). Lack of competency in project manager from client side (CPM) is also a major concern in Indonesia (KPK, 2014).

A report issued by Commission for the Eradication of Corruption in Indonesia, KPK (2014), identifies some competency weaknesses in individuals who handle public project, such as incapable to determine costs (owner estimate) and technical specifications of a project.

These are examples of the basic responsibilities of CPM in Indonesia. Even in some circumstances where a CPM may be supported by a technical team, the CPM is responsible for all decisions. In fact, due to CPM lack of project management technical capabilities, some were being taken advantage by contractors. Research conducted by Ng et al. (2002) found that CPM lack of technical knowledge can hinder their understanding and efficiency in making decisions pertinent to problem resolution processes. Therefore, CPM competency is paramount for project success.

Most project management literature considers the client as an external factor with a passive role in achieving project success (Koops et al., 2016) and there are few researchers paying attention to tailoring project management competency in public sector (Blixt and Kirytopoulos, 2017). As such, this paper is expected to enrich project management knowledge concerning project manager competency from the client side in public project contexts and examines the current condition of client project managers in Indonesia.

The structure of this review begins by elaborating some approaches to competency definition and some dimensions of project manager competency in current project management studies. Afterwards, the uniqueness and challenges of public project will be reviewed to gain an understanding of the conditions that client project managers encounter. Next, a definition of client project manager is proposed to clarify the reader in the definition of CPM that the authors use throughout this paper. Then, the units and officials involved in the public project in Indonesia are explained to provide insights into the current conditions of a client project manager and the problems that need to be addressed. Finally, the authors conclude with the limitation and an agenda for future research.

COMPETENCY

There are many definitions of competency. Le Deist and Winterton (2005) describe three dominant approaches to identify competency: (1) behavioural approach, (2) functional approach, and (3) multi-dimensional approach. The behavioural approach, a US model, emphasizes personal attributes with an aim for the worker to outperform others. This original US competency approach has evolved since the inclusion of attributes other than personal behaviour, such as knowledge and job skill.

Functional competence approach, a UK occupational model, refers to a person's ability to perform the required standard within the employment. The first part of the functional approach requires identifying the key roles, followed by breaking key roles down into units of competence and then into elements of competence. Next, each element of competence is transformed into the person's performance criteria for assessment and certification. In other words, work is a starting point to produce worker outcomes. As time passed, some researchers tried to develop their own model of competence by adding behaviour and other attributes, such as cognitive competence and ethical competence, instead of competence merely based on job performance alone (Le Deist and Winterton, 2005). The differences between the behavioural competency and functional competence approaches are summarized in Table 1.

Table 1. Comparison of behavioural approach and functional approach

	Behavioural Approach	Functional Approach	Sources
Name	competency, competencies	competence, competences	McMullan et al. (2003); Woodruffe (1993)
Other names	Generic approach; person-focused; generic competency; behavioural competency; worker-oriented	Performance approach; job-focused; technical competency; functional analysis; work-oriented	Dainty et al. (2005); Holmes et al. (1993); McMullan et al. (2003); Sparrow (1995)
Focus	Person	Job	McMullan et al. (2003)
Frames	Individual is the point of departure	Work is the point of departure	Garavan et al. (2001)
Purposes	Improve performance	Assessment and certification	Garavan et al. (2001)
Judgement	Grading system	No grading involved, pass/fail basis, competent/ incompetent	Rowe (1995)

Both behavioural and functional competence approaches are increasingly popular methods to define competency and competence. The functional approach has been heavily criticized because it wastes time to identify a long list of a job that rapidly changes in the current workplace situation (Russ-Eft, 1995). The same researcher also criticized the behavioural competence approach, based on the fact that an organization does not need "one kind of superstar" to achieve success. Another researcher argued that this approach results in general descriptions which means the attributes of competencies can appear in many different jobs (Sandberg, 2000).

Lastly, a multi-dimensional competency approach is a holistic model which considers many dimensions of competency instead of merely a behavioural or functional approach. Although there is no consensus on which dimensions make up competency, Dejoux in Le Deist and Winterton (2005) argue there are at least three competency dimensions commonly accepted in France: knowledge, skills and behaviours. Le Deist and Winterton (2005) conclude that the inclusion of knowledge and functional dimension in the US competency approach and behaviours and knowledges in UK competency approach indicate that a one-dimensional approach is insufficient to represent competency. As a result, the recognition of multi-dimensional approach has thrived. This research adopted the following definition of competency: *"Competency is a knowledge, skill, ability, or characteristic associated with high performance on a job, such as problem solving, analytical thinking, or leadership"* (Mirabile, 1997).

There is abundant research that discusses project manager's competency in project management literature. For instance, Hwang and Ng (2013) identified knowledge and skills required by project managers in managing green construction. Their results show that critical knowledge and skills required by green construction project managers are different to what is required of project managers in traditional construction. Their studies showed that three knowledge areas should be emphasized by project managers (in order): cost management, communication management, and schedule management. Empirical research conducted by Dogbegah et al. (2011) identified the competency required by project managers in Ghanaian construction. In their study, they categorised six competency knowledge areas, comprised of business and ethical, financial resource, risk and quality, project human resource and control, innovation and communication, and physical resources and procurement. Another researcher, Edum-Fotwe et al. (2000), examined the knowledge and skills required by construction project managers to perform roles that have been expanding due to changing business demand in the industry. Their results indicated that attributes of knowledge-skills which are categorized into primary lists to develop project manager's competency are acquired based on experience-skill, such as managerial skills, communication skills and technical skills. Meanwhile, the same researcher argued that the secondary list of knowledge-skills is comprised of attributes that are predominantly from academic programs, such as material procurement, forecasting techniques and quality control. Jabar et al. (2013) investigated the competency required by project managers to manage Industrialized Building System (IBS) project in Malaysia. Their result showed that construction managers in IBSs project require additional competency beyond conventional construction project, such as technical knowledge in installation process, material quality and selection of plant.

PUBLIC CONSTRUCTION SECTOR

Compared with the private project sector, public projects have their own unique nature. PMBOK Government Extension (2006) explains the uniqueness of public project as: (1) restricted by legal constraints (2) accountability demand, and (3) public fund utilisation. In regard to the first, while private projects are obliged to follow several regulations and laws, there are additional regulations and laws specific to public projects such as procurement law. In addition, each government body has its own authority, therefore, to proceed beyond their limit, they need to follow legal procedures. A project manager who works in the public sector should understand the regulations and laws associated with public projects to manage the project successfully.

Pertaining to second uniqueness, public projects must be accountable to project stakeholders and society. Moreover, since public projects utilise public funds, there is a designated mechanism to control expenditures and delivery of the project. This is because efficiency and effectiveness of utilizing of public funding are strongly emphasized in the public sector (PMI Government Extension, 2006).

A project manager should be aware of the challenges in managing a public project. These challenges include operating under media scrutiny, availability of pressure in effective use, dealing with conflicting goals, facing entrenched bureaucracies and administration, no correlation between performance and reward/ compensation systems, inability to select members of the project team, a workplace that is sometimes unfamiliar with result-oriented project management, and dealing with political appointees (Wirick, 2009).

In agreement with Wirick (2009), Blixt et al. (2017) conducted empirical research in the Australian public service to explore some challenges that project managers faced. Some challenges are an environment that is unfamiliar with project management, lack of clarity in objectives and success measures, process, decision-making that is slow due to the hierarchy, and dealing with various stakeholders.

These uniqueness and challenges of the public sector demand public project managers to have certain competencies. A study by Chong (2013) concluded that there are different competencies required by managers from either the private or public sector. The study assessed the competency of managers in the British private sector and the Singaporean public sector, and found that communication, integrity and organizational awareness are the top managerial competencies in the public sector. Meanwhile, business sense, achievement orientation, and tenacity are the primary competencies required by managers in the private sector. Research by Blixt et al. (2017) reveal that communication, accountability and business alignment are the three highest competencies required for delivery of public projects. However, these researches do not focus on infrastructure projects.

CLIENT PROJECT MANAGER

A project manager is a person who is leading a project to achieve the project's goals. A project manager plays important roles in achieving project success (Hwang and Ng, 2013). A project manager can be the representative of the client or contractor's side. A client project manager is responsible for "coordination of the entire design, contract administration, and construction effort"; whereas a contractor project manager is responsible for "all on-site activities" (Hashim et al., 2018; Hutcheson, 1984). Jawahar-Nesan et al. (1997) outline the main tasks of client representatives, which include (1) preparing and organizing projects, (2) developing client requirements, including feasibility studies, (3) managing procurement, (4) organizing teams, (5) coordinating design management, (6) monitoring safety management implemented by vendors, (7) monitoring performance, (8) communicating information, (9) motivating people, (10) coordinating projects, (11) documenting processes, and (12) evaluating projects.

A number of authors used different names in labelling project manager from the client side, for instance, "Intelligent public sector construction client" (Aritua et al., 2009; 2011), and "owner's/client's representative" (Jawahar-Nesan et al., 1997). The last studies pointed out that the usage of client's representative term has a general meaning because this term can be used for engineers, surveyors, project managers, and others who represent or are delegated by the owner.

Aritua et al. (2011) explained that an Intelligent public sector construction client is a person who works in the public sector, engages with the private sector, and monitors the project. Meanwhile, Jawahar-Nesan et al. (1997) described "the owner's representative could define as a professional body (can individual/ a firm, and/or in-house/ outside to the owner's organization) who has been delegated by the owner to advise and act on his/ her behalf as a distinct member of the project team". Even though there are various names to explain client project manager, it is unarguable that they receive a commission from the client and monitor the project to ensure client requirements are fulfilled. This research has adopted the following definition for client project manager:

> *Client project manager (CPM) is a person working in public sector who has been delegated by the client to act on his/her behalf to engage with other parties, implement the contract, and ensuring the client requirements is fulfilled.*

Although there is a wealth of research on project manager competency, only a handful of research pertinent to public sector project manager competences (Blixt and Kirytopoulos, 2017), especially client project manager competency in the public construction sector (Aritua et al., 2009).

CPM IN INDONESIA PUBLIC INFRASTRUCTURE

As a developing country, Indonesia is expected to be the fastest-growing construction market by 2020, followed by China, Malaysia and India, Myanmar and Singapore (AECOM, 2014). Based on the data issued by Ministry of Finance of Republic of Indonesia (2017), budget allocated for infrastructure has been increasing for the last five years. The Government of Indonesia plans to construct 15 airports, 65 harbours, 1448 kilometres of roads, 3258 kilometres of railways, and develop 193 kilometres toll roads, and several others infrastructure projects from 2015-2019 (Bappenas, 2014). These massive infrastructure developments need to be accompanied by competent client project managers in order the success of projects.

In the development of the public construction sector (including infrastructure), the Government of Indonesia has stipulated several parties that are involved in the procurement of public construction sector. Based on Presidential Regulation No. 54/2010 concerning the procurement of goods/ services which was amended several times most recently by Presidential Regulation No.04/2015, there are two ways to deliver a public project: 1) through external supplier or service providers, or 2) through in-house procurement. The procurement unit and project management officials responsible for delivering projects through external suppliers or service provider comprised of Budget Authority/ Proxy of Budget Authority (PA/KPA), Commitment-Making Officer (PPK/CPM), Procurement service unit (ULP)/ Procurement Official (PP) and Official receives the result of the work completed (PPHP). Meanwhile, the unit and project management officials involved in the in-house procurement comprised of Budget Authority/ Proxy of Budget Authority (PA/KPA), Commitment-Making Officer (PPK/CPM), Procurement service unit (ULP)/ Procurement Official (PP)/ Procurement Team and Official receives the result of the work completed (PPHP). Figure 1 depicts the parties involved in public procurement in Indonesia.

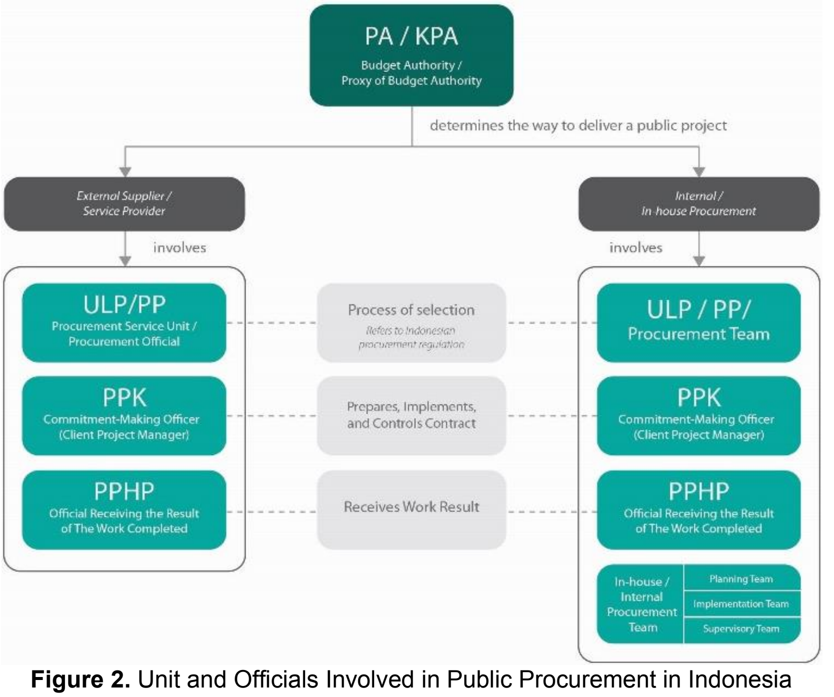

Figure 2. Unit and Officials Involved in Public Procurement in Indonesia

The Minister/ head of local government/ head of institution in Indonesia establishes the unit for procurement service, namely: Unit Layanan Pengadaaan/ Procurement service unit (ULP) whereas Budget Authority/ Proxy of Budget Authority (PA/KPA) in the Ministry/ institution/ local government assigns officials, namely: Pejabat Pengadaan/ Procurement official (PP), Pejabat Pembuat Komitmen/ Commitment-Making Officer (PPK), and Pejabat/ Panitia Penerima Hasil Pekerjaan/ Official receiving the result of work completed (PPHP).

The selection of external suppliers or service providers in Ministries/ Institutions/ Local Governments in Indonesia involves ULPs or PPs for which their authorities have been stipulated by Presidential Regulation No. 4/2015. The implementation of the contract will be monitored by PPK. The completion of the project will be examined by PPHP. If PPHP finds shortcomings/ defects in the works, PPHP through PPK asks the external supplier or service provider to do corrective action in accordance with the contract.

Even though there are four procurement parties required in any public construction project in Indonesia, this research will focus on PPK due to its level of importance and job complexities. According to Presidential Regulation No. 54/2010, the duties of PPKs in the planning stage are to determine technical specifications of the goods/services, owner estimate (OE) and draft contract. In the implementation stage, PPKs are authorized to control the works performed by the external suppliers or service providers, oversee the implementation of the work, and ask for periodic reports on the works performed by the external suppliers or service providers. In the handover phase, PPKs will accept the work completed by the external suppliers or service providers after all the work is completed in accordance with the provision of the contract. PPKs could defer the external suppliers or service providers' work performance instalment payment if the external suppliers or service providers fail/neglect to fulfil their contractual obligations. PPKs can also reschedule the task execution with an addendum to the contract with justifiable reasons.

Considering that PPK is an uncommon word in project management literature, we prefer to use another term to represent PPK. Hutcheson (1984) states that project managers can be representative of the client or contractor side. Moreover, project managers have to be responsible for project delivery to achieve project outcomes. The comparison of tasks of PPK and task of CPM is depicted in Table 2. Based on these similarities, PPK can be described as a client project manager due to activity as an owner delegation, acting on owner behalf, and being responsible for the project. Therefore, in this paper, the term of "Client Project Manager (CPM)" will be used as a broad term referring to PPK as a project manager representative of the client's side.

Table 2. Comparison of PPK and CPM duties

Stages	PPK	CPM (Client Project Manager)		
		Hutcheson (1984)	Jawahar-Nesan and Price (1997)	Hutchison (2009)
Preparation	▪ Devise Owner Cost Estimate, technical specification, and contract. ▪ May propose changes of procurement schedule or package work to budget user (PA). ▪ May establish a team that will provide technical	▪ Coordinate the entire design ▪ Coordinate the contract administration	▪ Prepare and organize the project ▪ Develop client's requirements including feasibility study	▪ Develop project strategy ▪ Create and monitor good project environment ▪ Set up a clear business case

Stages	PPK	CPM (Client Project Manager)		
		Hutcheson (1984)	Jawahar-Nesan and Price (1997)	Hutchison (2009)
	explanations to assist the implementation of the procurement working group (ULP) duties. ▪ Set the amount of advance payment to be paid to the external supplier or service provider.			
Procurement	▪ Issue Letter of Appointment of Goods/ Services (SPPBJ) ▪ Sign the contract		▪ Ensure the selection of external supplier or service provider obtained in the effective way ▪ Organize a joint management team	
Construction	▪ Monitor and control the execution of the contract ▪ Report the progress of work to budget user (PA/KPA) ▪ Document process	▪ Coordinate construction process	▪ Monitor safety management implemented by external supplier or service provider ▪ Monitor performance ▪ Communicate information ▪ Motivate people to work successfully ▪ Coordinate project ▪ Document process	▪ Monitor project progress at a macro level ▪ Prepare to implement corrective actions ▪ Communicate to the client organisation and project team ▪ Understand the needs of all parties ▪ Share knowledge on lessons learnt from other clients and projects
Handover	▪ May ask supplier or service provider to do corrective action in accordance with the contract.			

There is a national competency standard for parties employed in public procurement in Indonesia. This standard comprises four keys of competence and is divided into 29-unit competences. These four keys of competence are plan the project; select the provider of goods/ services, manage contract and in-house procurement of goods/ services; and manage logistics, performance and risk. However, there is no specific competency standard for CPM in infrastructure project. The national competency standards issued by the Ministry of Manpower of Republic Indonesia No.70/2016 apply to all officials and units that are involved in public projects. This might raise the question of “what are the competencies required by client project managers in managing public infrastructure?”, “what are the key competencies of CPM that should be emphasized to be improved?”. Understanding the key competencies required by CPM can assist in development and training to address their lack of competencies.

LIMITATION AND CONCLUSION

This paper has reviewed and provided further understanding on the definition and attributes pertinent to project manager's competency. This paper has also presented an overview of the units and officials involved in public projects and the challenges the public sector is currently facing. The lack of competency of client project managers, the lack of specific standards of competency for CPMs in Indonesia, few researches on client project manager competency in the public infrastructure sector, especially in Indonesia, highlights the necessity to further investigate this matter at hand. Empirical research is required to provide an in-depth understanding toward the problems and key competency required by client project manager in Indonesia. The result can serve as a reference to develop client project manager's competency in developing countries that have problems similar to Indonesia. This paper is a springboard for further empirical research that will be conducted in the near future.

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COMPARATIVE ANALYSIS OF THE MECHANICAL PROPERTIES OF CONCRETE BLOCK MASONRY USED IN CONSTRUCTIONS WITHIN ARGENTINA AND ECUADOR

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Abstract
A comparative analysis is presented between the mechanical properties of concrete blocks masonry that are used in the construction of houses in Argentina (city of Mendoza) and Ecuador (Chillos Valley). This allowed to evaluate the control applied in the elaboration of such materials. Therefore, the corresponding tests have been conducted to determine the absorption and compressive strength of the specimens. These tests are established in the current regulations of Argentina (IRAM-11561) and Ecuador (INEN 3066). In Ecuador, the study has been performed between 2014 and 2017 by means of a statistical analysis. Samples have been taken from 15 constructions in the Chillos Valley. Later in 2017, in the city of Mendoza the characteristics of the mechanical properties of 15 concrete blocks were determined. The yielded results demonstrated, that 100% of the blocks analyzed in Mendoza meet the requirements, while in Ecuador only 45% are accepted. With respect to the compression resistance, there is a great difference in the quality of the blocks between each studied country. Ecuador obtained only 2.7% acceptance in comparison with Argentina where it reached 72.73%. Regarding the economic cost, the block marketed in Ecuador have been cheaper by approximately 80%.

Keywords: *Absorption; Resistance to Compression; Density; Quality; Technical Control; Masonry*

INTRODUCTION

The mechanical properties and overall behaviour of concrete blocks masonry have been worldwide of fundamental importance for design and safety assessment purpose (Poon et al., 2002; Al-Jabri et al., 2005; Davidson et al., 2005; Hansen, 2014). Nonetheless, generally in a competitive market of developing countries, quality control and economic pressure may lead to a less adequate use of such construction material (Blalock and Gertler, 2004; Ngowi et al., 2005; Zhang et al., 2011). The lack of technical control in the construction is visible in some cities of Ecuador, this is directly reflected in the quality of the materials that the builders acquire for their works (Robalino et al., 2017). The present project arises from the classrooms, where hollow masonry blocks of concrete blocks are tested and it is visualized that they do not comply with the current regulations in Ecuador on quality of materials (Robalino et al., 2015). Reason for which it has been decided to conduct a new study that evaluates the mechanical characteristics of this type of input for the area of the Valley of the Chillos-Ecuador, finding unsatisfactory results in a large part of the tested samples of masonry.

The mechanical properties of construction materials allow us to compare them with standardized quality standards; which may vary according to the country or region to which they belong (Brandt, 2005; Rahal, 2007; Cachim, 2009). A concrete block is a prefabricated masonry, which is used in walls, have usually a prismatic shape, with normalized dimensions (Petroquímica Comodoro Rivadavia, 2006; Poon and Chan, 2006).

As part of the current comparative study, bibliographic we searched two countries within several Latin American countries, with priority to those that conform to the conditions of the study area. The city of Mendoza-Argentina as well as Ecuador has been on geological faults which causes telluric movements that destroy most buildings, so the masonry must be executed with compliance with the standards of resistance to compression, to safeguard life of human beings (Caiza et al., 2018; Navas et al., 2018; Toulkeridis et al., 2017). On the other hand, although Mendoza's climate differs as it is a semi-arid zone with precipitations, it has the peculiarity that it has aquifer deposits at a few depths (Kairis et al., 2014), giving a similar phreatic level to the Valley of the Chillos. This phenomenon is directly related to the pathological problems of humidity in masonry, which is measured with the mechanical property of percentage of absorption of the masonry (Lin et al., 2006; McGregor et al., 2014).

Argentina has the IRAM 11561 standard, corresponding to concrete blocks, which presents requirements and test methods similar to those of the Ecuadorian standard of quality INEN 3066, with which a direct comparison can be made between the two materials (Ecuadorian Normalization Service, 2014; 2016; Argentine Institute of Normalization and Certification, 1997a; 1997b).

STUDY AREAS

Chillos Valley, Ecuador

In Ecuador, the masonry of concrete blocks is the one that greater use has in the constructions, due to its affordable costs and easy acquisition (Schacher and Hart, 2018). The elaboration of this type of masonry has been performed in an artisanal and industrialized way, being the first the most predominant in use, reason why they do not present a rigorous quality control in the pieces that go out to the market. Additionally, their mechanical properties lack to be evaluated. Therefore, it is proposed to perform tests to evaluate the mechanical properties of hollow concrete blocks of buildings that are under construction in various sectors of the Valley of Chillos from 2014.

In the first step, a sample is established based on five types of hollow blocks of concrete according to their use and parameter presented in the INEN 639 and 643 regulations. Due to the fact that a revision to the standard has been conducted in 2016, such being replaced by INEN 3066, there is a change in the classification of the masonry (Table 1 and 2), which leads to a new calculation of the size of the sample (Equation 1).

$$n = \frac{Z^2\, p * q\, N}{e^2(N-1) + Z^2\, p * q} \qquad \text{(Equation 1)}$$

where, n, is the sample size; N, population or universe; Z, confidence level; p, probability in favour; q; probability against; and e, the sampling error.

For the current study, a population corresponding to 638 buildings has been considered, a value that is the sum of the construction permits issued by the Municipality of Rumiñahui in the period 2013-2015 (Robalino et al., 2015). The confidence level is taken to be 90%, with which the Z value is 1,645, this being the sample error of 10%, all being values with which a research is acceptable. The probabilities for and against are estimated 95% and 5%, respectively, since it has been observed that the construction use more frequently concrete blocks of handcrafted elaboration. Finally, the size of the sample is 13 constructions, which approximates 15 for the development of the project. On the basis of table 1, the masonry collected in different sectors in Ecuador is Class B.

Table 1. Concrete blocks according to their use. From Ecuadorian Normalization Service, 2016.

Class	Use
A	Structural masonry
B	Non-structural masonry
C	Glazing in slabs

Table 2. Concrete blocks according to density. By Ecuadorian Normalization Service, 2016.

Type	Block density (kg/m^3)
Light	< 1680
Medium	1680 – 2000
Normal	>2000

City of Mendoza, Argentina

In the city of Mendoza, the constructions are of small and medium height, there is masonry of concrete blocks and bricks. In general, a solidly built city with solid and heavy walls. The concrete blocks, in Mendoza, are similar in appearance to those of Ecuador, but their conformation and weight differ greatly. To manufacture them, there is an artisanal and industrialized form. The first is used only for specific projects, which are controlled by the municipality, who is in charge of coordinating the services of a specialized entity to evaluate the mechanical properties of the inputs produced by temporary blocks that are installed during the execution of such projects. On the other hand, who decides to build with this type of material independently must use the industrialized line, which is marketed by supermarket chains or warehouses of construction materials present in some sectors of the city and its surroundings.

For this reason, the sample with which the project has been developed, in September of 2017 has been determined according to the Argentine norm for tests of mechanical properties of concrete blocks of each variety and existing supplier in the market, which differ in resistance and use within the field of construction. The population of Mendoza acquires materials for construction in warehouses of materials located in the surroundings of the city. For the case of hollow concrete blocks, products coming from Córdova and Buenos Aires are commercialized.

MATERIALS AND METHODS

For the preparation of samples, tests and equipment used to obtain the mechanical properties of hollow concrete mamposums, the current regulations of Ecuador (NTE INEN 3066) and Argentina (IRAM 11561-2 / IRAM 11561-4) are used (Ecuadorian Normalization Service, 2014; 2016; Argentine Institute of Normalization and Certification, 1997a; 1997b).

Absorption

Absorption is one of the physical properties that are frequently tested on specimens of hollow concrete blocks, in order to determine the amount of water that materials can absorb when shaping the structures. They have a limit given by the regulations of each region so that these amounts of water do not generate pathological problems in the masonry. Tables 3 and 4 demonstrate the permissible limits for Absorption dictated by the INEN and IRAM Norms, pertaining to Ecuador and Argentina respectively.

Table 3. Maximum absorption of Class A blocks. Based on Ecuadorian Normalization Service, 2016.

Type	Density (kg/m^3)	Average maximum water absorption (kg/m^3)	Maximum water absorption per unit (kg/m^3)
Light	< 1680	288	320
Medium	1680 - 2000	240	272
Normal	>2000	208	240

Table 4. Absorption of water, for the average of 3 units according to density. Based on Argentine Institute of Normalization and Certification, 1997b.

Type	Density	Average maximum water absorption (kg/m^3)
Light	< 1700	290
Medium	1700 - 2000	240
Normal	>2000	210

The INEN 3066 standard stipulates only absorption limits for structural or load-bearing blocks, and only in kg/m^3, so it is fundamental to mention that at the beginning of the project it has been under INEN 639 and 643, where the limits of absorption has been generalized for all types of blocks and also a maximum absorption percentage of 15% has been specified. This value has been taken into account for the comparative analysis. Three samples of blocks have been tested for each construction in Ecuador and for each type of masonry in Argentina. In both cases were whole specimens and without defects each identified with Arabic numerals followed by an alphabetical number for the number of copies. The equipment that has been used according to the two regulations is the balance with accuracy of one gram and that allows additional the basket to measure submerged mass, in the centre of the dish and the oven or stove that has been able to be maintained between 100 and 115°C.

Test

The procedure in Ecuador and Argentine is similar, but they differ in the order of carrying out the processes in the laboratory. In Table 5, a comparison has been performed between the sequence and characteristics of the test method for each zone. Figures 1 and 2 illustrate photographs of the tests developed in Ecuador and Argentina.

Table 5. Procedure for absorption test in hollow concrete blocks

ECUADOR INEN 639 y 3066		**ARGENTINA IRAM 11561-4**	
Step 1: Saturation	Immerse the samples in water at a temperature between 16 and 27 ° C, during the period of 24 to 28 hours. Determine the submerged mass of the units.	**Step 1: Drying**	Dry the blocks at a temperature of 105 ± 5 ° C, for the time necessary so that, when weighing, the mass does not vary more than 0.25%.
	Remove them from the water for 60 ± 5 seconds, dry the sample superficially with a cloth and determine its mass. Repeat this process in the next 24 hours, until the mass more than 0.2% in consecutive weights.		Allow the environment to cool
Step 2: Drying	Dry in the oven at a temperature between 100 to 115 ° C for 24 hours and determine its mass until the difference in consecutive weightings is less than 0.2%	**Step 2: Saturation**	Immerse the samples in water at a temperature of 20 ± 2 ° C in water for 24 to 96 hours. Determine the submerged weight of the units, until mass is maintained.
			Take them out of the water and let them drain freely, dry the sample with a cloth and determine its mass. Repeat this process in the next 24 hours, until the mass more than 0.2% in consecutive weights.
			Place the specimen in a basket and weigh it submerged in the water

Figure 1. Absorption Test: Saturation – Drying in Ecuador

Figure 2. Absorption test: Drying – Saturation in Argentina

Resistance to Compression

Resistance to compression is the main mechanical property that has been analysed in the masonry units because they resist compressive forces during the useful life of the building. The compressive strength represents the maximum stress that concrete blocks can withstand, therefore, they need to comply with the requirements of minimum compressive strength for blocks, listed in Table 6, 7 and 8, which are found in the current mentioned regulations.

Table 6. Compression resistance, in non-supporting blocks- Ecuador. *1 MPa=10,2 kg/cm^2. Based on Ecuadorian Normalization Service, 2014

Description	Compressive strength (MPa)*
Average of 3 blocks	4,0
Individual block	3,5

Table 7. Minimum net compressive strength in concrete blocks-Ecuador. *1 MPa=10,2 kg/cm^2. Based on Ecuadorian Normalization Service, 2014

Description	Minimum net strength to simple compression (MPa)*		
	Class A	Class B	Class C
Average of 3 blocks	13,8	4,0	1,7
Per block	12,4	3,5	1,4

Table 8. Compressive strength -Argentina. *1 MPa=10,2 kg/cm^2. Based on Argentine Institute of Normalization and Certification, 1997b.

Description	Compressive strength (MPa)*	
	Net Section	Gross section
Average of 3 blocks	4,0	2,5
Individual unit	3,5	2,0

Sample Preparation

At least 3 block units were tested. The Ecuadorian standard indicates that the blocks should be stored at a temperature of 24°C ± 8°C and a Relative Humidity <80% for at least 48 hours, unlike the Argentine norm which indicates that the blocks should be at a temperature of 20ºC ± 2ºC and a Relative Humidity ≤ 50% and minimum two hours to the ambient air. We took dimensions of width, length and height of each specimen, which needed to be labelled (Figure 3). The machine used for the test must have an accuracy of ± 1.0%. The upper plate that allows the transfer of load must be supported on a sphere and be firmly attached to the upper head of the machine. To minimize the wear of the lower plate of the machine, it is recommended to use an additional hardened metal plate under the unit (Ecuadorian Normalization Service, 2016).

Figure 3. Specimens to the ambient air

RESULTS AND DISCUSSION

Absorption

In view of the fact that the project in Ecuador began in 2014, the date on which the INEN 639 and 643 standards were in use (Table 9), the first samples tested have the acceptance criteria, with only the absorption percentage being 15,06 ± 2,92. % (Table 10). It is taken into account that if only one of the three samples fail, the lot is accepted, otherwise it is rejected.

Table 9. Comparison of the two used standards

ECUADOR INEN 639 y 3066	ARGENTINA IRAM 11561-4
Step 1: Countersigned	
Samples must be countersigned if necessary and can be with high strength gypsum, sulphur materials for facing or mortar.	
A layer of mortar is placed on the upper and lower face of the block to level the samples.	It was not considered necessary to place a countersigned because the specimens were level. Therefore, only two neoprene plates were applied on the upper and lower face.
Step 2: Procedure	
The elements to be tested must be aligned vertically with respect to the centre of the load that is in the compression machine, these mats must be placed on the machine in the position in which they will be located in the construction. The applied speed must be adjusted during the test until it is constant (Ecuadorian Normalization Service, 2016)	The block is centred in relation to the ball joint of the compression machine. The direction of the load must coincide with the stress of the block during its use. The application of the load must be carried out at a uniform speed; When the machine is screw the advance is 0.125 mm / min and if it is a hydraulic machine, the increase in pressure is between 8 MPa (80 kgf / cm^2) of gross section per minute (Argentine Institute of Normalization and Certification, 1997).

Table 10. Values of percentage of absorption of hollow concrete blocks tested in Ecuador in 2014

Sample	Dry weight (g)	Saturated weight (g)	Absorption percentage
1A	7478,8	8725,8	16,7
1B	7041,4	7998,7	13,6
1C	6987,0	8209,3	17,5
2A	7124,9	8342,4	17,1
2B	6897,9	7863,9	14,0
2C	6816,9	8154,1	19,6
3A	7055,8	8013,4	13,6
3B	6840,3	8179,8	19,6
3C	7001,7	7892,4	12,7
4A	6985,0	7761,8	11,1
4B	7050,2	7887,8	11,9
4C	6938,1	7857,5	13,3

The table of results shows that two constructions acquired hollow concrete blocks that the lot is accepted when verifying the absorption percentage, the remaining samples are rejected. For a more simple and visual interpretation, a diagram is shown that represents the percentage of acceptance and rejection, taking into account the lot of each building. Table 11 lists the results of the samples tested under the absorption criterion by volume.

Table 11. Values of percentage of absorption of hollow concrete blocks tested in Ecuador in 2015

Block	Submerged Mass Mi (Kg)	Saturated Mass Ms (Kg)	Dry Mass Md (Kg)	Density kg/m³	Absorption		
					Kg/m³	Average	%
5A	5,4	12,1	10,1	1507,5	304,5		20,3
5B	5,6	12,3	10,4	1552,2	290,1	295,8	18,7
5C	5,3	11,9	10,0	1515,2	292,9		19,6
6A	6,4	13,2	11,6	1705,9	231,8		13,6
6B	5,7	12,8	11,2	1577,5	232,0	230,4	14,9
6C	5,8	13,0	11,4	1583,3	227,5		14,4
7A	5,0	12,1	10,1	1422,5	287,5		20,4
7B	5,6	12,6	10,7	1528,6	272,3	281,6	18,0
7C	4,7	11,7	9,7	1385,7	285,1		20,6
8A	5,7	12,1	10,6	1656,3	239,0		14,6
8B	5,4	12,0	10,5	1590,9	220,6	230,6	13,8
8C	5,5	12,1	10,5	1590,9	232,3		14,5
9A	7,1	14,4	11,8	1616,4	351,6		21,5
9B	5,9	12,6	10,7	1597,0	285,4	323,7	17,8
9C	3,4	8,1	6,5	1383,0	334,1		23,8
10A	4,8	11,6	9,9	1455,9	260,3		17,9
10B	4,8	11,4	9,8	1484,8	251,7	256,7	17,2
10C	4,5	11,2	9,5	1417,9	258,1		18,1
11A	3,0	8,3	7,1	1339,6	226,3		17,1
11B	3,0	7,9	6,9	1408,2	199,0	230,3	14,0
11C	3,1	8,2	6,8	1333,3	265,6		19,6
12A	2,8	8,0	7,1	1365,4	181,9		13,6
12B	2,9	8,2	6,8	1283,0	251,8	204,2	19,6
12C	2,9	7,9	7,0	1400,0	178,8		12,7
13A	4,5	11,3	9,9	1455,9	204,2		14,1
13B	4,5	11,5	10,1	1442,9	192,8	188,9	13,2
13C	4,9	11,9	10,7	1528,6	169,6		11,0
14A	5,7	11,8	10,4	1704,9	232,0		13,6
14B	5,6	11,9	10,4	1650,8	225,6	225,0	13,6
14C	5,2	11,9	10,4	1552,2	217,5		13,9
15A	6,7	15,3	13,9	1616,3	165,9		10,3
15B	5,2	13,3	12,0	1481,5	154,9	155,2	10,5
15C	5,7	13,7	12,5	1562,5	144,8		9,2

They are considered to be light blocks because their density is less than 1680 kg/m^3 (Table 3). When comparing the results with the specifications of the INEN 3066 standard, it is observed that 96% of tested blocks are accepted applying the maximum absorption criterion per unit [kg/m^3], since only two samples exceed the allowable 320 kg/m^3. However, when considering the criterion of maximum water absorption average of the three specimens tested, it has been observed that 18% are rejected (Figure 4). Undoubtedly, the revision of the 2016 regulations is more permissive and flexible in relation to the previous one, as when comparing the results with the criterion of maximum absorption percentage of 15% in individual samples (Figure 4). It is observed that 55% of the tested samples do not meet the requirements. The Argentine legislation mentions the process of calculation of absorption by volume and mass, that is to say in kg/m^3 and in percentage, but in the latter there is no required limit, which is why the maximum absorption percentage of 15% will be considered, in order to establish a direct comparison. The results of the tests conducted in Mendoza are detailed in Table 12.

Table 12. Absorption values in hollow concrete blocks tested in Mendoza Argentina

Block	Submerged Mass Mi (Kg)	Saturated Mass Ms (Kg)	Dry Mass Md (Kg)	Density kg/m³	Absorption		
					kg/m³	Average	%
1	12,3	18,0	17,0	2963,2	171,4		5,8
2	9,9	15,3	14,2	2620,5	203,6	203,7	7,8
3	10,6	15,6	14,4	2931,0	236,0		8,1
4a	6,4	10,5	9,6	2310,1	212,7		9,2
4b	6,9	11,0	10,2	2477,8	191,0	203,3	7,7
4c	6,1	10,4	9,5	2223,5	206,1		9,3
5a	7,2	11,5	10,6	2449,4	213,1		8,7
5b	6,9	11,3	10,4	2396,9	210,4	205,3	8,8
5c	7,0	11,3	10,5	2427,5	192,4		7,9
6a	6,3	10,3	9,6	2383,5	172,5		7,2
6b	5,6	9,9	9,2	2181,5	152,0	144,4	7,0
6c	5,8	9,9	9,5	2300,5	108,6		4,7
7a	6,4	10,5	9,8	2435,0	168,8		6,9
7b	6,1	10,3	9,7	2274,1	156,2	161,2	6,9
7c	6,0	10,3	9,6	2228,6	158,6		7,1

It is observed that the densities of the blocks that are used in this city are of normal type, since they have values higher than 2000 kg/m^3 (Table 4), for this type there is a limit of 210 kg/m^3, applied only to the criterion of maximum absorption of the average of the three specimens tested by type of hollow block of concrete found in the Mendoza market. When making a comparison between the obtained results, it is clearly observed that the hollow blocks of concrete used in Argentina, meet the quality standards dictated in the standards: However, the Ecuadorians, when they are made by hand, do not have the quality requirements. Figure 4 illustrates the absorption results for the two study areas, which demonstrate the quality of each of the masons.

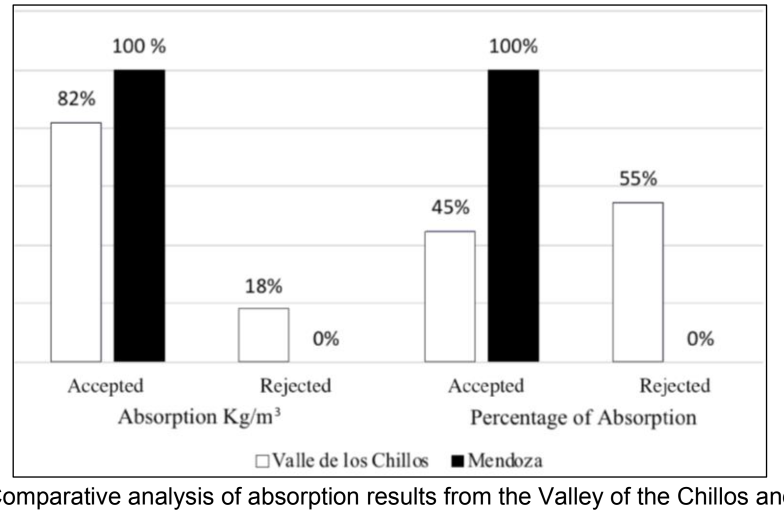

Figure 4. Comparative analysis of absorption results from the Valley of the Chillos and Mendoza

Compressive Strength

According to the Ecuadorian Standardization Service (2016) the minimum resistance of the average of 3 Class B blocks is 4.0 MPa, this means that the specimens collected from

Valle de los Chillos constructions and tested in the laboratory do not meet the minimum requirement due to that the results are less than 4.0 MPa (1.21±0.36). The results of the resistance to compression obtained by tests performed on specimens used in buildings of Mendoza-Argentina gave on average 4.84±1.38 MPa being a satisfactory value based on what are indicated by the Argentine Institute of Normalization and Certification (1997). These tests needed to be exposed with the minimum resistance of 4.0 MPa for the average of 3 blocks (Figure 5; Table 13 and 14).

The analysis of data in which a reliability level of 98.92% was assumed indicates that the data obtained in each test performed are within the limits established by $\bar{x} - 2{,}55\sigma$ and $\bar{x} + 2{,}55\sigma$, where $\bar{x}$ is the sample mean and σ the standard deviation. The maximum coefficient of variation calculated in the different series of data from the trials is 30%, since a low variability percentage is considered an acceptable analysis.

Table 13. Compression strength values of hollow concrete blocks tested in Ecuador

Block	Dimensions			Application area (cm^2)	Load of Compression Rupture (Kgf)	Compressive strength (kg/cm^2)	Compressive strength (MPa)
	b [cm]	h [cm]	L [cm]				
1A	14,20	20,10	48,30	685,86	11611,00	16,93	1,69
1B	14,00	20,00	49,00	686,00	12232,00	17,83	1,78
1C	14,20	20,05	48,50	688,70	11749,00	17,06	1,71
2A	14,00	19,00	37,50	525,00	3945,00	7,51	0,75
2B	13,60	18,50	38,00	516,80	4678,00	9,05	0,90
2C	14,00	19,00	39,00	546,00	4678,00	8,57	0,85
3A	13,90	18,90	39,00	542,10	4678,00	8,63	0,86
3B	13,80	19,00	38,50	531,30	4678,00	8,80	0,88
3C	14,50	19,00	38,00	551,00	3898,00	7,07	0,70
4A	14,00	19,00	39,00	546,00	5530,00	10,13	1,01
4B	14,30	19,00	38,70	553,41	5530,00	9,99	0,99
4C	14,00	18,70	39,00	546,00	5530,00	10,13	1,01
5A	15,09	19,75	40,01	603,48	8645,00	14,33	1,43
5B	15,12	19,55	39,90	602,33	8549,00	14,19	1,42
5C	15,10	19,62	39,39	598,84	8780,00	14,66	1,47
6A	15,31	19,74	40,00	613,12	11547,00	18,83	1,88
6B	15,49	19,59	40,00	617,80	13090,00	21,19	2,12
6C	15,30	19,15	39,90	611,04	9050,00	14,81	1,48
7A	15,24	19,57	39,87	607,83	10110,17	16,63	1,66
7B	15,13	20,03	39,97	604,40	9547,00	15,80	1,58
7C	15,04	20,01	40,00	604,00	9874,00	16,35	1,63
8A	15,09	20,00	40,00	604,24	8340,00	13,80	1,38
8B	15,19	20,09	39,90	602,00	7720,00	12,82	1,28
8C	15,10	20,01	40,00	606,76	8526,00	14,05	1,41
9A	20,00	19,50	39,90	799,80	6840,00	8,55	0,86
9B	15,13	20,00	40,01	605,68	7241,00	11,96	1,20
9C	10,02	19,68	39,90	398,95	6524,00	16,35	1,64
10A	15,12	20,15	40,00	607,60	8276,00	13,62	1,36
10B	15,42	19,98	39,90	613,06	8874,00	14,47	1,45
10C	15,35	20,10	40,10	610,88	8670,00	14,19	1,42
11A	14,00	19,00	37,5	525,00	3945,00	7,51	0,75
11B	13,6	18,50	38,00	516,80	4678,00	9,05	0,91
11C	14,00	19,00	39,00	546,00	4678,00	8,57	0,86
12A	13,90	18,90	39,00	542,10	4678,00	8,63	0,86
12B	13,80	19,00	38,50	531,30	4678,00	8,80	0,88
12C	14,50	19,00	38,00	551,00	3898,00	7,07	0,71
13A	14,00	19,00	39,00	546,00	5530,00	10,13	1,01
13B	14,30	19,00	38,70	553,41	5530,00	9,99	1,00
13C	14,00	18,70	39,00	546,00	5530,00	10,13	1,01
14A	15,01	19,04	40,02	600,70	6602,00	10,99	1,10

Block	Dimensions			Application area (cm^2)	Load of Compression Rupture (Kgf)	Compressive strength (kg/cm^2)	Compressive strength (MPa)
	b [cm]	h [cm]	L [cm]				
14B	15,02	19,01	40,04	601,40	8210,00	13,65	1,37
14C	15,02	19,04	40,02	601,10	6499,00	10,81	1,08
15A	15,10	20,00	40,30	596,45	6573,00	11,02	1,10
15B	15,20	20,40	40,70	603,44	7125,00	11,81	1,18
15C	15,00	20,00	40,00	590,25	4775,00	8,09	0,81

Table 14. Compression strength values of hollow concrete blocks tested in Mendoza-Argentina

Block	Dimensions			Compression burst load [kg]	Compression burst load [kN]	Compressive strength [MPa]
	b [mm]	h [mm]	L [mm]			
1	195	190	390	30200	296,16	3,89
2	190	190	390	26300	257,91	3,48
3	190	190	390	33500	328,52	4,43
4a	128	190	390	14000	137,29	2,75
4b	128	190	390	26500	259,88	5,21
4c	128	190	390	20469	200,73	4,02
5a	128	190	389	21000	205,94	4,14
5b	128	190	389	20900	204,96	4,12
5c	128	190	389	20978	205,72	4,13
6a	91	190	390	21200	207,90	5,86
6b	91	190	390	21800	213,79	6,02
6c	91	190	390	21487	210,72	5,94
7a	92	190	390	14900	146,12	4,07
7b	92	190	390	29400	288,32	8,04
7c	92	190	390	23700	232,42	6,48

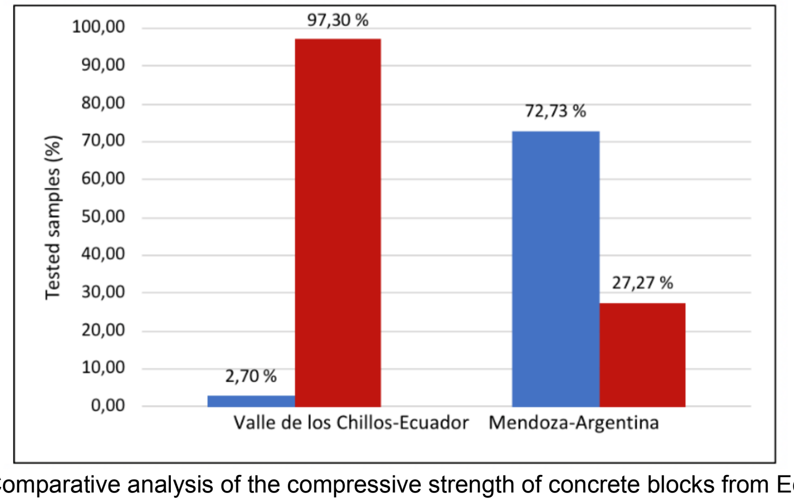

Figure 5. Comparative analysis of the compressive strength of concrete blocks from Ecuador and Argentina. In blue colour the accepted samples, and in red colour the rejected samples, respectively

To support the research, reference has been conducted to previous data from tests carried out on non-load bearing concrete blocks in the Materials Testing Laboratory of the National Technological University of Mendoza Regional Faculty, in which it has been demonstrated, that in 2011 the resistance compressions have been low, but over the years they have increased significantly, with resistance exceeding 6 MPa in 2014. The quality control applied by the entity responsible for the manufacture of blocks has been improving up to the present, which is why the difference in results between the two countries studied is evident.

Cost Analysis in Masonry

The masonry of hollow blocks of concrete differ greatly between those produced in Ecuador and Argentina due to the composition of each of them, because in Ecuador is used pumice material that is lightweight and low strength, mixed with cement; and in Argentina it is a mortar with fine aggregate and cement. Due to the factors described above, the price of masonry is higher in Mendoza. Table 13 lists a comparative cost analysis between the two study areas. In the case of Ecuador, the information is obtained from quotes and surveys made to the owners of the buildings that provided the samples to be tested and for Mendoza according to market prices, where this type of masonry was found during the stay (Robalino et al., 2015).

Table 9. Comparative analysis of costs of hollow concrete block masons for the study areas

Type Block	Average value per block in USD			
	Valle de los Chillos		Mendoza	
	Characteristic dimension (cm)	Cost in USD	Characteristic dimension	Cost in USD
Carrier	b=10	0,35	with finished for facade	2,8
	b=15	0,4		
	b=20	0,45	unfinished	2,2
Not carrying	b=10	0,27	b=10	1,22
	b=15	0,29	b=15	1,63
	b=20	0,34	b=20	1,83
Relief	b=10	0,31	Not used	
	b=15	0,33		
	b=20	0,35		

CONCLUSIONS

The seismic and aquifer conditions of the Valle de los Chillos in Ecuador and Mendoza in Argentina demonstrate similarity from the structural point of view and constructive pathology. Therefore, the current regulations, to evaluate the mechanical properties of hollow blocks of concrete, of both countries bear close similarity.

Although the external conditions in the two study areas are similar, the results of mechanical properties tests differ greatly, this is given by the type of material with which the masonry is made, since in Ecuador the blocks are light and porous, while in Argentine they are stiffer and heavier.

The parameters dictated by the INEN 3066 standard for absorption are currently more permissible, since when evaluating the Ecuadorian samples, it is found that the absorption by volume is fulfilled in most cases, but the same does not occur with the percentage of absorption exposed in the previous standard. Absorption is directly related to the type of material used, since if it refers to a lightened block, the porosity present allows water to rise through capillarity, causing pathological problems of humidity in the masonry of the buildings. Therefore, in the current study it has been considered appropriate to limit this mechanical property by mass and percentage.

In the samples tested in Mendoza the absorption parameters are met, both in mass and volume, since the material used for its elaboration is denser, which does not allow the rise of water and pathological problems with the passage of time despite the fact that have phreatic level conditions similar to those of the Valley of the Chillos. In particular that in Argentina, is less important to evaluate the mechanical property of absorption in concrete masonry.

Through the conducted tests in the laboratory it has been determined that the concrete blocks used in masonry Argentina meet the requirements of resistance to compression established in the IRAM standard, unlike the blocks used in buildings of Ecuador whose results gave resistance lower than the minimum required for Class B blocks (non-structural masonry), but could be used as relieves in the construction of ribbed slabs provided that the weight of the masonry does not exceed the maximum required.

With the development of this study article we expect to contribute in the manufacture of hollow concrete blocks in Ecuador, in order to take strict measures in quality control especially in the dosages with which it works as they do not meet the needs of quality and structural inputs. No doubt such control will raise production costs, but it is justifiable to expand the economic part as long as it is to safeguard the life and comfort of the population.

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EFFECT OF PASTE VOLUME, WATER TO CEMENTITIOUS MATERIALS RATIO AND FIBRE DOSAGES ON IN-SITU STRENGTH OF FIBRE-REINFORCED SELF-COMPACTING CONCRETE AS A REPAIR LAYER

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Abstract
Self-compacting concrete (SCC) can be easily used in the absence of consolidation, therefore; it is a good option as a repair material for repairing and retrofitting concrete structures. The quality of repair layer is highly effective on a successful repair. Accordingly, in this study, the factors affecting the quality of fibre-reinforced self-compacting concrete repair layer, including paste volume, the ratio of water to cementitious materials and the amount of fibre, are discussed. For this purpose, the in-situ strength of repair layers and cube samples with and without core are determined using pull-off method. Also, comparisons between in-situ strengths in different methods (with and without core, on cubes or on repair layer) with compressive and tensile strength of specimens have been done. Results show that, considering the great influence of shrinkage and tensile strength, with reduce in paste volume and cementitious material and also increase in fibre dosages, in-situ strength of repair layer increases. Moreover, we found that even in the best condition of concrete substrate layer (i.e. saturated surface dry) a repair layer has a lower strength than a cube specimen. Also, presence of fibres has the huge effect on the results of pull-off test depends on the method of the test (with or without core). Finally, results were predicted through linear regression and fuzzy logic.

Keywords: *Fibre-reinforced; Self-compacting concrete; In-situ strength; Paste volume; Water to cementitious material; Fibre dosage.*

INTRODUCTION

SCC (self-compacting concrete) is a new promising innovation in concrete industry in the last 20 years. Compared with traditional concrete, SCC does not need any vibration. This results in reduced number of workers, increased profit and improved working environment (Pade. C et al., 2008). Fibres are widely used in concrete, to prevent cracks. Polypropylene fibres prevent concrete cracking and shrinkage, especially in the early ages. The tendency to shrink causes tensile stress and cracking will occur. In some cases, cracks may spread to the entire thickness of concrete members and reduce its quality and durability (Bentur A. and Mindess S., 2007). High amount of fibres and its high tensile strength prevent these cracks.

Destruction of concrete pavements and concrete bridges under heavy use and environmental attacks over time are significant problems of these structures. Cracking in bridge decks during winter can causes concrete reinforcement corrosion and internal damage. To protect the decks against salt, applying a concrete repair layer is the most considerable effort. The best way to reduce destruction rate is to apply a concrete repair layer on decks of bridges which creates a waterproof barrier on decks to protect them against corrosion, water and chemical materials penetration (M. A. Issa and R.Z. Alrousan, 2008). Early-age shrinkage in repair layer is the most important reason for reducing durability of repair layers. Great attention has been always paid to SCC cracking, because, compared with ordinary concrete, SCC is designed with higher paste volume. Paste volume is usually defined as total volume of cement, water, minerals and chemical additives. Increase in paste will improve flow ability; however, it may cause adverse effects on mechanical properties and time-dependent concrete deformations. Among solutions proposed to control cracking which caused by shrinkage, one promising way is to use randomly distributed steel, carbon, polypropylene and other fibres in concrete that prevents the growth of cracks and creates bridge actions on crack's width. Fibres affect both width and length of shrinkage and reduce damage in concrete layer and substrate concrete interface. Additionally, fibres improve mechanical properties, durability, toughness, impact strength and fatigue strength of repair layer that is highly desirable for a repair layer (N. Banthia et al., 2006). Granju, (1996) states that fibres increase cohesion through crack growth control. Cracking results from non-uniform shrinkage in repair layer (Mailvaganam, N. et al., 2000; Suprenant B.A. and Malisch R.W., 1999; Suprenant B.A., 2002). It is also claimed that fibres have good effects on reducing cracking spread in repair layer (Tran et al., 2008; Lupien et al., 1995, 1990; Chanvillard et al., 1990; Chanvillard and Aitcin, 1990). In France, according to a study on the correlation between shrinkage and debonding of repair layer from substrate concrete, positive effect of reinforced repair layer on fibres was confirmed (Tran et al., 2008; Farhat, 1999; Granju and Chausson, 1995, 1996; Chausson, 1997; Turatsinze et al., 2005). It was also reported that the fibrous cement repair layer, compared with fibreless cement repair layer, is less affected by fatigue (Zhang et al., 2001; Rossi, 2002). Compressive strength of repair materials does not affect their bonding significantly. However, tensile strength has serious and positive effects on bonding.

Curing is an important factor to reduce shrinkage of repair layer and tensile stress on the interface with substrate. It prevents moisture drop and reduces early age shrinkage, as a result. It has other advantages as well, namely, reduce of plastic cracking risk, higher strength, improved durability and better wearing strength. Paulsson et al. (1998) recommended a minimum 5-day curing. It was also found out that direct sunshine has negative effects on bonding (Delatte et al., 2000). Proper substrate, appropriate materials and good curing result in long term appropriate interface properties. Findings show that repaired concrete beams and columns act like single layer (Silfwerbrand, 1987; FIP Federation Internationale de la Précontrainte, 1978). Pull-off method has been applied by many studies to determine in-situ strength and bonding. For instance, Mikami et al. (2015) discussed the effect of heat on bonding of FRP sheets to concrete, using pull-off test. Ghavidel et al. (2015) studied the effect of disc size on steel fibre-reinforced SCC through pull-off test. Sun et al. (2010) analysed three-dimensional finite element model of failure modes by pull-off test. Also, some researchers have worked on prediction of pull-off test and bonding between repair layer and concrete substrate using fuzzy logic, neural network analysis and other mathematical and heuristic methods (Sadowski et al., 2013; Naderi et al., 2012; Sadowski et al., 2015; Sadowski and Lukasz, 2013).

In this research the factors which affecting the quality of fibre-reinforced self-compacting concrete repair layer, including paste volume, the water to cementitious materials ratio and fibre dosage are discussed. For this purpose, the in-situ strength of repair layers and cube samples with and without core are determined using pull-off method. Also, comparisons between in-situ strengths in different methods (with and without core, on cubes or on repair layer) with compressive and tensile strength of specimens have been done.

LABORATORY EXPERIMENTS

In this study, three important parameters affecting rheological and mechanical properties of fibrous SCC, including the ratio of paste volume to total volume of concrete, the ratio of water to cement plus pozzolan and the amount of polypropylene fibres are discussed. The impact of any changes in these parameters on rheological properties of concrete is evaluated using slump flow, V funnel, L box and T50 tests. Afterwards, 15-cm cube samples for compressive strength test, standard cylindrical samples for Brazilian test and modulus of elasticity measurement and prismatic samples for shrinkage test were prepared. In order to determine in-situ strength of the repair layer, first, ordinary concretes with high strength (over 50 MPa) were made as cube samples which were divided into three equal 5-cm portions cut by saw. After six months (in order to the samples undergo all shrinkage in this long time and avoid shrinkage and possible errors after applying repair layer), 2-cm self-compacting repair layers were poured on the substrates. After 28 days of curing the samples in water, using pull-off, the in-situ strength of samples was determined. Pull-off was also used to examine the in-situ strength of the SCC poured on cube samples with and without coring, separately. Round gravels of maximum 12.5mm size, 2.64 g/cm^3 bulk density and 1.5% water absorption were used. The bulk density of the round sands used for this purpose was 2.6 g/cm^3 with 2.5% water absorption. In this study, 1-425 Portland cement was used, chemical composition of the cement is displayed in Table 1.

Table 1. Properties of the cement

Composition	SiO_2 %	Al2O3 %	Fe_2O_3 %	CaO %	MgO %	SO_3 %	CL %	K_2O %	Na_2O %	LOI %
Dosage	21.19	5.09	3.94	63.04	1.47	2.35	0.029	0.72	0.51	2.17

Micro silica with specific weight of 2200 kg/m^3 was added to the concrete mix as a portion of cement. Polypropylene fibres were 6 mm long. FARCO PLAST P10-3R super plasticizer was used which is based on modified polycarboxylate. Mix designs of the repair layers are listed in Table 2.

Table 2. Mix designs of the repair layers

Mix designs	Cement (kg /m^3)	Gravel (kg /m^3)	Sand (kg /m^3)	Limestone (kg /m^3)	Micro silica (kg /m^3)	Water (kg /m^3)	PP %	S.P. %	W/(C+MS)	V_{paste}
V1W1P0	405	740	850	229.5	45	144	0	1.5	0.32	0.4
V1W2P0	405	740	850	250	45	162	0	1.3	0.36	0.44
V1W3P0	405	740	850	200.5	45	180	0	1	0.40	0.48
V2W1P0	442	691	793	332.5	49.2	157	0	1.5	0.32	0.4
V2W2P0	442	691	793	277.5	49.2	177	0	1.2	0.36	0.44
V2W3P0	442	691	793	223.87	49.2	196.5	0	0.9	0.4	0.48
V3W1P0	486	636	729	357.7	54	173	0	1.1	0.32	0.4
V3W2P0	486	636	729	300	54	194	0	0.8	0.36	0.44
V3W3P0	486	636	729	239.5	54	216	0	0.6	0.40	0.48
V1W1P1	405	740	850	299.5	45	144	0.1	1.5	0.32	0.4
V1W2P1	405	740	850	250	45	162	0.1	1.3	0.36	0.44
V1W3P1	405	740	850	200.5	45	180	0.1	1	0.4	0.48
V2W1P1	442	691	793	332.5	49.2	157	0.1	1.5	0.32	0.4
V2W2P1	442	691	793	277.5	49.2	177	0.1	1.2	0.36	0.44
V2W3P1	442	691	793	223.87	49.2	196.5	0.1	0.9	0.40	0.48

Mix designs	Cement (kg /m^3)	Gravel (kg /m^3)	Sand (kg /m^3)	Limestone (kg /m^3)	Micro silica (kg /m^3)	Water (kg /m^3)	PP %	S.P. %	W/(C+MS)	V_{paste}
V3W1P1	486	636	729	357.7	54	173	0.1	1.1	0.32	0.4
V3W2P1	486	636	729	300	54	194	0.1	0.8	0.36	0.44
V3W3P1	486	636	729	239.5	54	216	0.1	0.6	0.4	0.48
V1W1P2	405	740	850	229.5	45	144	0.2	1.5	0.32	0.4
V1W2P2	405	740	850	250	45	162	0.2	1.3	0.36	0.44
V1W3P2	405	740	850	200.5	45	180	0.2	1	0.40	0.48
V2W1P2	442	691	793	332.5	49.2	157	0.2	1.5	0.32	0.4
V2W2P2	442	691	793	277.5	49.2	177	0.2	1.2	0.36	0.44
V2W3P2	442	691	793	223.87	49.2	196.5	0.2	0.9	0.4	0.48
V3W1P2	486	636	729	357.7	54	173	0.2	1.1	0.32	0.4
V3W2P2	486	636	729	300	54	194	0.2	0.8	0.36	0.44
V3W3P2	486	636	729	239.5	54	216	0.2	0.6	0.40	0.48

Figure 1 shows the slump-flow test based on EFNARC, (2005).

Figure 1. Slump Flow test

In order to determine in-situ strength of concrete, pull-off test with and without coring can be used. To assess the real quality of repair layer on substrate concrete, pull-off test without coring is used to find in-situ strength of the layer. Images of this process are illustrated in Figure 2.

Figure 2. determination of in-situ strength of repair layer using Pull-Off (without coring)

The in-situ strengths of repair concrete for sample cubes with and without coring were determined. Both tests with and without core were carried out to compare results and examine how much the tests without coring are reliable. The process has been shown in Figure 3.

Figure 3. Failure surfaces in Pull-off test on 15cm cubes (left: without coring, right: with coring)

RESULTS

Rheological and mechanical properties of the mix designs have been listed in Table 3. Modulus of elasticity and shrinkage have been determined using ASTM C469, (2002) and ASTM C157, (2004) respectively. Also, we used ASTM C1583, (2004) for Pull-off test.

Table 3. Rheological and hardened properties of the mix designs

Mix design	Slump flow (mm)	T50 (sec)	L – box (mm/mm)	V - funnel (sec)	Compressive strength (MPa)	Tensile strength (MPa)	Modulus of elasticity (GPa)	shrinkage (×10^6)	In- situ strength of non- core cubic specimen (MPa)	In- situ strength of core cubic specimen (MPa)	In- situ strength of repair layer (MPa)
V1W1P0	670	5.30	1	12.22	56.1	4.86	34.3	470	6.58	3.75	4.33
V1W2P0	630	3.85	1	11.20	55.2	4.77	33.1	600	5.63	3.67	4.08
V1W3P0	600	3.67	1	7.89	53.4	4.42	31.8	690	5.38	3.54	3.79
V2W1P0	730	3.50	1	8.05	53.5	4.67	33.4	520	5.50	3.50	4.08
V2W2P0	680	3.24	1	6.54	52	4.53	32.6	620	5.17	3.42	3.71
V2W3P0	650	2.86	1	4.65	47.3	4.02	30.7	760	4.63	3.33	3.29
V3W1P0	670	3.03	1	5.31	49.1	4.15	32.7	590	5.21	3.25	3.71
V3W2P0	550	2.67	1	3.87	46.2	3.93	31.4	740	4.75	2.92	3.04
V3W3P0	520	4.99	1	2.98	38.9	3.38	29.9	850	4.04	2.79	2.54
V1W1P1	650	5.80	1	13.40	57.9	5.47	37.7	450	6.67	4.17	5.75
V1W2P1	600	4.20	1	12.10	56.4	5.30	36.3	570	5.92	4.13	5.42
V1W3P1	560	4	1	8.66	52.8	5.04	34.8	660	5.04	4.04	4.83
V2W1P1	710	4.10	0.95	9	54.7	5.33	36.8	500	5.71	3.83	5.38
V2W2P1	660	4	0.90	7.63	51.4	5.11	35.5	595	4.83	3.75	5.17
V2W3P1	620	2.99	0.90	5.78	48.7	4.60	33.9	735	4.33	3.88	4.83
V3W1P1	660	3.60	0.85	6	49.5	4.76	35.6	560	5.67	3.08	5.25
V3W2P1	540	2.98	0.85	4.20	45.4	4.43	34.8	700	4.58	3.38	3.92
V3W3P1	500	8	0.70	3.30	37.2	4.20	31.9	790	4.21	3.08	3.58
V1W1P2	640	6	0.95	15	56	5.11	37.8	455	6.83	3.92	4.67
V1W2P2	570	4.5	0.95	14.10	54.8	5.04	35.4	580	5.79	3.79	4.54
V1W3P2	540	4.30	0.90	10.20	53	4.87	34.3	675	5.29	3.96	4.13
V2W1P2	680	4.60	0.85	10.60	52.8	5.14	36.3	515	5.33	3.71	4.58
V2W2P2	640	4.40	0.80	9.8	52.3	4.94	34.7	615	4.58	3.63	5.17
V2W3P2	600	3.26	0.80	6.90	46.6	4.32	33.1	750	4.83	3.42	4.63
V3W1P2	630	4.10	0.70	7.23	49.5	4.42	34.9	570	5.04	3.21	4.29
V3W2P2	530	3.5	0.70	5.10	45.7	4.13	33.4	710	4.96	3	3.58
V3W3P2	480	8.32	0.65	3.98	38.3	3.58	30.7	820	4.33	2.83	2.92

The first result is that in fibreless samples, slump flow and T50 of all mix designs are within proper limits. It is clear that with adding fibre and increase in its dosage, slump flow and speed of concrete flow, reflected in T50, have decreased. Additionally, increase in paste and water to cementitious materials ratios, as is observed from T50 test results, concrete flows faster. No failure was found, and all the mix designs were within proper limits, except for the last one in which the lowest limit of slump flow was not achieved. Increase in amount of fibre from zero to 0.1% and then to 0.2% caused slump flow to reduce and T50 test to take longer. With the change of fibre amount from 0.1% to 0.2%, reduction in slump flow and T50 time was almost doubled (1.94 times for slump flow and 1.80 times for T50). V funnel discharge time increased as the amount of fibre increased from zero to 0.1% and then to 0.2%. Blockage ratio which determined from L-Box test reduced with the amount of fibre increased from zero to 0.1% and then to 0.2%.

Tensile strength increases by adding 0.1% fibres, however; it drops if this amount rises to 0.2% but it is still higher, compared with fibreless samples. Changes in tensile strength which result from changes in paste volume and the amount of water, are similar to compressive strength so that increase in amounts of paste and mixing water lead to reduce in tensile strength. Adding fibres causes decrease in shrinkage which is due to positive effect of fibres on cracking control and tensile stress. In spite of controlling effect of fibres in samples with 0.2% fibres, compared with samples with 0.1% fibre, less shrinkage occurs i.e. for samples with 0.1% polypropylene fibres, it is -4.70% and for samples with 0.2% polypropylene fibres, it is -2.53%.

Results of in-situ strength of sample cubes without coring are similar to that of Brazilian test, i.e. decline in results occurs with increase in paste volume and water amount. However, in samples with fibre, results are not similar to Brazilian test. Considering that coring was not carried out for this test and only metal cylinder of pull-off test was fixed on the sample, the positive effect of fibres in tensile strength should not be neglected. Results confirm this point as well. Therefore, adding fibres to samples does not make any change in test results and they are almost similar to fibreless tests. Hence, it can be claimed that proper and reliable results are obtained from this test, once fibreless samples are evaluated. However, for samples with fibre, the test cannot represent tensile strength of concrete (when we don't core). While, regarding low or no impact of fibres on compressive strength, even in samples with fibre, the test can be an indicator of compressive strength and in-situ quality of samples.

The in-situ strength test for sample cube with coring indicates tensile strength and concrete quality, the process of which is similar to that of Brazilian test, i.e. reduce in in-situ strength occurs with increase in paste volume and water amounts. However, unlike non-core pull- off test, in fibre ones, results are similar to tensile strength of the concrete. The reason is quite clear. Considering coring and core failure at a depth of 2.5 cm, fibres will be effective and due to tensile nature of pull-off test, results improve, compared with fibreless ones. Additionally, despite better results compared with fibreless samples6, samples with 0.2% polypropylene fibres show weaker results compared with specimens with 0.1% polypropylene fibres, which is an indicator of optimal 0.1% polypropylene fibres amount.

The in-situ strength test for repair layer evaluates the quality of the layer on concrete substrate. Due to considerable impact of polypropylene fibres on reduction in shrinkage and also huge impact of shrinkage on the quality of repair layer, the quality of this layer improves via adding 0.1% polypropylene fibres. This is while the same test on sample cube did not indicate any change in results. This issue shows considerable effect of shrinkage on declined quality of repair layer and also positive impact of fibres on control of shrinkage and increase in quality of repair concrete. With an increase of fibres from 0.1% to 0.2%, results drop again, but they are still better than fibreless specimens. In-situ strength, as expected, depends on the amount of water to cementitious materials and paste volume, so that an increase in either of them causes a decline in the quality of repair layer. To determine in-situ strength of concrete, pull- off test is the best choice, with partial destruction in core and no destruction in non-core specimens. Considering that both tests were carried out in this paper, a comparison between two methods has been done.

It is obvious that results have high dispersion, therefore; there is no reasonable correlation between the two methods for fibrous concrete samples (R^2= 0.3068), because failure occurs on sample surface and fibres do not have any impact on pull-off (without coring) results.

For pull-off test with core, we obtained R^2=0.9032 for fibreless specimens. So, it is quite clear that there is a significant correlation between compressive strength and in-situ strength of core pull-off which proves the reliability and validity of the method to evaluate concrete quality. Also, we obtained R^2=0.7126 for fibrous ones. A comparison between that and the former one shows that the accuracy of correlation between the two methods has decreased because fibrous samples were evaluated. Fibres do not have significant impact on compressive strength, while core pull-off results are highly affected by fibres. Hence, the method accuracy declines.

Figures 4 and 5 shows the relationship between pull-off test results for both core and non-core in-situ strength and sample cube compressive strength for both fibrous and fibreless samples. Although fibres do not have any significant impact on non-core pull- off test results, the diagram shows low accuracy of the method ($R^2 < 0.7$). For fibreless samples, core pull-off test is the best option, however; for fibre samples, non-core pull-off test is better for estimation of compressive strength. Results from in-situ strength of repair layer and cube samples are displayed in Figure 6. As it is observed, in-situ strength of the repair layers, in most cases, is lower than cube samples which is an indicator of the impact of shrinkage on the quality of repair layer.

PREDICTING THE IN-SITU STRENGTH OF REPAIR LAYER

As was mentioned before, the in-situ strength of repair layer is highly affected by shrinkage, therefore; fibres play the main role in the amount of this strength, unlike the non-core in-situ strength of cube samples with no considerable effect of fibres on results. Using linear regression, a correlation was obtained to estimate the in-situ strength of repair layer. Parameters related to the in-situ strength include paste volume, ratio of water to cementitious material, fibre dosages, shrinkage, compressive strength and tensile strength. Suitable accuracy of the obtained equation (R^2=0.870 and the average error of about 5%), approves the impact of the parameters on the in-situ strength. It is noteworthy that the effect of parameters which was proved earlier in laboratory and test results. Following equation was obtained:

INSITU = -15.007+ 16.294(PV) + 16.878 (WTOC) 0.623+(F)- 0.004(SH)+ 0.035 (CS) + 1.510 (TS)

Where: INSITU is the in-situ strength of repair layer by non-core pull- off test after 28 days in MPa. PV is the ratio of paste volume to total volume of the concrete, WTOC is the ratio of water to cementitious materials, F is fibre dosages (by volume of the concrete), SH is the amount of shrinkage × 10^6, CS is 28- day compressive strength of 15-cm cube sample in MPa and TS is results from Brazilian test on standard cylinder after 28 days in MPa.

Also, we use the table look-up scheme, recursive least square and nearest neighbourhood clustering for the prediction. The input variables are paste volume (PV), water/cementitious materials ratio (W/CM), polypropylene (PP), shrinkage $\times 10^6$ (SH), compressive strength (CS) and tensile strength (TS), and the output variable is in-situ strength (IS). Before applying the methods, we randomly choose the 70 percent of the samples as the train data and the others as the test data. For all the methods, the membership functions are considered as the Gaussian functions. In the Table look-up scheme, the number of the membership functions are set to 5 for all the input and output variables. For the recursive least squares, the same number of the membership functions are set for the input variables. Finally, in the nearest neighbourhood clustering method, we set the radius equal to 1.2. It should be mentioned that we used all the results that we obtained from the experimental results. In Table 3 we just mentioned the averages of the results, but in this section, we use all of them to reach more exact predictions.

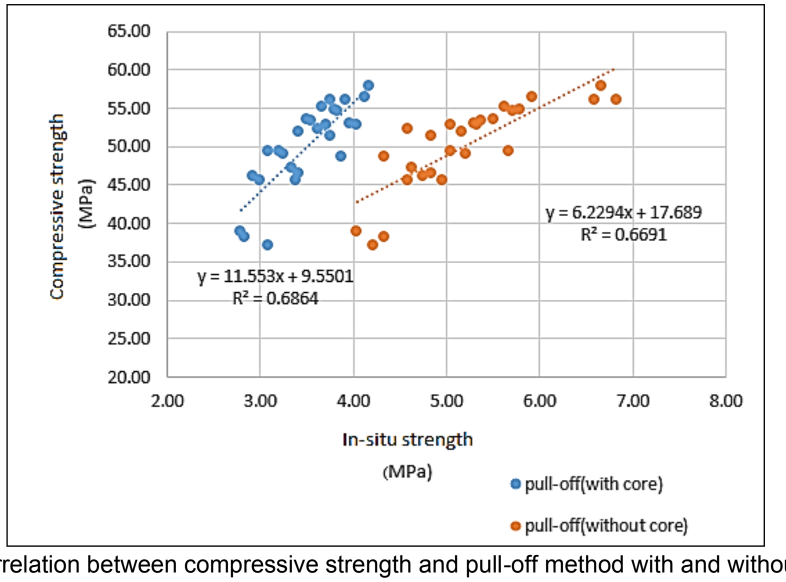


Figure 4. correlation between compressive strength and pull-off method with and without coring for fibreless and fibrous specimens

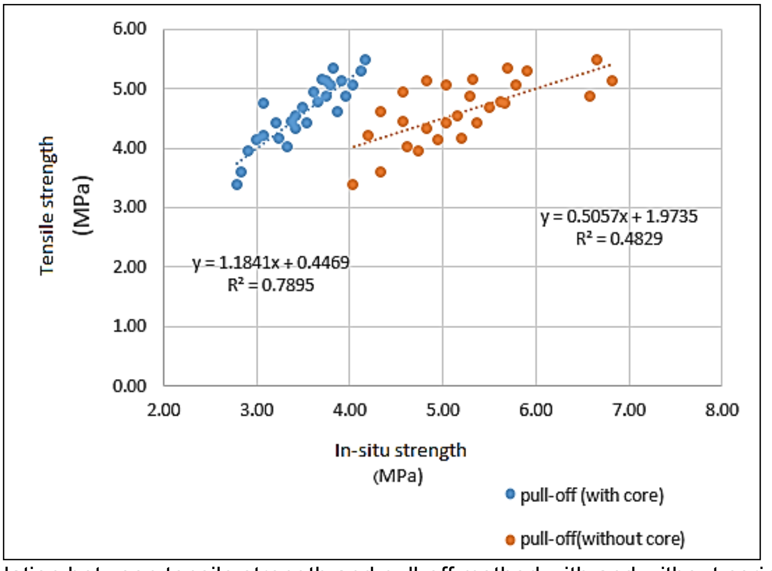


Figure 5. correlation between tensile strength and pull-off method with and without coring for fibreless and fibrous specimens

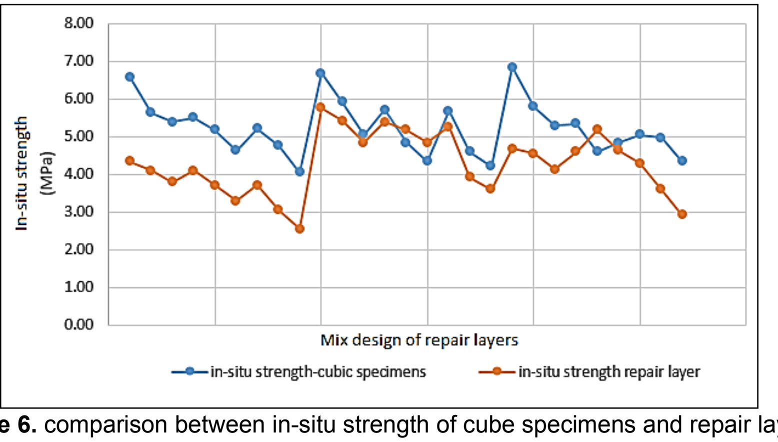
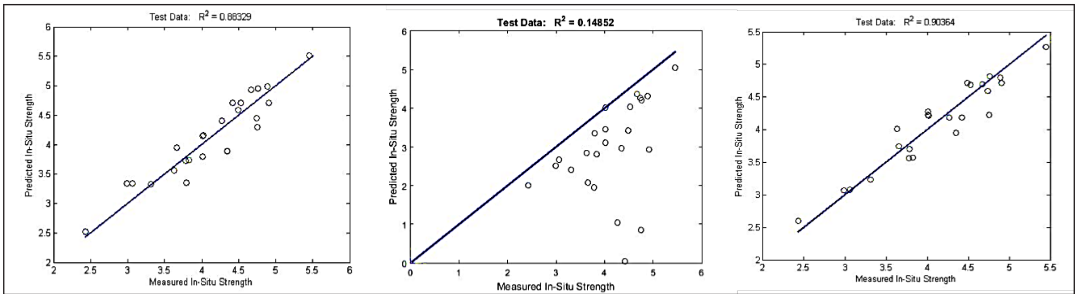


Figure 6. comparison between in-situ strength of cube specimens and repair layers

Figures 7, 8 and 9 shows the correlation between predicted and measured in-situ strength for all the mentioned methods for the test data, train data and all data, respectively. The results, in Figure 7, demonstrate that the recursive least square approach gives the worst matching between the generated outputs (predicted data) and the given targets (measured data). The results obtained by the table look-up scheme are better than the results generated by the recursive least square and worse than those attained by the nearest neighbourhood clustering method. The nearest neighbourhood clustering method provides the best generalization when compared with the other methods.

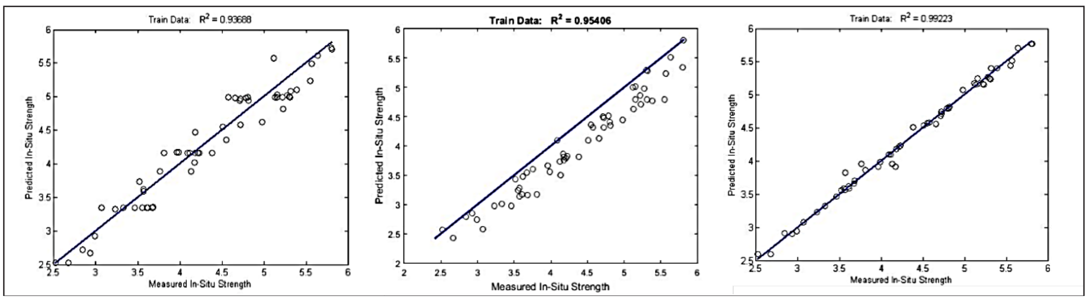
As shown in Figure 8, the same relationships hold true for the train data. Therefore, the nearest neighbourhood clustering method generates both the best training and the best prediction among the three mentioned approaches.

Figure 9 shows the total performances of the algorithms. As this figure illustrates the nearest neighbourhood clustering method provides better solutions.



(a) (b) (c)

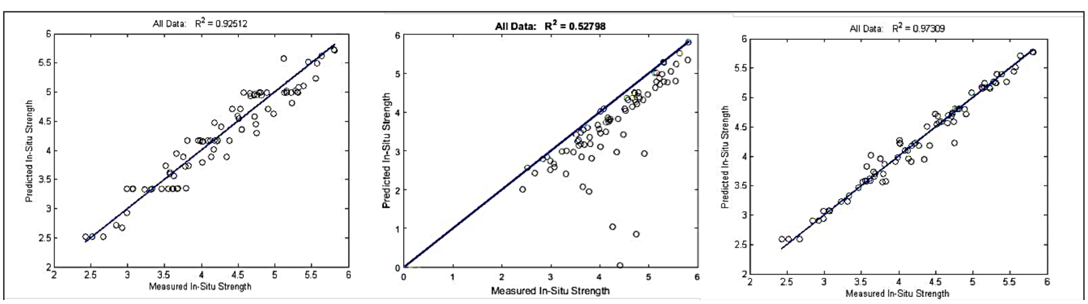
Figure 7. Correlation between measured and predicted In-Situ Strength for the test data; (a) results of the table look-up scheme, (b) results of the recursive least squares, and (c) results of the nearest neighborhood clustering.



(a) (b) (c)

Figure 8. Correlation between measured and predicted In-Situ Strength for the train data; (a) results of the table look-up scheme, (b) results of the recursive least squares, and (c) results of the nearest neighbourhood clustering.

As we can see, the best results are obtained using the nearest neighbourhood clustering method (R^2=0.97309) and the second method is the table look-up scheme (R^2=0.92512). They predict the experimental results better than linear regression. Recursive least squares method has the worst results with R^2=0.52798 and so it is completely unreliable in this field.



(a) (b) (c)

Figure 9. Correlation between measured and predicted In-Situ Strength for all the data; (a) results of the table look-up scheme, (b) results of the recursive least squares, and (c) results of the nearest neighbourhood clustering.

RESULTS AND DISCUSSION

In this research, we determined the effect of paste volume, water to cementitious ratio and fibre dosages on in-situ strength of repair layer. Also, we showed that in repair concrete without fibre, pull-off test without coring which it is more simple and absolutely non-destructive, could be used instead of pull-off test with coring. But with usage of fibre in repair layer, the results of these two-method showed different character of the concrete repair layer.

1. Adding fibres results in reduce of slump flow and flow ability of concrete which is reflected in T50 test. Additionally, with the change of fibre amount from 0.1% to 0.2%, discharge rate of V funnel increases 2.77 times and the blocking in L box test reduces 2.01 times.
2. Adding polypropylene fibres results in the increased tensile strength of concrete. On average, 0.1% fibres cause 14.53% and 0.2% fibres cause 7.24% more tensile strength.
3. Adding fibres results in increasing of modulus of elasticity, which is 9.43% for 0.1% and 7.65% for 0.2% fibres. As is seen, samples with 0.1% fibres show better results.
4. Adding fibres results in reduce of shrinkage. On average, adding 0.1% fibres causes 4.70% and adding 0.2% fibres causes 2.53% reduction in shrinkage.
5. Increase in ratio of water to cementitious materials ratio and paste volume, caused reduction in results of pull- off tests.
6. In fibreless conditions, there is a reasonable correlation between the results of pull-off test for cube samples with and without coring.

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THE IMPORTANCE OF SPACE OCCUPANCY AND UTILIZATION AUDIT OF A MALAYSIA PUBLIC UNIVERSITY FOR FUTURE PLANNING

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Abstract
The purpose of this paper is to present the importance of space audit finding of a Malaysia Public University (MPU) focusing on the space occupancy and utilization assessment criteria. The recent research conducted for the Ministry of Higher Education in Malaysia has introduced five space audit criteria that can be adopted for space audit exercises in MPU. The space audit exercises promote a more strategic approach of facilities management to support the strategic objectives of the university. The space audit is conducted within three months at a selected public university in Malaysia. Data were collected through fieldwork activities, documentation review and retrieval including archival records review. This paper provides an insight into a better understanding of space audit application within a Malaysia public university. The findings of the audit reveal that the space occupancy and space utilization rates play an important role in determining the future space required by a faculty or a university. The data assisted the university management to clarify whether to maximize the use of existing space or to request for new learning space. Thus, it is important for the university to have a proper space management audit, focusing on occupancy and utilization criteria for future planning and decision-making process.

Keywords: *Facilities management; Space audit; Space capacity; Space frequency; Space management*

INTRODUCTION

The current era of globalization or world without borders has contributed a significant impact on the demand for transformation of higher education institutions. It increases the importance of facilities thus demanding a more strategic approach to their management (Hamid, 2009). As a reflection to these scenarios, the requirement of academic space for university keep on increasing from time to time. The increasing demands of academic spaces at the university is the caused by a drastic increasing number of students. As example, the total number of foreign students in Malaysia has been increased from 27,872 students to 80,750 students since 2002 until 2010, (Higher Educational Dept. MOHE, 2013). The Malaysia Ministry of Education had published a book entitled ‘Space Management Best Practice for Public Higher Education Institution’ in 2013 based on the space audit conducted at Universiti Sains Malaysia, Universiti Sains Islam Malaysia and Universiti Teknikal Malaysia Melaka (MOE, 2013). The book introduced the space management of higher education institution related to policy, guidelines and procedures that can be adopted by the university organisations.

The aim of the space audit is to ensure that the space not only had been used in an optimum and economical way, but also conducive for the purpose. Therefore, this paper focuses on services related to the demand of accommodation for a university which covered on strategic space planning and management. In the space management context, it is important to identify the range of different space’s types provided in the university in order to satisfy its strategic goals and objectives (Hamid, 2014). The types of university space can be classified as academic space, administrative office space, commercial area, student support centre, library space, residential space and others. The space audit can be conducted to review the current performance or the existing position of the available space within the university. The opportunities identified will ensure that university has the ability to maximize the space use as well as improve the quality of space. In addition, the management of space plays a critical role in the iterative process of planning and development of the university (SMG, 2006a). Therefore, most of the universities had developed their own space management policy and guidelines for users.

Hamdan et al. (2012), had identified several issues occurred in managing building space in higher education institutions in Malaysia. First, lack of information recorded in space usage inventory is one of the most critical issues in the space management. This might influence facilities management strategy in supporting core business of the public higher education institutions in Malaysia. Second, optimization space in higher education is still low especially in building academic space. This is due to fact that the public higher education institutions in Malaysia are more focusing on preparedness and producing space rather than getting higher optimization of space usage. All of these issues exist due to the lack of understanding on optimization concept in space management by the management public higher education institutions. Third, there were a few researches related to utilisation rate in higher education institutions in Malaysia and the overall findings show that it is quite low (Wahab, 2005; Abdul Rahman, 2009). This is related to the unsystematic timetable management by the academic sector in determining the allocation of classes. Lastly, the occupancy rate for space in higher education institution is still lower especially in building academic space which is also due to the unsystematic timetable management. Therefore, this paper presents the finding from the space audit by focusing on the space occupancy and space utilisation criteria in determining the optimization of the existing space at one public university in Malaysia.

LITERATURE REVIEW ON SPACE MANAGEMENT AUDIT

The space management is a vital component in managing infrastructure facilities of a public university due to the significant main role which is to maximize the existing usage of building space infrastructure and minimize new development of building space. Therefore, the beneficial impact of the space management not only contribute to the high space utilisation and minimizing on new development, but also the space which is conducive, functional, effective and efficiency of timetabling schedule, appropriate and flexibility of space usage as well as economical of maintenance cost (MOE, 2013). Facilities planning and space management is a decision-making process that gives direction to manage and provide adequate space for administrative and academic units on a campus. Effective facilities planning and space management is critical in maximizing the use of available resources which includes the unit assignment, building inventory, utilization, modification and evaluation of academic and administrative facilities on the campus (Blancheette, 2010). Space management is the most important aspect of the physical resource's public management and not only in terms of optimization, but also related to the cost of maintenance operations (Downie, 2005; Ibrahim et al., 2011). Therefore, university management needs to analyse the space availability for future planning especially in terms of student's enrolment (SMG, 2006b). The space audit criteria that has been developed for space audit assessment is focusing on space utilisation rate as a tool for assessing space needs (SMG, 2006c). The space utilisation assessment model had been developed by the National Office Audit, New Castle, United

Kingdom in 1996. This model focused on the occupancy rate and frequency rate to determine the percentage of utilisation rate. According to NOA (1996), the level of utilisation rate that achieved more than 35% is considered as good. The space utilisation assessment model had also been adopted by the Space Management Group in 2006 by using a similar calculation method as published by National Audit Office. There is no requirement for submission of utilisation rate for approval of additional new space. In Malaysia, the space utilisation analysis has been used to evaluate the existing assets to accommodate current and future needs of the space within the organisation (Haron et al., 2007; Abdul Rahman et al., 2009). Malaysia Public University used the guidelines to determine space norms and approval application as developed in the Guidelines and Rules for Building Planning by Standard and Cost Sub-committee, Economic Planning Unit (EPU) Malaysia (EPU, 2008; EPU, 2015). The recent research conducted for the Ministry of Higher Education in Malaysia has introduced five space audit criteria namely space capacity, space frequency, space occupancy, space utilization and space condition (MOHE, 2012). These criteria determined the importance of the space audit assessment and the relevance for the Ministry and university management decision making process. The data obtained from a space audit is used to update the university's building and academic space inventory. The space capacity assessment criteria measure the rate of academic space usage and determine the maximum space capacity and the density population of space usage. The space capacity audit includes room-specific tours to confirm that the space is used and occupied as designed as well as to assess the quality of the space and physical conditions of each academic space. The result can be used in identifying space with low and high usage, estimating or forecasting future space need and for decision making process that is related to the space allocation or space planning (Hamdan et al., 2016a).

Besides frequently determined the space availability for future planning, the space frequency audit criteria also identify the usage rate of the academic spaces based on the actual usage of the spaces (MOE, 2013). The result of space frequency creates an opportunity for the university management to efficiently and productively use available space such as renting it out to another organisation. Meanwhile, the space occupancy audit criteria referred to the number of people using a room such as meeting room, classroom, laboratory, workshop and others. Therefore, in order to measure the level of occupancy rate of an academic space in the university, a calculation mechanism has been set up by scholar in space management studies by dividing the actual number of students in a week into the maximum capacity of space in a week (TEFMA, 2009; Shahril et al., 2012). The space utilization criteria determine the overall performance of space management in public university and measure how the space is being used. The result of space utilization measurement and analysis will show indication whether or not, each room of space are fully optimized by every academic session in each semester. Utilization rates can be assessed in terms of both actual use and predicted use. Each of the result contributed different significant impact to higher management and stakeholders.

This shows that conducting space audit assessment is a must in order to understand the relevance of space utilization analysis in public university. Therefore, to prove the relevance of space utilization criteria in this study, it has been tested at a public university in Malaysia. The study applied space utilization standard based on Tertiary Education Facilities Management Association (TEFMA, 2009). The fifth criteria introduced is a space condition or physical assessment of the space. As put forward by Hamdan et al. (2016b) in the performance audit assessment of academic space in Malaysia public universities, there is a need to study on space capacity and also the physical space condition of the room. The downside to the model evaluating the management of existing space is it focuses more on the determination of the space use level. Nevertheless, the aspect of the compliance assessment standard provision of space and the evaluation of the functionality of existing space which have a major impact on space management academics in universities are not addressed. In fact, the meaning and understanding of space audit assessment is the process of activities to inspects, measure and re-evaluate the use of space, the space standards compliance and also the condition of the room. In addition, space audits provide information for the assessment of space allocations, prospective planning to accommodate changing situations and realignment of program priorities. Furthermore, the space management issues faced by the MPU contributed to the constraints of the effective facilities management decision making process. Therefore, there is the need for the development of a good space management practice for MPU to overcome these constraints (Hamid and Hamdan, 2014).

METHODOLOGY

The case study involved 986 learning spaces or academic rooms with equivalent of gross floor area of 87,428 square meters and usable floor area of 52,433 square meters at University A. The academic building comprises of main lecture hall, classrooms for teaching and learning activities, studio for architecture, art and design students, computer room, laboratory for science and engineering students, workshop for architecture and engineering students as well as postgraduate room. The first phase of space audit was conducted by six researchers and 39 research assistants on the 6th to 11th December 2016. The second phase of audit was conducted on the 19th to 20th January 2017. The summary of space audit for academic building is shown in Table 1.

Table 1. Summary of Space Audit

No	Type of Spaces	No of Spaces (unit)	Gross Floor Area (GFA) m2	Usable Floor Area (UFA) m2	Percentage (%)
1	Classroom	562	35,007	20,980	64.0
2	Studio	79	7595	4557	5.0
3	Laboratory	129	20,056	12,034	4.0
4	Computer Room	128	12,060	7236	11.0
5	Workshop	16	2388	1433	1.0
6	Lecture Hall	41	7498	4499	13.0
7	Postgraduate Room	31	2824	1694	2.0
	Total	**986**	**87,428**	**52,433**	**100.0**

Space Occupancy and Utilization Measurement

There are several guidelines and requirements in measuring the space occupancy and space utilization of a space building:

a. Determine which categories of the population load. The load of this population may refer to the planning standards by the government or the body of the relevant professional agencies in the construction industry.
b. Determine the usable floor area (UFA) space and allocate 60% from the gross floor area. The calculation is taking into accounts the whole area deduction or reduction of space that cannot be occupied thus used as walls, columns, stairs, utility room and so on. Here are the types of the calculated area based on the requirements:

i. Gross Floor Area (GFA) - Total gross floor area encompasses an area including the main floor and the floor area ancillary.
ii. Main Floor Area (MFA) - The total area enclosed spaces are only measured from wall to wall including M & E (Room motor elevator, water tank, AHU etc.) but does not include space ancillary.
iii. Ancillary Floor Area (AFA) - The total area covered and open space, measured from wall to wall, including sidewalks, air space, open veranda and parking.
iv. Usable Floor Area (UFA) - Total floor area that can accommodate users by taking into account the suitability of the related.

c. Determine the space capacity maximum measurement.
d. Determine the space frequency measurement.
e. Determine the space occupancy assessment as shown in Table 2.

Table 2. Space Occupancy Assessment Method

Data Evaluate	Method of Assessment	Assessment Impact
1. Gross Floor Area (GFA) 2. Usable Floor Area (UFA) 3. Student Enrolment (SE) 4. Maximum Capacity (MC) 5. Maximum Capacity with Adjustment (MCA)	No of Student x 100 Maximum Capacity Adjustment	1. Determine overall optimum space 2. Student enrolment planning 3. Planning for new space development 4. Strategic space planning 5. Expenditure planning for space management

f. Determine the space utilization assessment as shown in Table 3. Classroom utilization is used to determine efficient use of classrooms and the appropriate quantities of classrooms needed of various sizes. Classrooms in scheduled that use 40 hours per week are defined as 100% utilized. The design goal is for each classroom to be utilized between 85% and 100%.

Table 3. Space Utilization Assessment Method

Data Evaluate	Method of Assessment	Assessment Impact
1. Gross Floor Area (GFA) 2. Usable Floor Area (UFA) 3. Student Enrolment (SE) 4. Space Frequency Rate (SFR) 5. Space Occupancy Rate (SOR)	Space Occupancy Rate x Space Frequency Rate / 100	1. Determine overall optimum space 2. Student enrolment planning 3. Planning for new space development 4. Strategic space planning 5. Expenditure planning for space management

g. Determine the space occupancy and utilization score (Hamdan et al., 2016). Table 4 shows the space occupancy and utilization score with the index guidelines.

Table 4. Space Occupancy and Utilization Score

No	Category Score	Capacity (%)	Category Score	Description
1	Crowded	More than 100	Not Comply	Exceeded maximum capacity
2	High	76-100	Comply	Optimum and efficient space usage
3	Medium	60-75	Partial Comply	Partial optimum and efficient usage
4	Low	0-59	Not Comply	Space not optimum and inefficient usage

FINDINGS AND DISCUSSION

Space Occupancy Audit

From Table 5, the current students' enrolment in 2017 for this University is 29,637. The students' enrolment is expected to increase to 32,972 in 2018 and 36,294 in 2019 and later reach 40,000 number of students in 2020 as projected by University A, Strategic Planning Division. The maximum academic capacity is 32,421 students based on the availability of academic furniture available on site. The space occupancy rate for the overall University A in 2017 is 82.0% which is considered as optimum usage with the current students' enrolment. However, in 2018 and 2019, the occupancy rate reaches more than 100%. This can be seen from the space occupancy rate for Faculty A, Faculty E, Faculty F, Faculty H, Faculty L, Faculty M and Faculty P reach more than 100%. The findings indicate that, total occupancy rate stands at 82% in year 2017, of which has reached level of optimum and efficient space usage as per description in Table 4; Space Occupancy and Utilization Score. However, the findings will be treated as a guide to the Management of the University in particular the Strategic Planning Division to further advise and consult the Office of Academic Affairs about the space capacity rate. The issues could be addressed by the Academic Affairs as follow:

i. Re-arrange the lecture timetable by considering the actual space capacity and space frequency. The normal practice of space frequency, from 8 am to 5pm (Typical time) or else to stretch the lecture time from 8pm to 10pm (Non-typical time). Thus, it shows that the space occupancy and space frequency must be read together to achieve optimization of space utilization.
ii. To ensure that not only availability of the academic space, but the space must be in good condition with sufficient facilities and be able to operate and fit for teaching and learning exercise.
iii. Re-arrange number of students for every classes to attain maximum level and at very best endeavour to achieve uniformity throughout the programme and courses offered at the university.

The findings for space occupancy rate shows that unstable dispersion throughout faculties, where some faculties known as Faculty A, Faculty E, Faculty F, Faculty H, Faculty L, Faculty M and Faculty P have exceeded optimum level of 100%, while some faculties recorded compliance to optimum level between 76% – 100% and remaining three faculties (Faculty B, Faculty J and Faculty K) have space occupancy rate below than 50%. The issues could be addressed by the Academic Affairs as follow:

i. To provide sustain projection of student enrolment based on program and to engage with every faculties at the university prior to any decision of new enrolment.
ii. To strictly identify the so-called less utilized space with indicates low frequency and below space capacity optimum level. If deemed necessary, the space caretaker (Academic affairs / Vice chancellor office) to surrender the identified spaces to another program or faculty.

Table 5. Summary of Space Occupancy Audit

No	Faculty	Students Enrolment (2017)	Forecast (2018)	Forecast (2019)	Maximum Academic Capacity	Space Occupancy Rate % (2017)	(2018)	(2019)
1	Faculty A	1162	1250	1349	1010	115	124	134
2	Faculty B	56	83	112	194	29	43	58
3	Faculty C	644	888	1158	984	65	90	117
4	Faculty D	1751	1930	2109	2313	76	83	91
5	Faculty E	2260	2873	3090	2236	101	128	138
6	Faculty F	1841	1890	1969	1841	100	102	106
7	Faculty G	2291	2394	2496	2653	86	90	94
8	Faculty H	2042	2234	2451	788	259	283	311
9	Faculty I	701	929	1189	1346	52	69	88
10	Faculty J	600	600	600	1648	36	36	36
11	Faculty K	160	218	278	699	23	31	40
12	Faculty L	3522	3671	4070	2713	130	135	150
13	Faculty M	3040	3475	3877	2009	151	173	193
14	Faculty N	478	615	767	916	52	67	84
15	Faculty O	605	594	615	1000	61	59	62
16	Faculty P	1990	2261	2529	2576	77	88	98
17	Faculty Q	4725	5108	5452	4542	104	112	120
18	Faculty R	1769	1959	2183	2953	60	66	74
	Total	**29,637**	**32,972**	**36,294**	**32,421**	**82**	**101**	**113**

Space Utilization Audit

The space utilization rate is based on the space occupancy rate and space frequency rate for each faculty and the overall University. The overall of 986 nos. of academic space in the University recorded frequency rate of 28% which is low and does not comply with the index developed for this study. The data shows that out of 35,920 maximum hours available for the whole academic spaces, only 9977 hours have been utilized by program and faculties. The overall space utilization rate for the University is 23% as in Table 6. This indicates that even with higher space occupancy rate, there are still some spaces that have yet to be fully utilized by the faculties. Only three faculties (Faculty A, Faculty H and Faculty L) reach more than 50% of space utilization rate and this indicates that the faculties spaces are at an optimum level of space usage. This also indicates that even if the occupancy rate is higher and complied with the guidelines, the utilization criteria is still important in determining the real usage of the available space within the faculties or campus. Based on this situation, the university management needs to decide on the best suitable approach for space allocation. With the low utilization rate for a certain faculty, it creates an opportunity for the management to think about the current usage and prepare for the intervention action that can improve the situation.

Table 6. Summary of Space Utilization

No	Faculty	Students Enrolment (2017)	Maximum Academic Capacity	Space Occupancy Rate % (2017)	Space Frequency Rate % (2017)	Space Utilization Rate % (2017)	Level of Optimum
1	Faculty A	1162	1010	115	69	79	Optimum
2	Faculty B	56	194	29	73	21	Not achieved
3	Faculty C	644	984	65	33	21	Not achieved
4	Faculty D	1751	2313	76	22	17	Not achieved
5	Faculty E	2260	2236	101	24	24	Not achieved
6	Faculty F	1841	1841	100	30	30	Not achieved
7	Faculty G	2291	2653	86	18	14	Not achieved
8	Faculty H	2042	788	259	36	93	Optimum
9	Faculty I	701	1346	52	55	29	Not achieved
10	Faculty J	600	1648	36	26	9	Not achieved
11	Faculty K	160	699	23	33	8	Not achieved
12	Faculty L	3522	2713	130	43	56	Optimum

No	Faculty	Students Enrolment (2017)	Maximum Academic Capacity	Space Occupancy Rate % (2017)	Space Frequency Rate % (2017)	Space Utilization Rate % (2017)	Level of Optimum
13	Faculty M	3040	2009	151	17	26	Not achieved
14	Faculty N	478	916	52	11	6	Not achieved
15	Faculty O	605	1000	61	17	10	Not achieved
16	Faculty P	1990	2576	77	21	16	Not achieved
17	Faculty Q	4725	4542	104	28	29	Not achieved
18	Faculty R	1769	2953	60	19	11	Not achieved
	Total	**29,637**	**32,421**	**82**	**28**	**23**	**Not achieved**

CONCLUSION

All academic space in a public university must be managed efficiently and effectively by the organization of University. To obtain space occupancy index, every university needs to conduct space audit in stages or when it is deemed necessary by the top management. The ability to measure every learning space capacity in relation to the total number of students and standard of space planning will produce the result of density for every space. The density will highlight the current space availability and also to be used for future space planning. The space occupancy index is one of the significant indicators to measure space usage besides capacity rate, frequency rate, and condition rate.

Based on the case study, it is indicated that, most of the learning space has achieved high density with the average occupancy index of more than 100%. However, there are still some faculties with the occupancy rate of below than 80%. This highlights the condition where some available spaces are yet to be fully utilized by the program and faculty. In terms of utilization rate, 15 faculties have not achieved more than 50% of utilization and the increasing number of students for future is possible and aligned with the strategic direction of the university. The result of the utilization index needs to be read together with frequency, occupancy, capacity and condition rate before any decision can be made for space strategic planning either to maximize existing space or propose new requirement space by new building construction. The research findings have shown that space management must be treated at strategic level, where academic spaces as main resources of the university. It must be well managed by the competent personnel and in Malaysia scenario Building Surveyor as professional that having ability to perform space management practices.

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CODIFICATION AND APPLICATION OF SEMI-LOOF ELEMENTS FOR COMPLEX STRUCTURES

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[2] *Institute of Ocean and Earth Sciences (IOES), University of Malaya, Malaysia*

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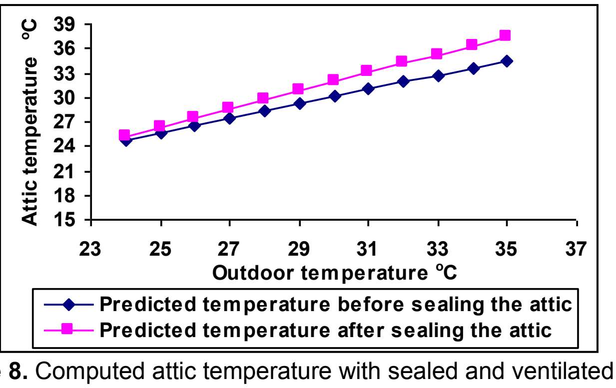

Figure 8. Computed attic temperature with sealed and ventilated attic

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Table 1. Recommended/Acceptable Physical water quality criteria

Parameter	Raw Water Quality	Drinking Water Quality
Total coliform (MPN/100ml)	500	0
Turbidity (NTU)	1000	5
Color (Hazen)	300	15
pH	5.5-9.0	6.5-9.0

(Source: Twort et al., 1985; MWA,1994)

Reference: Times New Roman, 11pt. Left indent 0.64 cm, first line left indent – 0.64 cm. **Reference should be cited in the text as follows:** "Berdahl and Bretz (1997) found…" or "(Bower et al., 1998)". References should be listed in alphabetical order, on separate sheets from the text. In the list of References, the titles of periodicals should be given in full, while for books should state the title, place of publication, name of publisher, and indication of edition.

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