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Sustainable Building and Infrastructure Systems : Our Future Today

Editors:
Abdul Naser Abdul Ghani
Md Azree Othuman Mydin
Noor Faisal Abas

Universiti Sains Malaysia
School of Housing, Building & Planning

PROCEEDINGS OF INTERNATIONAL BUILDING & INFRASTRUCTURE TECHNOLOGY CONFERENCE 2011

"Sustainable Building and Infrastructure Systems: Our Future Today"

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Abdul Naser Abdul Ghani
Md Azree Othuman Mydin
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Proceedings Editor's Foreword

Welcome to the International Building & Infrastructure Technology Conference 2011 in Penang, Malaysia.

The main objective of this conference is to provide a unique international platform and forum for academicians, industrial players, key trade and investment policy makers, and PhD students with a clear aspiration to bridge the gaps existing between the aforementioned members in engineering, science and technology sectors.

The submissions leading to the formation of these Proceedings are testament to the continual effort by the participants to produce the collective creative output which can only be achieved by bringing together all building and infrastructure technology disciplines in a unifying conference like the International Building & Infrastructure Technology Conference 2011.

The inclusion of presentations by local and international key industrial players offers a practical dimension to the progress of research in science and technology. Furthermore, the additional inputs from the professionals provide a perspective of reality to combined academic and practical approach to building and infrastructure technology research and development.

We would like to extend our appreciation to the contributors of these Proceedings and the presenters at the International Building & Infrastructure Technology Conference 2011 making the event a convention of thought-provoking and innovative ideas.

International Building & Infrastructure Technology Conference 2011
Ir. Dr. Abdul Naser Abdul Ghani
Sr. Dr. Azree Othuman Mydin
Dr. Noor Faisal Abas

About the Conference

The discipline of building and infrastructure technology is very related and important to the development industry. Building and infrastructures such as road, drainage, sewerage, water supply, power supply and communications are always part and parcel of any development works. In the 9th Malaysia Plan, the government has allocated more than RM 14 billion for infrastructure development and another almost RM 5 billion for infrastructure maintenances.

In order to achieve this goal we need the contributions of academicians, researchers, professionals and postgraduates to underpin the effort in advancing innovation in building and infrastructure technology. It is this factor that School of Housing, Building and Planning, Universiti Sains Malaysia gather for the Conference and by understanding the current phenomenon of sustainability development in building and infrastructure technology through research showcase and academic forum, together we encompass a shared vision, mutual understanding and trust in addressing the industry about the opportunities and challenges for growth in the future.

This conference will bring together experts and practitioners from local and overseas who work in infrastructure, building and construction fields. The participant and presenters will discuss about case studies, emerging technologies, and the business side of building infrastructure technology. This event will be the best platform to discuss new strategies for the coming Malaysia's RMK10 projects especially in terms of emerging technologies.

Objectives:

- Providing an opportunity for Malaysian and overseas academicians, researchers, professionals and postgraduates to present and challenge the cutting edge research projects that facilitates exchange of knowledge, acknowledges latest findings and stimulates innovative approaches.

- Offering a platform for Malaysian researchers and professionals to pursue their learning and research interests through the presentation of specific case studies that encourages inter-disciplinary discourse and provides opportunity for recognising the connection between innovation and commercialisation.

- Fostering a network for academicians, researchers, professionals and postgraduates through the formation of special interest groups in the field of building and infrastructure technology from Malaysia and overseas that enables research and education collaboration.

CONTENTS

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THE PERFORMANCE OF CONCRETE BEAMS REINFORCED WITH EMBEDDED CFRP PLATES

R.B. Ohu [1], M.S. Jaafar [2], J. Noorzaie [3], F. N. Aznieta [4] and A.H. Alwathaf [5]

[1,2,3,4] Civil Engineering Department, Universiti Putra Malaysia, Serdang, 43400, Malaysia

[5] Civil Engineering Department, Faculty of Engineering, Sana'a University, P.O. Box 12544, Sana'a, Yemen

[1]budan60@yahoo.com

ABSTRACT: FRP plates are known to be traditionally used for strengthening or retrofitting of structural elements, however it is proposed herein that treated FRP plates in place of FRP grids or even FRP bars could be a useful alternative technique in reinforcing structural elements with the added advantage of further reducing the overall weight of the structure together with its non-corrosive benefits. This proposed technique could also be adapted for use in section enlargement of beams. The present work is carried out to study the performance of this proposed technique and it showed good performance results in terms of ductility performance similar to conventional reinforced concrete with an increase in ultimate load capacity of more than 100% in comparison to conventional reinforced concrete beams.

Keywords: embedded, carbon fiber reinforced polymer plates, performance, failure mode, surface treatment

1. INTRODUCTION

The traditional use of fiber reinforced polymer plates/strips has been in the strengthening/retrofitting of concrete structures. There already exists a wide range of research articles that have looked into the behavior of fiber reinforced polymer plates/strips as externally bonded reinforcement and more recently as near surface mounted reinforcements [Chen et al., Hassan et al., Teng et al., Benjeddou et al., De Lorenzis and Teng, Mazzotti et al.]. One of the major conclusions and detriments from these research findings has been about the premature de-bonding of the plates from the concrete surface with an associated brittle mode of failure [Ritchie et al., Rahimi and Hutchinson]. This premature de-bonding occurs due to the inadequate bond between the FRP plate and the concrete. Rasheed et al. recently made a successful attempt to improve this by using transverse anchoring reinforcement to control this premature de-bonding. The result was a more ductile behavior due to the transverse strengthening and concrete confinement effects. In order to further eliminate or reduce the risk of premature de-bonding and to better improve on the bond between the FRP plate and the concrete leading to a more ductile structural response, a new reinforcing technique is herein proposed wherein CFRP plates are embedded within the concrete section. This proposed reinforcing technique is an adaptation which stems from the existing externally bonded reinforcements and the near-surface mounted methods with the aim of improving on the use of FRP plates in construction while simultaneously taking advantage of its most beneficial properties in terms of strength, weight, corrosion resistance and ease of application.

The proposed technique could serve dual purposes as both a reinforced concrete design concept or in the repair of structures (such as section enlargement). The proposed technique also has the advantage of further reducing congestion of reinforcement in a section as well as reducing the overall cost of reinforcement which will be gained in the long term due to reduced maintenance costs.

2. EXPERIMENTAL PROGRAM

Five small beams of length 750mm and cross-section 150 x 150mm were cast, cured and tested under a two point load as shown in Figure 1(a). The beams had an average concrete strength of 40MPa and were reinforced as shown in Figure 1. All beams were simply supported and loaded with specific incremental loads until failure. Beams denoted by RC (steel reinforced concrete beam) and EBR (externally bonded reinforced concrete) served as control specimens and were each reinforced with 4no.s 6mmØ mild steel rebar's with fy = 250N/mm^2. Three other beams were internally reinforced by embedding CFRP plates within the stirrups as shown in Figure 1(b). These beams were given denotations based on the treatment provided on each embedded CFRP plate which consisted of beam NE (No Epoxy), BE (Bottom Epoxy) and TE (Top Epoxy). 6mmØ mild steel stirrups (fy = 250N/mm^2) placed at 100 centres were used for all beam specimens.

The CFRP plate used for all the beams except the control beam RC were 100mm wide with a thickness of 1.2mm and a cross-sectional area of 120mm^2. The E-modulus is 165,000N/mm^2 and the tensile strength is 2800N/mm^2 according to tests done by the manufacturer (Sika Kimia Sdn. Bhd. Malaysia). The lengths of the CFRP plates used for the embedded beams were 720mm while the length for the EBR was 400mm according to design procedures (ACI 440).

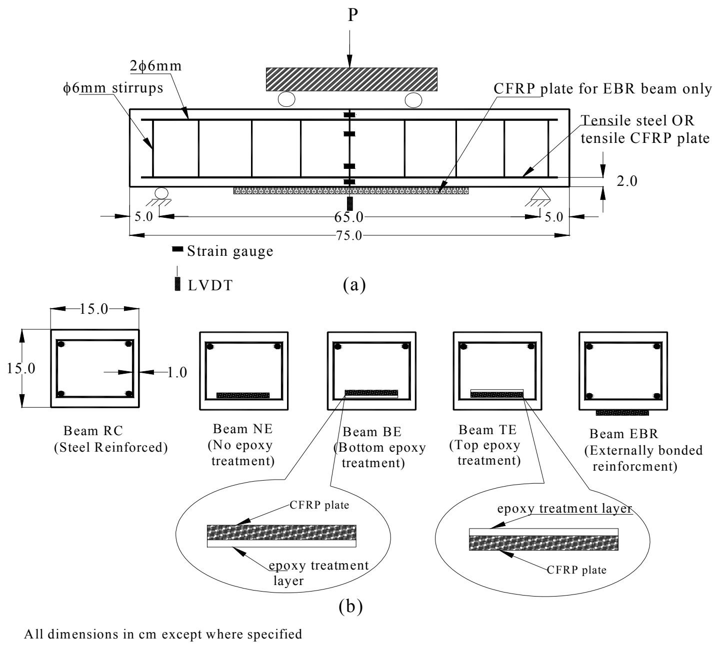


Figure1. *a) Test setup; b) Reinforcement details*

2.1 Embedded CFRP beams

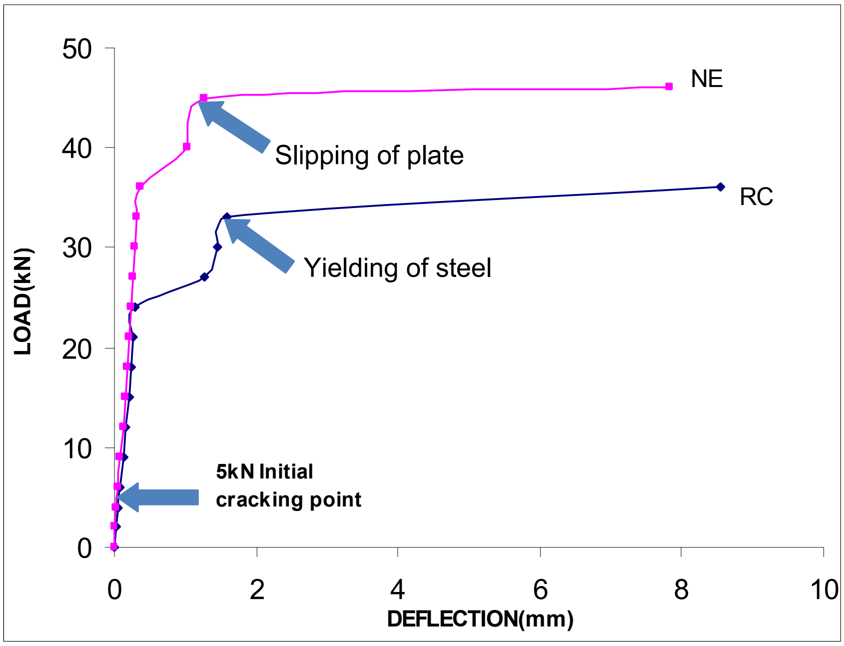
In order to increase the roughness of the plate surface and hence enhance the bond between the concrete and the FRP plate, a thin layer of epoxy was applied to one side surface as depicted in Figure 1(b). In beam BE the side face with epoxy treatment was placed downwards towards the concrete cover while in beam TE the epoxy treated face was placed upwards in the section of the beam. Beam NE had no plate surface treatment. After treatment, the CFRP plate was placed within the reinforcement cage after which casting of the concrete was carried out in three batches followed by proper vibration. All beams were cured for 28days before testing. Beam EBR was externally reinforced (using epoxy) with a CFRP plate after curing of the beam was completed according to the manufacturer's specifications (Sika Kimia Sdn. Bhd. Malaysia) before testing. The instrumentation included strain gages placed at the mid-span for all beams and one LVDT connected to the TDS-530 data logger. Incremental load was applied by using a manual hydraulic machine. Observations made on all beams included deflections, concrete strains, cracking loads, crack patterns, ultimate loads and failure mechanisms. All beams were loaded until failure.

3. RESULTS AND DISCUSSIONS

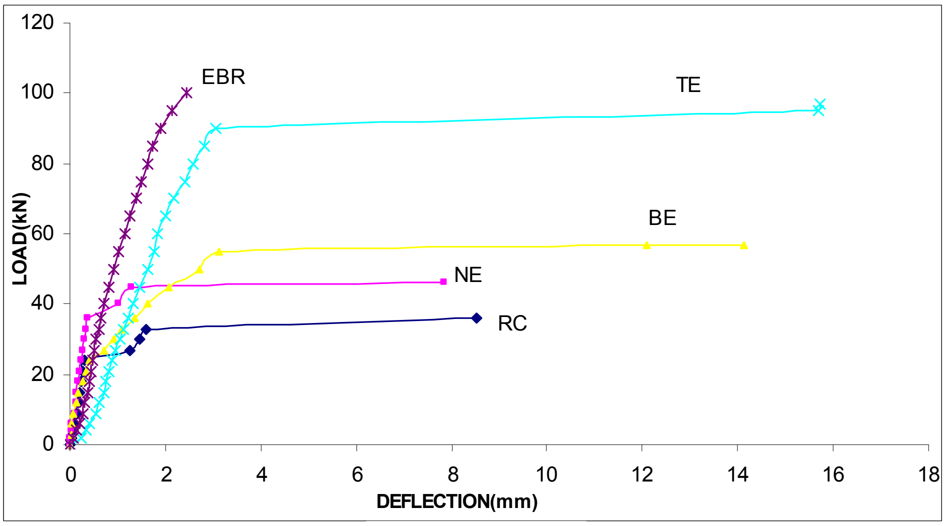
3.1 Load-Deflection Response

The load-deflection curves for all beams are shown in Figure 2 (a) and (b). In Figure 2(a), it was observed that both beams RC and NE had similar load-deflection responses throughout the loading process. Both beams initially had steep slopes which correspond to the uncracked section of the beams wherein deflection is proportional to the load applied. It can however be seen that beam NE exhibited a relatively steeper slope than beam RC at this initial elastic stage with a corresponding delay in cracking as shown in Figure 2(a). This response of beam NE is attributed to the higher tensile properties of the CFRP plate over the steel rebar. After cracking and further load application the difference in stiffness between beam NE and RC was about 18% (the stiffness here refers to the load per unit deflection). A similar pattern of behavior was also observed between beams RC and NE mainly due to the untreated CFRP plate which led to the bond between the concrete and the plate being due to frictional forces only, which is similar to the conventional yielding of mild steel used in beam RC. This is unlike the bond being due to both frictional forces and mechanical interlock of grooved surfaces as is already established for high yield steel. However as beam NE approached failure, the beam deflection also increased significantly and at failure the difference between beam RC and NE was merely 8%. This behavior is thus similar to the load-deflection response of beams reinforced with CFRP bars (Rafi et al. 2008; Benmokrane et al. 1996; Nanni 1993) wherein a reduced stiffness behavior was observed in comparison to beams reinforced with conventional steel mainly due to the lower elastic modulus of the CFRP bar/plate as the case may be.

In Figure 2(b) all beams behaved in a similar fashion before cracking becomes wider and slipping of the plate or yielding of the steel occurs. This can be observed by the initial linear slope of the load-deflection graph. After cracking, the next part of the beams response gives an insight as to the quality of bond that exists. In this segment the rate of increase in deflection rapidly increases with a corresponding decrease in beam stiffness. As expected beam EBR exhibited a predominantly linear response unlike the other beams. The load-deflection behavior of beam EBR did not show the same ductile trend as seen in the other beams.



(a) Beams RC and NE



(b) All Beams

Figure 2. *Mid-span deflections*

The treatment of the plate with epoxy in beams BE and TE resulted in a higher stiffness performance after cracking as shown in Figure 5. From the same figure it can be seen that the load-deflection curve for beam EBR is predominantly linear until failure in comparison to all the other beams especially the beams with embedded CFRP plates which exhibited a more ductile trend similar to beam RC. At the failure load of beam NE, beams BE, TE recorded higher deflection values of 73%, 81% more than NE respectively. This could be attributed to the treatment of the plates in beams BE and TE which thus increased the bond between the plate and concrete thereby leading to a satisfactory load-deflection response and thus improved ductility from the start of the loading process until cracking occurred. However, beam TE showed a slightly stiffer response compared to beam BE at service mainly due to the position of the treated face of the plate which resulted in further increased bond characteristics due to higher effective depth by bond. Epoxy at the top of the plate ensured that there was an increased distribution of stresses on the plate together with increased bonding mechanism between the plate and concrete thereby leading to higher load-deflection response in comparison to the other beams except beam EBR.

3.2 Cracking and Ultimate loads

The cracking loads used herein refer to the load at which the first cracks become visible and wider during the loading process and is shown for all tested beams in Table 1. From the results, the beams with embedded CFRP plates; NE, BE and TE all showed an increase in cracking loads of 50%, 13% and 50% respectively in comparison to beam RC. This indicates that beams with embedded CFRP exhibited a good performance in terms of delaying the onset of cracking to higher loads than for beams reinforced with steel.

Table 1. *Cracking, Ultimate loads and failure modes*

Beam	$^*P_{cr}$ (kN)	P_u (kN)	Δ at P_u (mm)	Δ at initiation of yielding/slipping (mm)	Δ at service (mm) At 35% P_u	Failure modes
R.C	24	36	8.54	1.59	0.17	Flexural failure
N.E	36	46	7.84	1.28	0.15	Flexural failure associated with plate slip
B.E	27	57	14.13	3.13	0.30	Flexural Shear failure
T.E	36	97	15.73	3.04	1.12	Shear failure
EBR	57	100	2.43	1.90	0.65	Shear failure associated with plate debonding

**Refers to the first visible wide crack*

The crack patterns for all test specimens are shown in Figure 3. The patterns of beams RC and NE mainly consisted of flexural cracks originating from the tension face of the beam at mid-span within the maximum moment region. These cracks propagated upwards towards the compression zone of the beam. In addition to the flexural cracks, beams BE and TE also exhibited shear crack patterns as the load increased. In both beams these shear cracks were not only diagonal formations but also consisted of large diagonal shear cracks along the path line of the embedded CFRP plate. As the load increased, the flexural and shear cracks increased in width and depth forming a definite path eventually leading to failure of the beam.

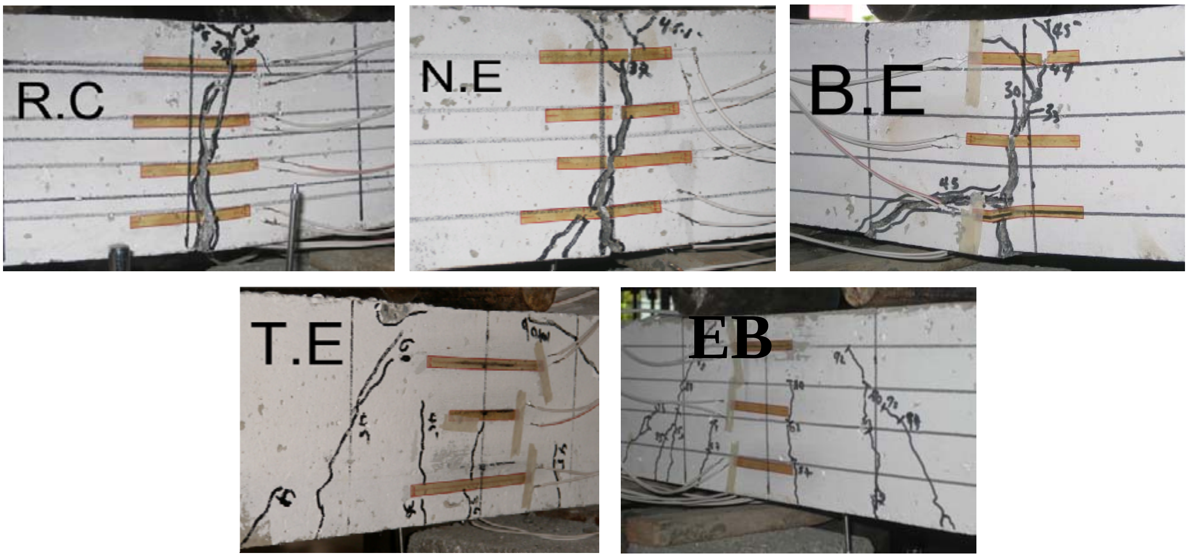


Figure 3. *Crack pattern for all test specimens*

After the initial flexural cracks in beam TE, it was observed that after about 46kN shear cracks began to form and at about 70% (68kN) the failure load of beam TE a large diagonal shear crack formed which grew very wide as the failure of the beam approached. In comparison to the other beams except EBR, it was observed that the crack propagation for beam TE was much slower. It was further observed that both beams TE and EBR exhibited similar crack patterns of both flexural and shear cracks throughout the loading process until failure. This therefore gives an indication of the bond quality between the treated embedded plate and the surrounding concrete.

Results of the ultimate loads shown in Table 1 show that the beams with treated CFRP plates; BE and TE performed better than the beam with the untreated CFRP plate NE and were therefore capable of attaining higher ultimate load capacities. Beam TE showed an increase in ultimate load capacity of 70% more than BE and 110% more than NE. The difference in ultimate load capacities between the beams with embedded CFRP plates could be attributed to the location of the treatment. As long as there is adequate bond action this technique has the potential to attain higher ultimate load capacities. Also considering this technique from the surface area point of view, it is good to mention that the embedded CFRP plates which have a wider surface area and thereby increased contact with the surrounding concrete led to the high ultimate loads observed.

4. CONCLUSIONS

The following conclusions can be drawn based on this study;

1. The ductility trend of beams with embedded CFRP plates without treatment is similar to steel reinforced concrete beams but with an increase in ultimate load of 28% more than steel reinforced concrete beams. Although beams with treated embedded CFRP plates showed improved bond characteristics resulting in higher ultimate loads, they however, exhibited reduced stiffness properties with an average difference of 68% at service condition in comparison to the beam without plate treatment.

2. The use of embedded CFRP plates shows improved performance over plain RC beams however, if the plate is untreated then premature slip of the plate occurs resulting in early failure of the beam unlike beams that were treated with epoxy. This is because the bond that occurs is mainly due to frictional forces. Therefore the treatment of the embedded CFRP plates helps to improve the bond characteristics and thereby result in higher ultimate load capacities of up to 24% and 110% respectively in comparison to untreated embedded CFRP plates.

3. The improved performance shown in beam TE over beam BE could be attributed to the location of the treated surface area. In beam TE, the treated surface is within the effective concrete area of the section which therefore led to a better overall performance. While in beam BE the treated surface was in the weak tensile concrete area further weakened due to cracking of the section thereby resulting in a faster initiation of slip and hence failure in comparison to beam TE. At failure, the difference in deflection between both beams was on the average about 10% with BE being less than beam TE.

4. Flexural-shear mode type of failure was observed in the beams with embedded CFRP plates treated with epoxy associated with crushing of the concrete in compression. The shear failure mode was however more predominant in beam TE and led to the failure of the beam and plate rupture.

5. Similar to using FRP rebars, the use of FRP plates resulted in concrete crushing in compression at beam failure especially when the FRP plate is treated to increase the surface roughness which is similar to results of studies carried out by other researchers [Rafi et al.].

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TRAFFIC ACCIDENTS ANALYSIS IN LIBYA

Hussin A.M. Yahia[1] and Amiruddin Ismail [2]

[1, 2]Sustainable Urban Transport Research Center (SUTRA), Department of Civil and Structural Engineering, Faculty of Engineering and Built Environment, Universiti Kebangsaan Malaysia, 43600 UKM Bangi, Selangor D.E., Malaysia

[1]Husseinukm@yahoo.com , [2]abim@eng.ukm.my

ABSTRACT: Road traffic accidents is one of the most important problems facing developing countries, More than 1.2 million people die around the world in traffic accidents, and between 20 to 50 million people are injured each year around the world, In the World Health Organization report issued in 2008, revealed that 50 thousand people died in Libya On public roads during the period 1969 - 2009, the population in of Libya 5.125 million (Statistics 2000) and area 1,760,000 square kilometres. Libya suffers from concentration of population in major cities, and this in turn led to congestion and the number increase of accidents inside the cities. The purpose of this study is the analysis of road accidents in 6 major cities recorded the highest proportion of accidents in recent years and knowledge of the most important factors and solutions that can reduce from traffic accidents.

Keywords: road accidents, population.

1. INTRODUCTION

Human societies began long ago in the payment of tax advances that seeks to running fast behind him driving her self in the direction of all the forces of its economic or social and among the most what you pay these communities after by the so-called effects of traffic accidents and their implications for psychological, social and economic community, and Libyan society and one of the communities taking into growth and progress since the discovery of oil by as the number of motor vehicles used in Libya of about 18 thousand a vehicles in 1960 to about 101 thousand vehicles in 1970, then increased to about 265 thousand vehicles in 1975. In 2008 about 1524429 cars, it was a result of this massive increase in motor vehicles is the high rates of traffic accidents and increase the cost of these incidents, humanitarian, economic and resulted in many social problems, which include the surviving spouse's families and children lost their parents. The Road Statistics Review of Libya M. O. I. (1996) we find that the proportion of deaths of young people was caused by traffic accidents , and road traffic accidents in Libya is considered high if you compare with another countries , In (1990), the population was 3,821 million and number of accidents 7,847, in (2000) the population was 5,125 million and number of accidents was 10,667 , Increased incidents in recent years was in (2008) 18662 accident. Fatal accidents represent not only tragic family losses but also serious economic losses to the community in respect of their education and training. Property damage from traffic accidents cost the Libyan economy 8 million $ (10 million LD) annually (Road Accident Statistics, Libya, 2001). It is not only mortality that has to be considered, but also the temporary and permanent incapacity resulting from road traffic accidents. In addition to the pain and suffering caused and the tragedy of death or permanent disability, serious economic losses to the community arise from road traffic accidents.

2. THE ARE OF STUDY

Libya is an Arab country, located in the North Africa continent and in the south coast of the Mediterranean Sea. It is bounded by the Egypt on the east, Sudan on the southeast, Chad on the south, Niger on the south-west, Algeria on the west and Tunisia on the north-west. In this study, we chose 6 major cities in terms of congestion, the area, population's density and the proportion of accidents are Tripoli, Benghazi, Sirt, Al margheb, Al joufra and Misurata.

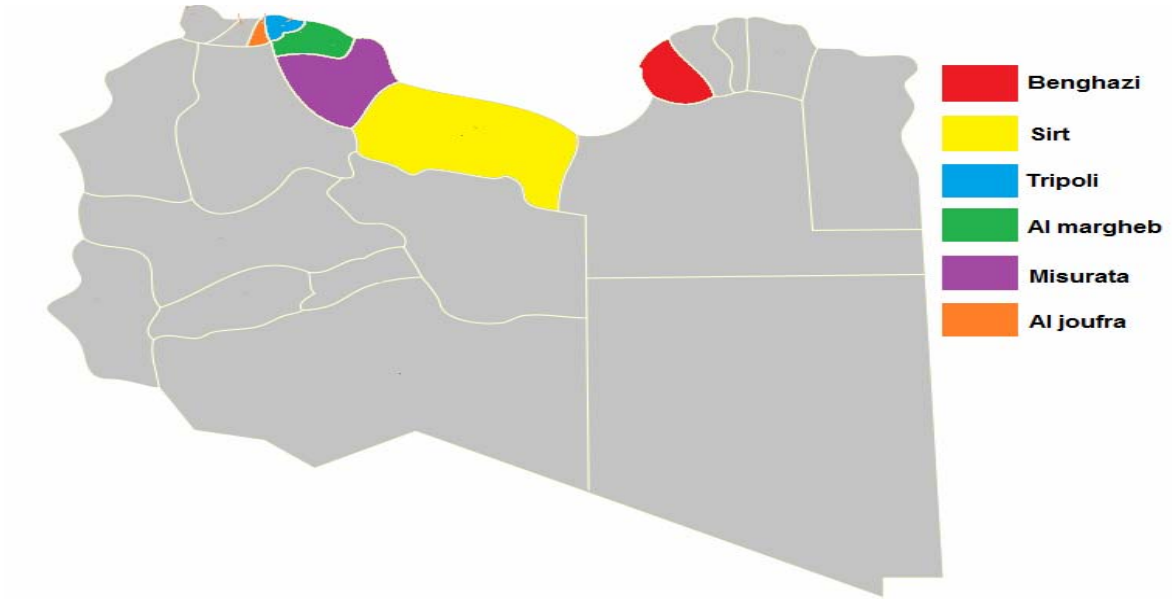

***Figure 1**. The location of states in Libyan.*

3. MATERIALS AND METHODS

The motor vehicle accidents statistics used in this study were taken from General Administration of Traffic, The General People's Committee of General Security and from Yearly Statistical Report and the Annual Reports of the Directorate of Traffic (Directorate of Traffic Annual Reports 1990 to 2000), the study was based on the collection of data about all fatal motor vehicle accidents that occurred In all Libyan cities during the period 1 January 1970 to 31 December 2005 and we in this study will focus on the 6 major cities, Tripoli, Benghazi, Sirt, Al margheb, Al joufr and Misurata , and also the annual statistical report contains information such as the number of registered vehicles, number and nature of accidents, causes of road accidents, number of fatalities and casualties, age and gender of victims. Additional data were obtained from various sources, including the Ministry of Health, Health Statistics Annual Report for the period 1990 to 2008 (Ministry of Health Annual Reports 1990 and 2000) .

4. ROAD TRAFFIC ACCIDENTS IN LIBYA

Traffic accidents increased dramatically in Libya and the deaths will reach 25 cases per day in the coming years if there is no real measures as duplication , separate roads ,the and providing the means for road safety ,lighting control on the quality of vehicles and the introduction of high import for Cars from the European countries and Asian ,and the application of traffic law strictly , and the total number of traffic accidents and injuries and the victims is increasing continuously as show in figure (2). World Health Organization (WHO) 2008 has stated that one of the most serious losses of people lives in Libya comes from car and road accidents. Statistics have shown a very clear increase in road deaths number, Injuries and the accidents during the last ten years. There is also a decline in the number of injuries, accidents and the death during the years 1992-1999. This decline can be explained by the slowdown in traffic volumes throughout the cities streets in Libya as a result of UN embargo to Libya and the relative economic recession which took place during these particular years.

5. RESULTS

Table 1. *The latest vehicles statistic until 2009*

Type of Vehical	2005	2006	2007	2008	2009
Motorcycles	559	660	784	873	1040
Private car	957226	1106894	1343422	1525429	1703362
Truck	253674	289718	345099	367824	398080
Public taxi	54307	61360	72827	77320	80967
Trailer truck	21617	24288	30874	36790	44786
Traction lorry	11848	13193	17204	24334	32838
Tractor	2072	2165	2723	2983	25144
Winches and automobiles	9227	10081	13600	17126	25144
Total	1310530	1508359	1826533	2052679	2289763

Course: *General Administration of Traffic and Authorizations (GATA)*

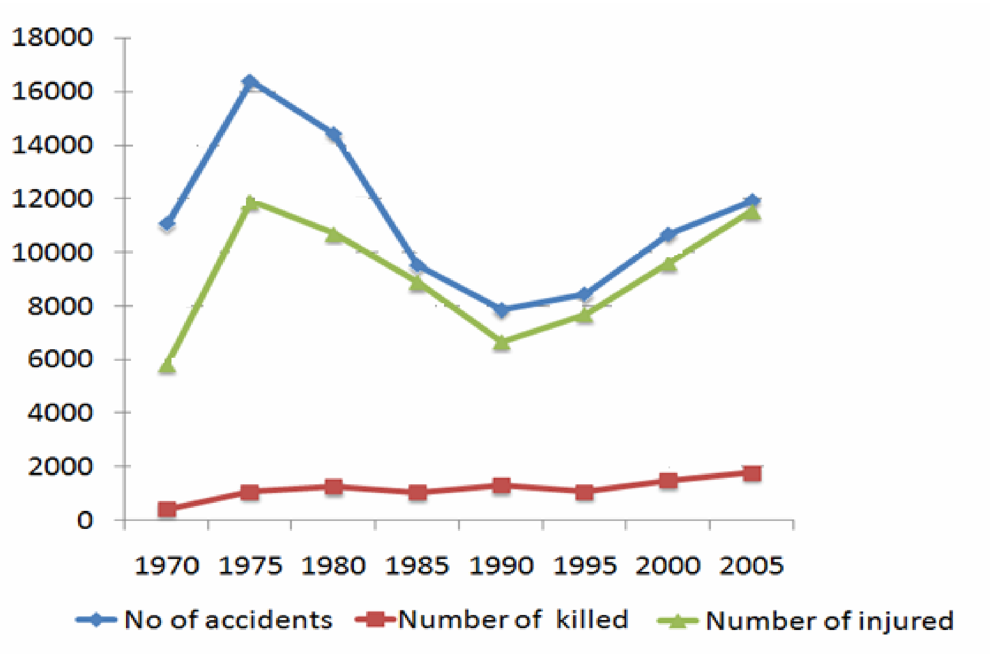

Figure 2. *Traffic accidents (1970-2005)*

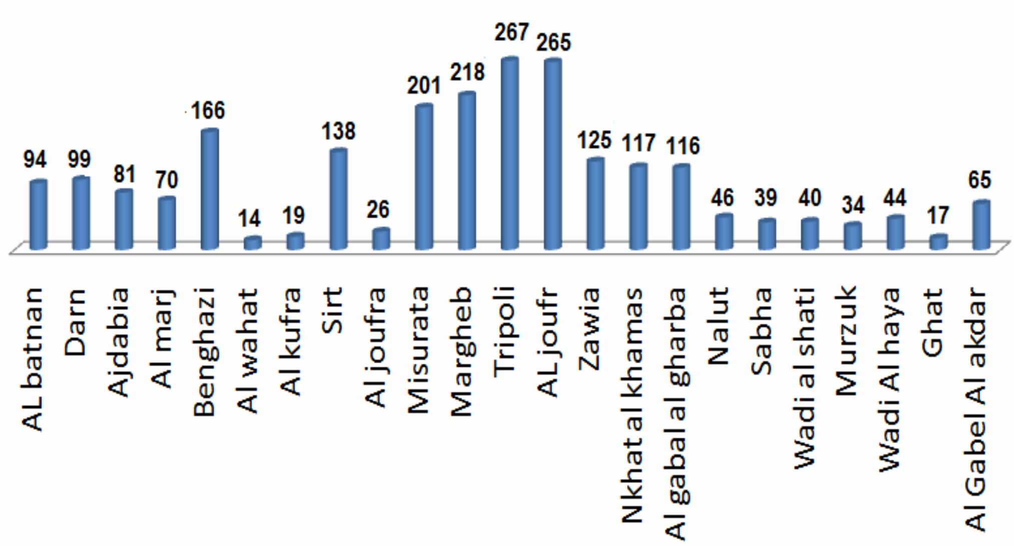

Figure 3*. Traffic accidents in 2009*

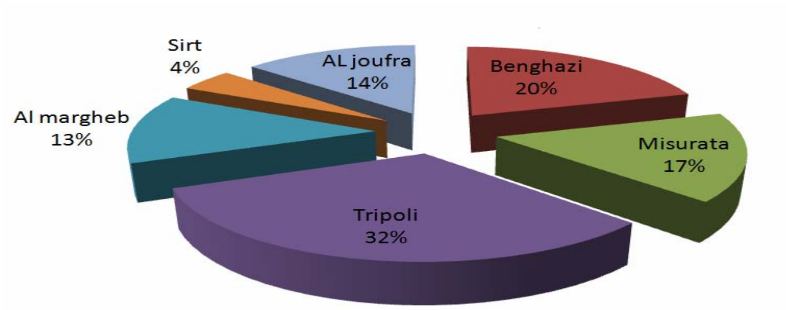

***Figure 4**. Percentage of population total*

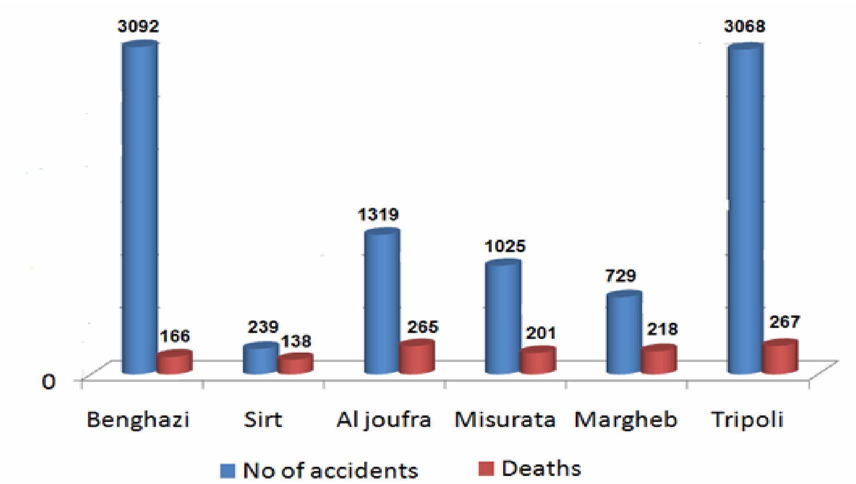

***Figure 5.** The relationship between the number of accidents and deaths*

***Table 2**. Distribution of Libya and the total population - area (km2) population density*

States	**Area Km2**	**%**	**Libyan of population**			**Grand Total**		
			Population	**%**	**Population Density**	**Population**	**%**	**Population Density**
Benghazi	11372	0.86	622148	11.74	54.71	674951	11.93	95.35
Misurata	29172	1.74	511628	9.66	17.54	543129	9.60	18.62
Tripoli	835	0.05	997065	18.82	1194.09	1063571	18.80	1273.74
Almargheb	6796	0.41	410187	7.74	60.36	427886	7.56	62.96
Sirt	86399	5.15	131786	2.94	1.53	141495	2.50	1.64
Aljoufra	2666	0.16	422999	7.98	158.66	451175	7.97	169.23

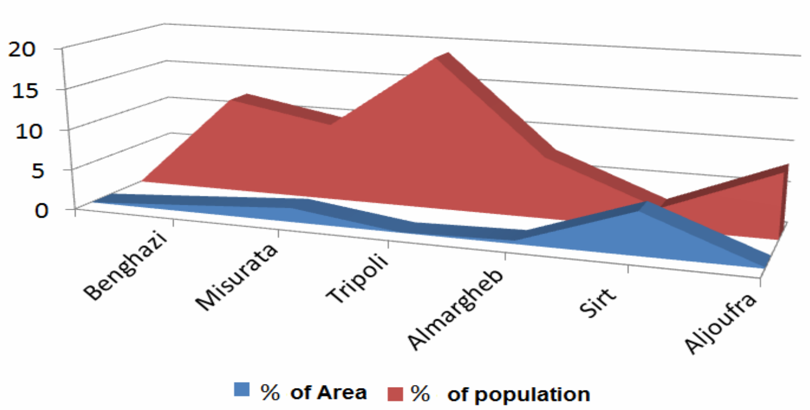

***Figure 6.** The relationship between of % of area and % of population*

Traffic accidents types of in libya

According to statistics, and reports of traffic accidents in Libya, Car accidents are classified into 6 classes, vehicle – vehicle, vehicle-fixed object, vehicle – pedestrian, overturning - vehicle, out-of-control, and other types of accidents

Causes of traffic accidents in libya:

The causes of traffic accidents in Libya can be divided into specific and general causes:

A. General causes

a. A large increase in the number of vehicles and population in major cities and expansion of road networks within and between far cities.

b. National development projects which require the development of supporting transport systems in major cities in Libya for example (Tripoli, Benghazi).

c. Increased number of expatriates from different countries with different habits and culture who are unfamiliar with local driving.

d. Concentration of population in major cities in turn led to the overcrowding and increase in accidents number.

B. Specific causes

a. Most accidents occurred as a result of driver negligence and error.

b. Over 50% of the traffic accidents are due to excess speed and using mobile phones during driving and violation of signals at intersections.

c. Road safety and vehicle condition contribute in reducing from traffic accidents, particularly those on open roads.

Therefore errors and negligence of drivers contribute significantly to traffic accidents throughout Libya.

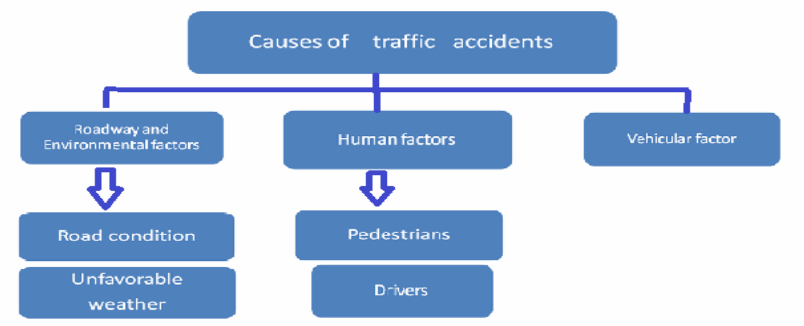

***Figure 8.**, Causes of traffic accidents in Libya*

6. DISCUSSIONS

Road traffic accidents are one of the leading causes of death in Libya , a review of traffic accident and their resulting casualties in Libya between 1970and 2005 and review of police records, we find that traffic accidents have increased dramatically, as show figure (2), we find that the population increases progressively with the passage of years, but traffic accidents registered an increase in the mid-seventies and then declined because of the economic embargo on Libya by the United Nations (1992-2002) , After the lifting of economic blockade increased the number of cars as well as the number of incidents and recorded in 2008, 13352 incident and 2332 death (Statistics 2009) , Injuries and the number of incidents grow quickly and increase in the number of cars contributed to the increase in the number of accidents ,the increase in population and all the way also led to an increase in the number of usage of vehicles on the road have reached in 2006 to 1310530 vehicles while the population was 5212000 million. The ratio of vehicles to population is 3.97 people per vehicle. In table(2) , If we show at Figure (1) we find that all the cities that took place in the study are located on the Sea beach , as a result of the more populous cities are located on the sea beach where the weather is moderate , from through statistics, we find the city Tripoli is the least in terms of area 835 km2, but it is more in terms of population and density of population, population density rate 1273.74 persons in km2 ,while the city of Sirt , the most area between the cities of the six is area 86399 km2 and least population density 1.64 persons in km2 , city following the Tripoli city , in terms of population density is Aljoufra because the Aljoufra is an extension to the city of Tripoli , this diversity in the distribution of the population on the coastal cities has helped to increase vehicles on the coastal road (Amsaad ,Ras Igdir) , in order to focus density residential areas on the coastal cities. Libya as well as space for large and long distances between cities and in the absence of good public transport. Many factors influenced the movement of road traffic in Libya the absence of public transport between cities, as well as the lack of trains or the good transport inside cities centre which caused the increasing of number of flights between cities, this led to increase the number of cars, which helped to increase the number of accidents in the absence of good transport system.

7. RECOMMENDATION

Road traffic accidents can not be removed entirely, but by the efforts of citizens, traffic police, engineers and the governments can reduce road accidents, the following is a set of recommendations which could be considered as remedies to reduce the effect of road traffic accidents.

a. The government should improve general public transportation among cities and encourage people to use and regulate the import of foreign auto.

b. Intensify the efforts of traffic police should be rising in the days before the end of the week (i.e., Wednesday and Thursday) because the social visits and trips go up at these days.

c. All media should be utilized to increase the traffic awareness among people and drivers and the dangers of traffic accidents.

d. Ambulances should be equipped with medical equipment and respiratory equipment at all times and speed in the event of a traffic accident because the time is an important element in reducing the severity of Accidents.

e. Speed is one of the main reasons for accidents in Libya, so must apply the law to those who drive their car at high speed

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SEISMIC RISK ASSESSMENT OF BUILDINGS IN KOLLAM INDIA

Narayanan Sambu Potty[1] and Sirajuddin, M[2]

[1]Department of Civil Engineering, Universiti Teknologi Petronas, Malaysia.

[2]Department of Civil Engineering, TKM College of Engineering, Kollam, India

[1]narayanan_sambu@petronas.com.my , [2]msiraj23@rediffmail.com

ABSTRACT: Many earthquakes (EQ) were recorded from coastal margins of Indian peninsula during the last 200 years. The vulnerability of Kerala in India to EQ was highlighted at a meeting of Government and NGOs held recently in association with United Nations Development Fund (UNDP). The entire state is in Zone III, a moderate risk zone. Many cities deserved more attention owing to high concentrations of population. While an EQ cannot be prevented, the damage to life and property can be minimized if effective steps are taken. Each area differs in terms of climate, culture, methods of construction and living standards. Materials used also differ. A localized survey can find out the methods and materials for construction, general pattern of the structures etc. This will enable the identification of damage prone structures during a seismic event. Suitable retrofit measures can also be planned. This study carries out such as survey of Kollam district in Kerala, India. The data was analysed and suitable retrofit measures have been suggested.

Keywords: earthquake, seismic, risk assessment, damage prone structure

1. INTRODUCTION

The Indian standards have suggestions for seismic resistive measures. For assessing expected seismic performance of existing load bearing masonry wall buildings one approach is to compare the safety provisions in the building code IS 4326 with the actual condition of building. Where it complies with code, it will be considered safe and acceptable. When deficient, it will be considered as weak and damageable. It will require up gradation or strengthening or retrofitting (Arya and Agarwal). The pattern of construction and construction methods varies from place to place. Due to this diversity, a localized survey is the only method to obtain the exact details of the structure, methods of construction and materials used. Kollam was selected for survey due to its proximity to the authors. This study is significant since Kerala was elevated from zone II to zone III. Local tremors have been reported in various areas of Kerala in the near past (Bhattacharya and Dattatrayam, 2002). There exists no rule regarding the magnitude of earthquake (EQ) up to which a particular structure is safe. Only predictions and suggestions can be made about how many structures are to be retrofitted to withstand seismic shocks. Kollam has different types of soil varying from rocky, sandy to reclaimed soil type. Data collected will have a representation from all types of soil. In this study the general pattern of structures is determined and the number of structures that can with stand moderate tremors are identified.

2. LITERATURE REVIEW

The peninsular shield was long held to be non-seismic. The Peninsular coast is now observed to be vulnerable to intermittent seismicity (Banerji et al., 2001). Indian Meteorological Department (IMD) has precise data on relatively recent EQs and their after effects. IMD provides information on seismic zones and seismicity map for the period from 1505 to October, 2005 showing EQs with M>5.0. In Kerala, EQ are of Magnitude 5 to 6.9. Since authentic historical records of seismicity along the peninsula cost are virtually unavailable, the likely recurrence interval between shakes in such sectors cannot be gauged. Regional seismicity since historic times gives hints about major faults in this part of south India. The region has witnessed several slight to moderate magnitude EQs in the past. The most significant one is the Coimbatore EQ (*M* 6.0) of 8 February 1900. Two EQs occurred in 2000 and 2001 (M 5.0 and 4.8 respectively) (Bhattacharya and Dattatrayam, 2002). No major damage was reported;

however cracks occurred in some buildings in the epicentre area. The chances and types of damages that are possible during an EQ in Kerala are discussed in Rajendran et.al (2009). The vulnerability of Kerala was highlighted at a recent meeting of Government, NGOs and UNDP.

3. OBJECTIVES

- To conduct a localized survey on the materials and methods of construction, geometry and general pattern of the structures etc in Kollam (no such survey has been reported in literature)
- To identify the buildings that require retrofitting and to compile suggestions for retrofit of these structures.

4. METHODOLOGY

Kollam Corporation has 52 wards with estimated one lakh legal structures. The different areas of the city have nothing in common considering the geology. The coastal areas have sandy soil while the eastern zones have laterite soil. Some regions have marshy soil also. After discussions and field verifications six soil categories was included. Under “type of structures”, reinforced walls, concrete slab over steel truss, timber structures and fully steel houses were eliminated as they were not used locally. In slums, mud bricks were commonly used. Framed, reinforced masonry, ordinary masonry and mortar free construction were included. The number of floors, floor areas, height of walls etc was included. The use of the building was also considered as the number of causalities during an EQ differs in various types of structures. Special consideration was given to strategically important buildings. To ascertain the general strength of masonry, type of masonry, mortar composition and the plastering was included separately. Most of the structures being masonry, wall shear failure will be high during an EQ. Hence the longest wall length was considered. The size and positions of openings in bearing wall, the most important criteria proposed by IS codes was included. Similarly the pier width between consecutive openings, distance of the first opening from inside corner of outside wall etc were included. Laterite, wire cut and country burnt bricks; hollow cement block, solid cement block, random rubble masonry, Interlocking bricks, wooden planks etc were included “materials used building blocks” category. The roof type covers RCC- flat, RCC-sloped, Tiled, AC sheet, Tin sheet and Thatched. Filler slab was also included but only a few houses were found in that category. To check the overall stability of the structures, especially at corners, the provision for lintel and plinth beam all around the building were noted. The symmetry and age of structure play an important role during an EQ. For sub structure, the points noted were the presence of plain cement concrete at the bottom of trench and type of foundation. The foundation types include random rubble, isolated footing, strip footing, raft foundation, brick foundation, pile foundation etc. The average number of occupants was noted irrespective of the type and use of the structure. The provision of cellar parking spaces, high weight RCC overhead tanks etc. was also noted. The presence of high rise towers adjacent to a selected structure was considered. The aim was to collect a minimum of 250 data from each ward. Care was taken to select various types of structures from each ward so as to get a clear cross section of structure types in the locality. The survey succeeded in collecting data in good quality and quantity. The accuracy of this study would increase with the number of data collected. Time constraints forced to reduce the number to around 200 from each ward. Interaction with incumbents gave more details about the structure. Details were available from the City Corporation office. Those found correct in all respects were included. Problems faced during field survey included uncertainty (1) whether plain cement concrete was placed at the bottom of trench for the substructure (2) presence of plinth beam and (3) type of cement mortar used for construction of super structure. As many structures are plastered, there was no way to find the composition of mortar used. Same problem was faced with plastered walls. Application of wall putty made the walls hard and the mortar combination was not known and difficult to find. The type of roof covering also created some problems. Many multi storied structures had sloped roof at the top. The doubt was where to include the data, as flat slab or as sloped roof. It was decided to include in sloped roof segment. The questionnaire had 30 questions. IS 1893, IS 113828 and IS 4326 were used for preparing the questionnaire and also assessing the areas of a structure that is more prone to seismic damage. It helped in pinpointing the parts of a structure that has to be strengthened to attain seismic resistivity.

5. RESULTS OF SURVEY AND ANALYSIS

A total of 6817 data were collected. Those having not enough details or ambiguity were removed, resulting in a total of 6800 data for analysis. In Kollam city, residential buildings constituted 64.2%, Commercial buildings 19.2%, Government buildings 3.5%, Public buildings 4%, Educational buildings 5.1% and hospitals 4%. Kollam, once an industrial area has now only 1% industrial buildings. Educational standards and facilities are high and hence educational buildings contribute to 5.1%.

General Pattern of Structures in Kollam: It is seen that residential buildings are increasing in numbers, compared with commercial ones. They are constructed largely by individuals, with their own likes and knowledge; seem to have no seismic provisions. The general method of making structure strong by adding "more cement, more steel" creates an over reinforced structure only. This is not a good practice since the structure will not give enough warning before collapse. In this study more attention was given to residential structures. Norms and standards for residential structures up to two floors were considered.

Detailed analysis of residential structures (RS): For RS, 17.63% does not satisfy the requirement of minimum distance from the inside corner of the outside wall. IS 13828 (1993) requirement is 23 cm. A minimum pier width of 45 cm between consecutive openings (IS 13828,1993) was not satisfied by 5.88% of the total buildings. The ratio of width of the opening to the length of the wall was limited to 0.40 to 0.46 by IS code. It was modified as no attics and mezzanine floors are seen in Kollam. The value was interpolated to 0.45. On this, a total of 77.77% of buildings satisfies this requirement. Majority of RS satisfy the three important requirements proposed by IS codes. Even though these code provisions draw less attention of the builders, accidently or fortunately majority have satisfied the requirement. All the surveyed buildings using masonry had cement mortar proportions not less than 1:5. To attain more strength for structures many house owners spend lavishly on cement and sand, while the code proposed a minimum of 1:6. Only 10.45% buildings do not have all-round lintels. This can be accepted to a certain level as this percentage contains buildings that are mortar free, thatched sheds etc. It was noticed that some of RS, that too multistoried ones, did not have all-round lintels. In some places lintels are as thin as a line with a very small percentage of reinforcement. It has been found that these structures were constructed not by individuals, but by contractors. Surely the contractor is exploiting the owner. 27.45% of RS were found to be non symmetric. Symmetry plays an important role during EQ as stability is directly related to symmetry. If the structure is symmetric, the additional shear and moment that occurs at the time of quake gets distributed uniformly over the structure. Otherwise it will create an unbalanced effect and the structure concedes easily. Symmetry has not received sufficient importance in RS due to the reasons like aesthetics, better cross ventilation, space constraints, non availability of enough land, extension works etc. Regarding subsurface details; 86.92% of the residential buildings rest on soft soil strata of which 93.23% rests on sandy soil, 4.5% rests on clayey strata, 1.3% on marshy strata and 0.97% on reclaimed land. It is good to see that all RS constructed on marshy and reclaimed strata were provided with plinth beams and plain cement concrete at base. 30.97% of the buildings in the sandy soil have plinth beams. This is concern because, at places where sandy soil was supported by some other weak soil strata, cracks were seen on structures. These areas require special attention since a mild quake can topple these structures. But another surprising fact is that 75.18% of buildings in sandy soil have got a PCC bed beneath the foundation. The genuiness of this is bit difficult to prove. Percentage wise distribution of various types of masonry structures showed that ordinary masonry structures constituted (90.32%), framed structures (5.66%), mortar free construction (0.84%) and reinforced masonry (3.18%). On surface finishes, 92.81% had plastered walls. This makes good sense as during an EQ, it holds the walls and transmits the forces. Of the unplastered buildings, 36.36% are non-masonry and 44.33% are random rubble construction. 19.31% of masonry buildings are left unplastered. Non plastered walls exist not due to aesthetics or non availability of raw materials or labour. Many owners admit that it is due to the cost incurred for plastering, with plan to plaster it in the immediate future. As it is a normal procedure, 95.42% of residential buildings have foundation and basement constructed using rubble. Out of the remaining minority, 60.32% have got laterite or brick foundation. This type of buildings has got mainly tile roofs. A small percentage has got isolated footings. The isolated footing-plinth beam construction is seen in many newly completed buildings. A new trend of using concrete for substructure is observed in Kollam city. When it comes to roofing, lions share is occupied by RCC roofs, 84.31%. 4.19% are tiled roof houses, 2.61% uses tin sheet or AC sheets and 8.89% uses thatched roofing. The last two are found commonly in coastal areas. It is good to use this type roofing as many possibilities exist for sea-erosion. This type of roofs and structures can keep losses to a minimum. No residential structures were found to be older than 70 years. Even structures aged about 50 to 60 years are altered and modified. Interactions revealed that it is not due to aesthetics, but on fear about the strength of the structure, many had done alterations. The percentage wise age of residential structures showed “less than 50 years” (1.32%); (21-50 years) 13.72%; (11-20 years) 7.84% and (0-10 years) were 77.12%. It is evident from the above data that buildings aged between 21 to 50 years are comparatively more. These structures on an average should have further minimum life of 15 more years. This needs the prime attention. Here retrofits are to be provided to increase the life span and seismic resistivity. This group contains both RCC buildings and tiled roof buildings. For retro fitment, classifications are to be deduced out based on the type of structure. A total of 80.39% of residences has got water tanks. The percentage wise distribution of different types of tanks is Fibre tanks (86.17%), RCC tank (7.31%) and masonry tanks (6.52%). Even though RCC and masonry tanks are stronger and can be cast as per size and shape requirements, they pose a big danger as in most of the cases it is impossible to place these tanks in the central point of the structure. The eccentricity of the water tank may cause it to overturn during an EQ. The water tank itself is a big structure. On break up during an EQ, the incumbents are affected by the debris too. Thus permanent water tanks add to the secondary damages of EQs. Fibre tanks are more advisable than other types. For larger capacity more number of tanks can be used with interconnections and these tanks can be placed uniformly throughout the structure.

Commercial Structures (CS): Out of CS surveyed 55% were framed structures, 7.5% were of reinforced masonry, 32.5% of ordinary masonry and 5% mortar free construction. The mortar free buildings were small pan shops, tea shops and provision stores of temporary nature. About 5% of CS was found to be more than 50 years of age. They include establishments in cashew and coir fields where lot of people are engaged in work. 40% of the buildings are between 20 to 50 years of age, 15% are 10 to 15 years and 40% are less than 10 years in age. An important point noticed during analysis was the absence of big type commercial complexes. The highest was a five floor structure owned by government. Medium type commercial structures ranging up to three floors are common. All of them have stair cases rigidly attached to the structures which are not a good procedure for seismic resistivity, though they conform to minimum requirement of IS codes and building rules. 73.22% of the CB can be considered as rigid frames since they have structural support between walls and roof. Those that do not fall under this criteria poses risk, but many are single storied commercial structures with light weight roof or RCC. Only 50% of the structures satisfy the minimum requirement of width of opening to length of wall of 0.40. This is for maximum exposure of the shop for exhibiting the products to the public. Majority of first floor shops have this value greater than 0.40. Almost the same is the case of width between adjacent piers. About 69% satisfies the minimum width of 50 cm. The recommended minimum distance from inside of the outside wall to first opening by code is 30cm. 88% of buildings satisfy this criterion successfully. For space saving and for obtaining maximum floor area, symmetry was achieved by majority of structures. 93% of CS were found to be symmetric. Only 20% of CS had over head water tank. Here also economy had played a vital role. Two-three storied CS were served by four or more number of tanks connected together. It constituted about 12% of the total commercial structures. The rest have RCC over head tank, most being at centre portion of the structure creating a balance. Considering roofing, 71% has RCC roofs, 12% has tiled roofs and 17% has AC or Tin sheet roofs. The tiled roofs account for most of the aged structures. 63% of the structures rest on isolated footings, 35.3% rests on random rubble masonry. Just above 1% have pile foundation and 0.7% has a raft base. The presence of strong soil and absence of big business malls may be the reason for this low percentage of pile and raft foundations. The pile foundations were all cast in situ concrete piles. 71% of the commercial establishments had all-round lintels, also all of the cashew factories got intermittent concrete horizontal bands in addition to lintels. About 21% of the commercial buildings had cellar floors. Most of them are used as parking lots or storage spaces. The existence of cellars merely over columns is not advisable from seismic point of view. In Chile, where EQ struck recently, most of the commercial structures overturned or tumbled because of cellar openings. However strong the basement, there are chances of structural failure in case of an EQ. Space constraints for parking etc are making constructors to go for cellars.

Detailed Analysis of Hospitals, Government Buildings and Public Buildings: These structures that attract the public at all times play a vital role during an EQ. On hospital buildings majority about 90% rests on isolated footings with tie beams and plinth beams. All are well plastered and have the required width to length of wall ratio. About 80% of these hospitals have got separate over head water tanks, which is a good thing. 65% of these hospitals exists in the very core areas of the city and have another hazard from adjacent towers. The towers erected by mobile communication service providers for repeating signals are a point of danger if an EQ strikes. These towers can fall over hospitals thereby harming their smooth functioning as relief centres. The average occupant rates in the surveyed hospitals were 45. Thus the magnitude of danger posed by these towers is comparatively high and has to be avoided immediately. These towers may be kept away from hospital and city centre, or to make use of some new technique for mobile receptions. Due to expansion, 95% of hospitals have lost symmetry. Only a few seems to have planned growth pattern. However construction joints were provided on the expanded structures. The designers deserve credit for these measures which reduce the risk of total collapse. Majority of hospitals have RCC covered roof. And many have additional tin sheet covering, few have tiled roof. It can be accepted as there are no inpatient wings functioning in these hospitals. The government buildings seem to conform to all the code regulations and buildings rules, but the quality of work is a matter of concern. Even a government building less than 10 years of age was found to have large cracks on main walls. In the planning stage engineers can avoid risk by keeping the structure symmetric. Only one building was found to be non-symmetric throughout this survey. Most government buildings had overhead water tank, but all are of fibre material. The credit is due to the PWD schedule of rates for incorporating fibre tanks in them. 67% of government buildings were aged between 10 to 20 years, 13% in between 21 to 30 and 5% between 31 to 40%. Just a few were greater than 40 years of age. But they were also face lifted and retrofitted in the near past. About 30% of the government buildings are tile roofed. Rest are RCC covered. A few government structures were with AC sheet roofing, but they were negligible in numbers and were eliminated from analysis. About 20% of the government buildings were adjacent to high structures in the form of over head water tanks or mobile towers. This is the source of hazard. The strategically important structures were very few in number. The KSEB substations were made conforming exactly to the code provisions. They are also symmetric, had plinth beam and lintels all-round and isolated footing as foundation. Same was the case with the Communication department also. They extensively use random rubble masonry with concrete bands in between. But some structures were not symmetric. Any way the strategically important structures can function well if an EQ happens. Public buildings were largely in the form of clubs, libraries and reading rooms where the pattern of occupancy is more in the evening time. About 78% of the public buildings were less than 10 years in age. This is due to the fact that the local aid schemes for people’s representatives are being made very good use for these type structures. Hence they got plinth beam, lintels, isolated footings and RCC roof covering. Only less than 3% buildings were more than 30 years of age. Just 6% has got overhead water tanks, that to fibre made. Symmetry was there in most of the structures. 35.7% of structures fall short of the criteria of width of opening to length of the wall. This is due to the usage of mild steel grills extensively, especially in the case of reading rooms. Some other public buildings like fish market simply rests on columns with roof and tie beams posing a big question mark about seismic resistivity. In general, public buildings appear to be safe during an EQ according to the general standards.

Detailed Analysis of Educational Buildings: About 5% of the total buildings fall under this category. Educational institutions having age greater than 60 years were seen. The most peculiar features found from the analysis are as follows - 98.7% of buildings are symmetric, 76.8% have RCC roofs, and 23.2% have tiled roofs. All the tiled roof buildings got intermediate bands of concrete in addition to all round lintels. A few of the tiled roof buildings were two storied with wooden floor, which is finely performing. The width of opening to length of wall ratio, pier to pier opening distance and minimum corner edge distance was found to be satisfied by all the structures with tiled roof, i.e. those having age between 30 to 50 years. It is pathetic to say that the comparatively new buildings of RCC fall well short of this criteria. About 48.7% does not satisfy any of the three provisions regarding openings. 67% schools got separate over head water tanks. Structures which have overhead water tanks use fibre ones for the purpose. As most schools have play ground adjacent to buildings, the possible danger of high rise structure is avoided. Educational buildings has got random rubble foundation for 55%, isolated footing for the rest of 45%. The average number of occupants for each class was about 45. As the old buildings do not have rigid structural support between walls and roof; it seems a sort of danger. 52% of schools have multi-storey structure. But the maximum number of storeys was limited to three, that to a minimum number. No educational building was found to have raft or pile foundation.

6. SUMMARY OF ANALYSES

Residential Structures: Structures in soft soil strata constitute 86.92% and hard soil strata 13.08% of the structures. Information on various aspects of structures in soft and hard soil was collected. The most important feature that affects the seismic resistivity of masonry structures is the structural support between walls and roof. On an average of 31.13% of buildings doesn’t have that support. Hence they need the prime attention for retrofit. Also an average of 11.41% doesn’t have all round lintels. Here retrofit in the form of wire-meshed concrete at corners is necessary. In general, an average of 21.27% structures requires essential retrofits.

Commercial Structures: Framed structures constitute 55% and non-framed structures constitute 45% of the structures. The important factors that affect the seismic resistivity of commercial buildings are the ratio of openings and the cellar parking provisions. 50% of the commercial structures have to be modified for necessary ratio of openings. 21% of the structures have to be altered to reduce the risk due to cellar parking. Hence a total of 35.5% commercial structures require essential retrofits.

Hospital buildings: The secondary damages caused due to the collapse of high rise structures may affect the proper functioning of hospitals after an EQ. The functionality can be harmed by the collapse of water tanks also. Hence in an average 41.65 **%** of hospital structures are likely to be affected by EQ.

Government buildings: Majority of GB have full filled the requirements by IS codes. But the quality of workmanship is a matter of concern. The structural support between walls and roofs are essential and 42% of government buildings require retrofit in that section. The high rise structures also pose danger. Hence an average of 31% of GB is to be retrofitted suitably to avoid the danger of EQ. Most of the public buildings do not satisfy the criteria of minimum opening ratio. Hence 64.3% of public buildings are to be retrofitted. Absence of structural supports is seen in 42 %. Therefore an average of 53.15% has to be considered for retrofit to ensure their successful functioning. Educational buildings have an average of 45 incumbents per class. Hence the prime importance must be given to the structural support between walls and roof and the opening ratios. As per analysis, both the criteria are not full filled by majority. Hence an average of 38.15% of educational structures requires retro fitment. Overall 36.78% of the existing building structures in Kollam Corporation require essential retrofitting for smooth and safe functioning.

7. METHODS OF RETROFIT

The masonry building can be retrofitted effectively by strengthening the corners. For this, the wall plastering of the corners is to be removed and an iron mesh with exactly same width as the plaster removed is nailed in the gap. Concreting can be done over that additional reinforcement using a rich mortar mix of 1:1.5:3 or 1:2:4.The surface can be then plastered and trowelled smooth. This process is comparatively cheap and requires less skilled labour. The wall will look the same as before retrofit. The most important problem of lack of structural connection between roof and walls of ordinary masonry structures can be remedied by using any of the following technique. A rigid connection is kept between the roof and wall using specially made clamps and bolts. This method is comparatively cheap and can be adopted in a region like Kollam very easily. Timber structures can be retrofitted using MS plates at joints of the members with the help of nut and bolts. The joints which are generally weak are held together using two plates placed on the both sides of the members. Bolts are inserted through holes drilled and the members are held in tight position with the help of nuts. This method has the advantage of easy maintenance and can be used for many years. Periodic checking and tightening, if necessary, can be done in this method. Existing CS can be retrofitted using mild steel beams and channels. An additional brace-cum-girder arrangement can be provided for commercial buildings. The loads are carried by these new beams and shear walls and low ductile beams are exempted from carrying heavy loads. Thus safety of the structure is ensured. For new multi storey constructions, cross bracings can be provided on columns and base isolators on the foundation to reduce the effect of quakes. Trenches around the structure called ‘MOAT’ is formed for the easy placement of structure over the base isolators. A shear core is formed exactly at the centre of the structure to receive any shocks during quakes. Shear walls are also positioned towards the centre of the structure.

8. CONCLUSIONS

In general, buildings in Kollam city have sufficient resistance against moderate EQs as per Indian standards and general criteria. Immediate and important attention is required for the RS. The growth rate of RS is very high and just a few have got seismic resistivity of its own. Government should impose new rules regarding seismic resistivity regulations and should educate the public about the possible damages due to EQs. The tendency of providing large openings and asymmetric designs are to be curbed. Economic retrofits in the form of wire meshed concrete on the corners of masonry walls in tiled roof structures and division of longer walls to shorter ones can be done for structures greater than 30 years of age. In CS strict rules must be imposed to avoid huge capacity overhead water tanks.

The provision of open cellar area for parking must be avoided. Tie beams at regular intervals should be provided for structures having floor height higher than 4m. The government should seriously think about changes to be incorporated in the IS codes regarding EQ resistivity. The water tanks must be kept away from the buildings in the campus. The structural support between roof and walls of old tiled buildings should be improved by using cleat and angles connection. Long and big construction in a single stretch must be avoided. On the other hand smaller structure must be placed well apart.

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COMPUTER APPLICATION IN EARLY PHASE OF DESIGN OF INTELLIGENT BUILDINGS

Sutapa Das[1] and Abhijit Chaudhuri[2]

[1] Indian Institute of Technology Kharagpur, India

[2] Tata Consultancy Services, Kolkata, India

[1]sutapa@arp.iitkgp.ernet.in , [2]abhix.c@gmail.com

ABSTRACT: Compared to earlier days, modern building projects have significantly evolved in terms of size, complexity and speed of completion resulting into a multi-user multi-tasking design environment. Buildings have become a complex integration of various systems and services. The trend has been technically supported by development in information and communication technology and resulted in intelligent building (IB). In case of IB, trandisciplinary integration must cater to the places, processes, people and management in a synchronized manner. Hence, with a huge array of possible technical solutions, the importance of design management and decision making in order to critically assess alternative schemes from early phase of design is emphasised more than ever before. Unlike other engineered products, building design varies from project to project. Hence virtual prototyping or testing the alternatives in design phase itself is recommended leading to integrated project delivery. The iterative process of design can be made systematic and conscious decision can be drawn by using various CAD based tools such as Building Information Modelling (BIM), where computing can reduce significant amount of time and confusion through calculation, drawing, drafting, presentation, communication and data handling. Apart from these generative tasks, computers help in analytical tasks of design evaluation – a mandate especially for high-tech intelligent buildings. Starting with a brief history of evolution of IB and its unique requirements, this paper illustrates the whole design process with reference to IB in details with examples. The paper also includes the potential of BIM in IB design along with a future scope of its application in facility management of IB.

Keywords: BIM, computing, design, integrated project delivery, intelligent building.

1. INTRODUCTION

Since last two decades, we are undergoing major transformation of how buildings are designed, constructed and operated. The reasons are not only the complexity and size of the buildings have risen steeply along with the users' demand for a better performing building, but an, emphasis on life cycle costing, expedited project deadline, energy efficiency and building automation have added to the 'Market pull'. At the same time, there has been a 'Technology push' via tremendous growth of information and communication technology (ICT) based hardware to aid to integration of building services (BSI & BuildingSMART, 2010). As a result, the intelligent building (IB) concept emerged in 1990s in a multi-user multi-tasking environment (Wong et al., 2005). IB differs significantly from its predecessors in terms of design, construction, operation or maintenance. This paper presents a state-of-the art review of designing of IB using computers.

2. IB VS. CONVENTIONAL BUILDINGS

Concept of IB has changed over time. A building which has fully automated building service control systems (Cardin, 1983 - sited in Wigginton & Harris, 2002) or one which integrates various systems to effectively manage resources in a coordinated mode to maximize: technical performance, investment and operating cost savings, flexibility (The Intelligent Building Institution in Washington, 1988 – sited in Clements-Croome, 1997) has a purely technical focus and no consideration of human productivity carried the potential threat of functional obsolescence.

On the contrary, a recent definition by CIB (1995) encompasses human factors and describes IB as a dynamic and responsive architecture that provides every occupant with productive, cost effective and environmentally approved conditions through a continuous interaction among its four basic elements: places, processes, people and management and the interrelation between them.

IB differs significantly from conventional buildings in terms integration of its systems rather than sophistication of the same (Arkin & Paciuk, 1997). This integration follows a hierarchy (Figure 1) which provides 'qualities that create a productive and efficient environment such as functionality, security and safety; thermal, acoustical, air-quality and visual comfort; and building integrity' (Bradshaw & Miller, 1993).

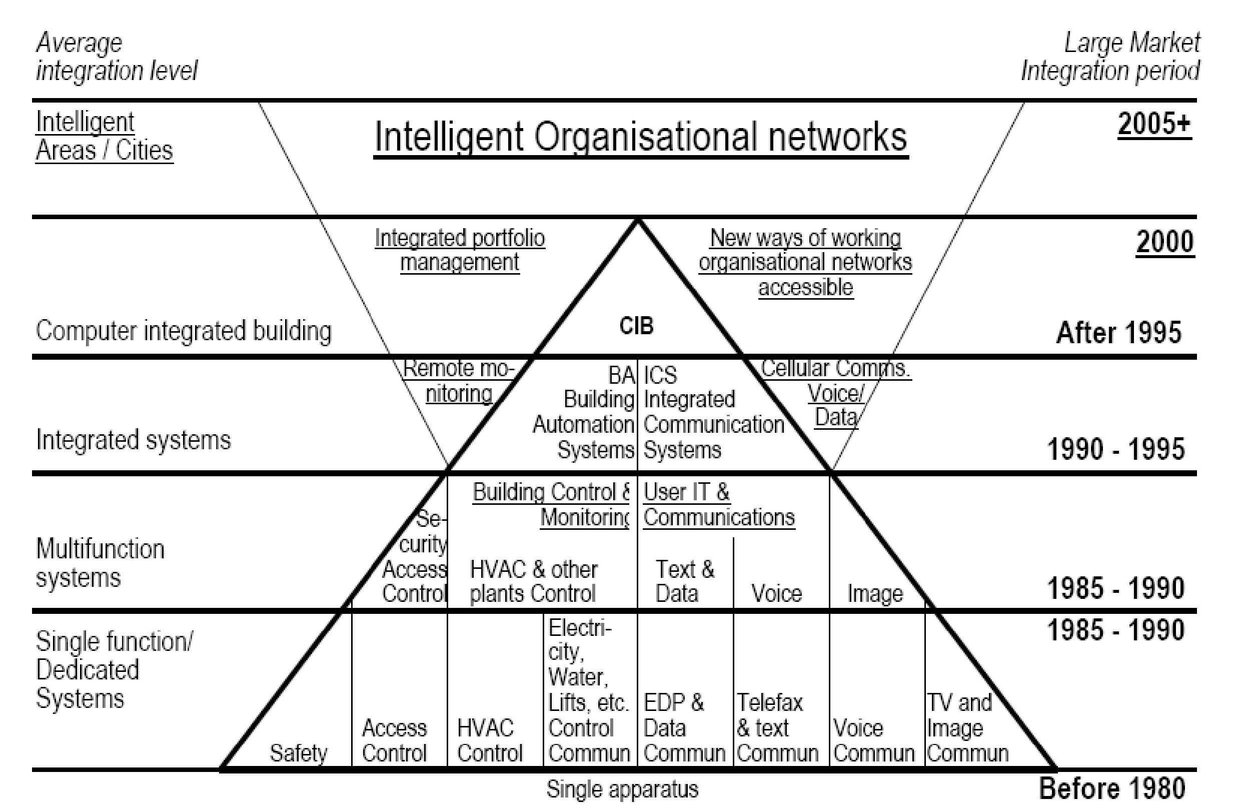

***Figure 1**. The Intelligent Building Pyramid (Himanen, 2003)*

Because of the critical emphasis on integration, design and the design management of IB also differs significantly from that of conventional buildings. Often IB projects follow integrated project delivery (IPD) process such as design-build. In this case, project members are involved at an earlier phase of design and goals are set during the concept phase itself (Figure 2).

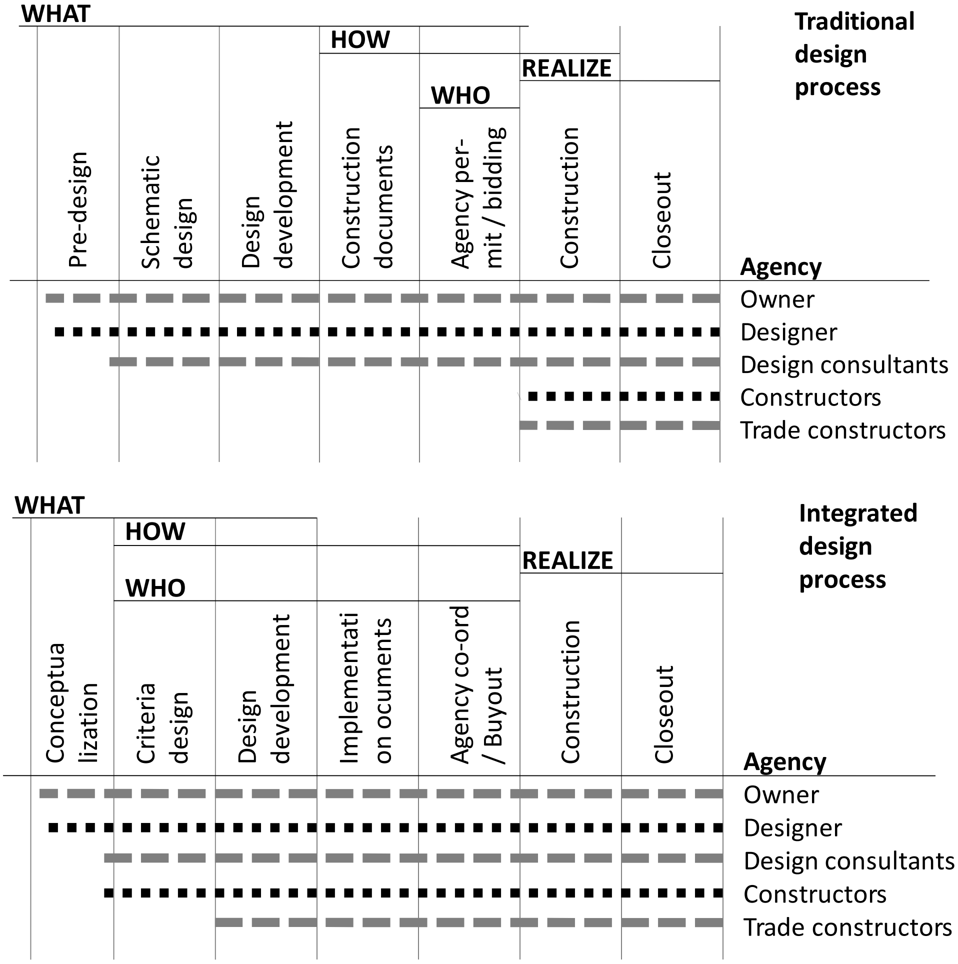

Figure 2. *Traditional Vs. Integrated Design Process (AIA 2007)*

3. LEVEL OF INTEGRATION IN IB

Over time, focus of building design has changed as:

- 1960: Functional efficiency.
- 1970: Costs in use / life cycle cost.
- 1980: Quality and effectiveness.
- 1990: Human productivity and sustainability.
- 2000 onwards: Flexibility, adaptability, service integration and high standards of finishes – all leading to intelligent threshold.

As IB encompasses all of the above mentioned goals, Clements-Croome (2000) explains further why integration and transdisciplinary is the ultimate keyword in IB industry by stating:

"Any consideration of intelligent buildings, whether learning, designing or managing them requires a freedom of thinking which can embrace transdisciplinary ideas and systems. The word transdisciplinary is a truly holistic and highly interactive concept. Intelligent building strategies are dealing with multiple criteria and attempting to integrate ideas over a very wide range."

After this comment, more than a decade has passed. But the same trend is being followed and with a much higher intensity. Sinopoli (2011) predicts the trends in smart building design as:

- Traditional Building Management Systems (BMS) will be obsolete due to their slow and narrow response to latest IT applications and will be configured with open communication protocols.

- Energy management software with agility and speed will flourish irrespective of the size or fame of the companies developing them.
- Large organizations with significant needs for facility technicians, engineers and managers will collaborate with technical institutions and offer internships to build the human resource pool.
- Telephone or cable TV service network may also provide telemetry service for building operation or energy related information and majority of the wireless connectivity will be through 'White-Space' devices (WSD).
- Certified and accredited designers, contractors and operators for integrated building systems (e.g. LEED, BIM) are in high demand.
- Concepts such as smart grid or demand response will find extensive application.

It is also worthy to note that while achieving transdisciplinary integration, not only regular services such as HVAC or elevator are included but the major part of ICT related services are also included (Abu Sharkh, 2007). These services are namely, cabling, networking (wired or wireless), smart building solutions (CCTV, remote access, security, fire detection, management system, car park control, alarms, etc), PBX with regular or IP telephony.

4. BENEFITS OF EARLY INTEGRATION IN IB

Here the vision is defined and design of all systems and services begin at the concept phase. Earlier the involvement of various key players - better is the result in terms of cost and performance. Such benefits are captured by MacLeamy Curve (Figure 3). While comparing the traditional and integrated project delivery, MacLeamy (CURT, 2004), argued that as the cost of making design changes rises over time, so the ability to design improvement and cost reduction decreases. These factors are plotted in a diagram known as the MacLeamy curve. It means that an earlier integrated effort in conceptualizing the design will save money and will be performance wise more effective compared to effort put in later phases (AIA, 2007).

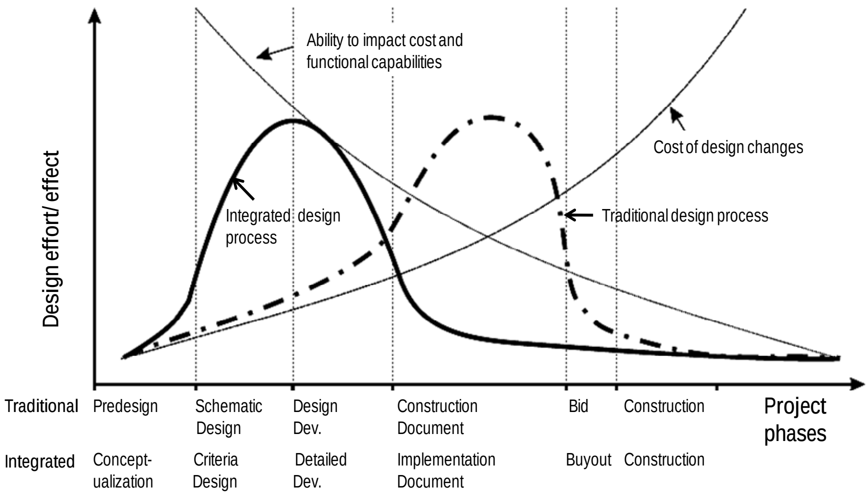

***Figure 3**. Macleamy Curve Showing Benefits of Early Integration*

More precise figures of these benefits are available where the benefits of early integration are quantized through cost saving or energy saving. For example, a typical IB compared to its traditional version can save in (CABA, 2009): first cost (56%), annual operation cost (32%), annual maintenance cost (82%), utility costs (10%) and net present value in 10 years at ROI 9% (24%). Only in terms of energy saving, the figure can be 21-35% depending on building type, size, climatic condition etc (Elliot, 2009).

5. THE DESIGN PROCESS

Before we discuss about design process of IB in particular, it is required to understand the design process in general which will explain why design of IB is different from a traditional building. For any design, there is no unique and ultimate solution for a problem, but the optimum one is elected from possible alternatives based on various objective and subjective parameters. Hence, designing is rational decision making and problem solving process to meet certain functions. Design can be generative or analytical. In first case, numerous solutions are generated, for example various fenestration designs for a room, but selection of the best among them is beyond the scope of this process. On the contrary, the analytical method checks the viability of all proposed alternatives using the following methods (Radford & Gero, 1988):

- Non-dynamic: Evaluates formula and check compliance with rules.
- Dynamic: A function of a building component is run and upon variation of different parameters the outcomes are checked.

For a high value project like building, conventional prototyping is infeasible in terms of time of cost. Hence in both cases appraisal of design alternatives is done through virtual prototyping which means testing and refining the scheme when still on drawing board (Maver, 2004).

5.1 Appraisal of Design Alternatives Using Models

The first appearance of architectural drawing by 2800 BC can be considered as the first step of modelling. For next 5000 years, plan, elevation, section, other sketches and physical scale models – all of which are devoid of virtual development remained the core model for building design.

However, according to system approach, a critical evaluation in true sense is done through a model that can reflect operational behaviour and formal characteristics of the proposed system. Here, input is the design hypothesis; output is prediction of operational behaviour and functional characteristics under a particular set of context or extraneous variable. If the output does not comply with the desired result, the deviation is taken as a feedback and another scheme is developed. Hence design process is iterative (Figure 4a). For example, heat loss from a window can be regulated by various design input variables such as size, location, glazing material, shading device etc, but external factor climate also plays a big role (Figure 4b).

Here apart from technical aspects, other aspects such as functional, aesthetic, social and economic aspects are also considered. Sometimes they can be contradictory also. For example, a window must have higher light transmission and lower heat gain. Both are not achievable at the same time unless low emission or double layered glass is used (Koengsberger et al., 1973).

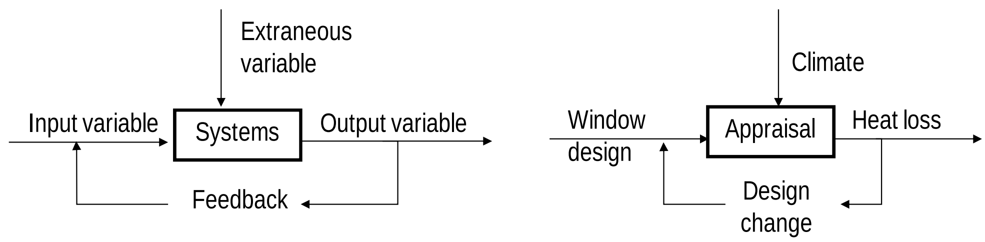

Figure 4. *Design Appraisal in System Terms (a) Generic, (b) Specific (Maver, 2004)*

5.2 Why CAD Based Models Are Popular

The iterative design process essentially comprises of four stages, namely, representation, measurement, evaluation and modification. It is found that, compared to traditional paper-based or scale models, computer generated model performs significantly better in all these stages especially in the second one because they are predictive and dynamic instead of descriptive and static. If first two phases are set and made available, other two processes become dynamic and embedded within design itself. For example:

- Representation – The design hypothesis is input in the computer. E.g. building plan, bridge elevation, printed circuit layout, mechanical linkage etc. It links designer's mental model and CAD model i.e. it works as man-machine interface.

- Measurement – The software models the behaviour of the hypothesised design and generates cost and performance on a number of relevant criteria as output. E.g. bill of quality, deflection profile for bridge span, resistance between two circuit nodes, angular velocity of linkage arm etc.
- Evaluation – The designer or client applies value judgement on deviation of actual output from desired one. E.g. To achieve 2% minimum daylight factor, window size may be increased which in turn increases the cooling load a lot. In such case, what should be prioritized?
- Modification – Any appropriate change is suggested to the initial scheme.

A first generation of dynamic model is shown in Figure 5 and was used by D'Cruz et al. (1983) in their research to find the optimum window design for a corner room in intermediate floor of an office building in Hobart, Australia. They changed the opening size and shading device to evaluate performance against thermal, lighting and spatial efficiency along with cost. The problem formulation used a form of Pareto optimal dynamic programming optimization.

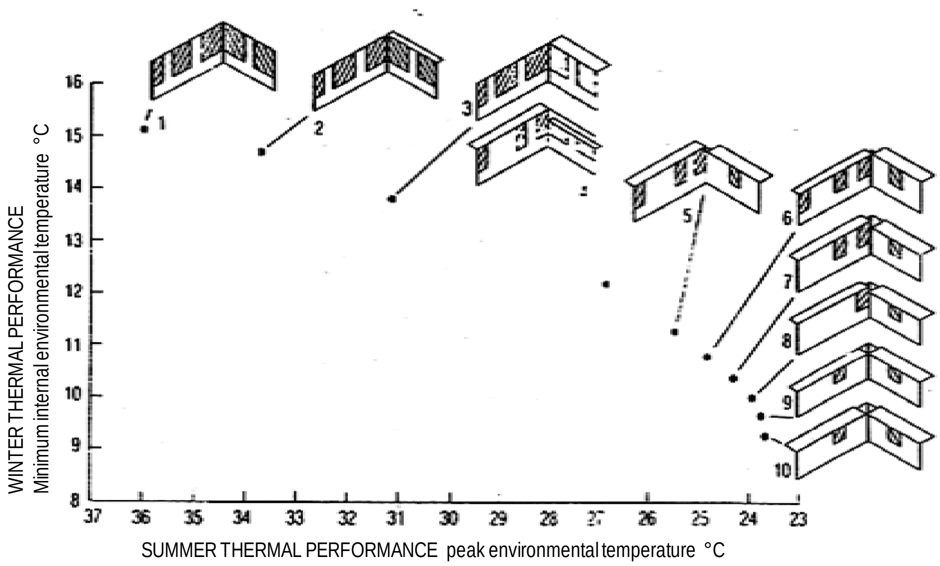

Figure 5. *Design Optimization for Window Using Pareto Set of Performance*

Today's developed software programs and smart technology together have expedited and fine-tuned the whole process to a great extent. Now it is an ordinary event to work on the same project simultaneously by many people in design team using computer. Apart from speed, CAD based models offer the following benefits essential for IB:

- Widened search – By changing any of the input parameters, numerous output or design alternatives can be generated compared to 2-3 alternatives generated by traditional method. It allows dynamic comparison of cost vs. performance of these models.
- Greater integration in decision making – Previously, members of design team used to work on a scheme individually and in stages. Feedback from one profession often used to result in massive redesign of other disciplines. Now whole team can not only work together, but also people can check their work against predefined parameters before fine tunings are made.
- Improved design insight – Study of influence of varying input parameters on performance outputs sets up causal relation among parameters.
- Differentiation of objective and subjective judgements – Based on quantitative rather than intuitive.

6. BIM AND OTHER TOOLS IN IB DESIGN

IB is extensively service oriented and is seldom erected from scratch at site, its components and services are manufactured separately in different factories and even at different countries and assembled at site. As a result, the architect or the other core design team members coordinate with an integration consultant who possesses knowledge of the products suited to the vision of the team and who only specifies the requirements of the integration (Elliot, 2009). Many details and working drawings are left to the manufacturer cum contractor (Al Bizari & Gray, 2004). In this regard, Building Information Model (BIM) is a major break-through which goes much ahead of other simulation tools for individual services such as acoustic, lighting, structural performance etc.

BIM a 3D object oriented CAD approach which structures building related information in such a way that the data can be shared (Sinopoli, 2009). Widely recognised BIM softwares are Tekla, Revit, ArchiCAD, Microstation etc. A BIM is a digital model of a building in which information about a project is stored. It can be 3D, 4D (integrating time) or even 5D (including cost) – right up to 'nD' (a term that covers any other information). The key strength of BIM lies in information management as (BSI & BuildingSMART, 2010):

- Information, once captured, can be reused and repurposed
- Information can be reviewed and revised, corrected and controlled
- Information can be checked and validated.
- Information is object oriented and inter-operable following IFC (Industry Foundation Classes) standard.
- Information-flow within the team is simple and comprehensive (Figure 6).

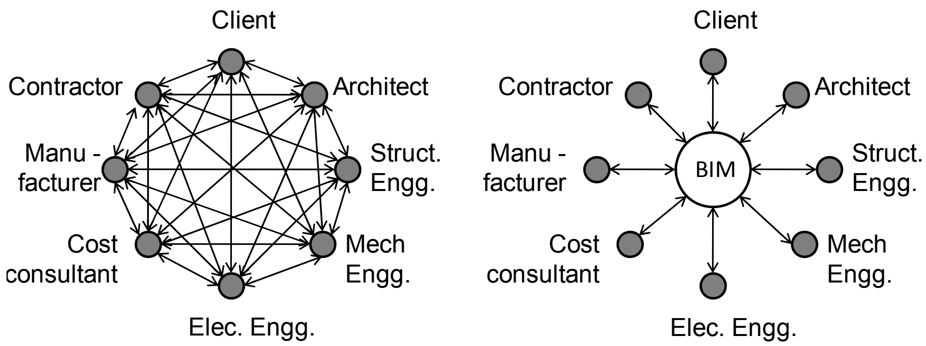

Figure 6. *Information Flow in (a) Traditional Design (b) BIM Based Design*

Among various advantages of BIM, the salient ones are (Maunula, 2008):

- Designers can connect project program and tools via web.
- Detailed BOQ/ cost estimation.
- Possible conflicts in design and construction are identified.
- Marketing of construction approaches
- Lesser errors and hence rectifications – saves costs and resources.
- More "what if" analysis, such as construction sequencing options, shuffling of human resources, fine-tuning of cost factors, etc.
- Understanding and visualizing the process and end products help in conscious and informed decision making

6.1 Future Application in Facilities Management

Facilities management is more considered as mere repair – maintenance but it has become a part of the design process where the ease of operation and maintenance is planned like performance of building systems and services. This is the longest and revenue-earning period of a building life cycle and BIM can be used effectively. For example, the 'as-built' information stored in BIM can be used in creating the maintenance manual, to observe the tasks of facility management virtually, and that the model could be updated when renovation is conducted (Manaula, 2008). In this regard, BIM is expected to coordinate with other smart tools used in IB industry such as (MDCS, 2005):

- Computer Integrated Facility Management (CIFM): Integrated technology and processes supporting facility management and real estate services.
- Computer Aided Facility Management (CAFM): Calculation of building area and tracking of space classifications.
- Computerized Maintenance Management System (CMMS): Manages scheduled and on-request building maintenance.
- Electronic Document Management System (EDMS): manages the documents associated with facility design and management.
- Geographic Information System (GIS).

7. CONCLUSIONS

This paper summarised why and how IB differs from its traditional counterparts. Next the importance of transdisciplinary integration in IB is explained considering the present and future trend of IB industry. The point is further illustrated with generic and specific examples. Once the goal of IB is established, the paper explains how the design of IB can be approached systematically using models especially the CAD based ones and how it can help in selecting the optimum solution. Finally the most recognised modelling technique of BIM is described briefly along with its realised advantages over previous models and its future potential in facility management of IB is explored.

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COMPRESSIVE STRESS-STRAIN RELATIONSHIP OF FOAMED CONCRETE AT HIGH TEMPERATURES

Md Azree Othuman Mydin

School of Housing, Building and Planning, Universiti Sains Malaysia, 11800, Penang

azree@usm.my

ABSTRACT: Even though foamed concrete has low mechanical properties compared to normal weight concrete, there is a potential of using this material as partition or load-bearing wall in low-rise residential construction. Before it can be considered for use as a load-bearing element in the building industry, it is necessary to acquire reliable information of its mechanical properties at ambient and high temperatures for quantification of its fire resistance performance. This paper will present the results of experiments that have been carried out to examine and characterize the compressive stress-strain relationship of foamed concrete at high temperatures. Foamed concrete with 650 kg/m^3 and 1000 kg/m^3 density were cast and tested. The compression tests were carried out at ambient temperature, 100, 200, 300, 400, 500 and 600°C.

Keywords: stress-strain, foamed concrete, high temperatures

1. INTRODUCTION

Foamed concrete is not a new material in the construction industry. It was first patented in 1923 (Valore, 1954) and a limited scale of production was instigated in 1923. The use of foamed concrete was very limited until the late 1970s, when it was started to be consumed in Netherlands for ground engineering applications and voids filling works. In 1987 a full-scale assessment on the application of foamed concrete as a trench reinstatement was carried out in the United Kingdom and the achievement of this trial led to the extensive application of foamed concrete for trench reinstatement and other applications followed (Brady et al., 2001). Since then, foamed concrete as a building material has become more widespread with expanding production and range of applications.

Although foamed concrete has low mechanical properties in comparison with normal weight concrete, there is a possible of utilizing this material as partition or load-bearing wall in low-rise residential construction. Before it can be considered for use as a load-bearing component, it is essential to obtain dependable information of its mechanical properties at ambient and high temperatures for quantification of its fire resistance performance. Mechanical properties of concrete due to exposure to high temperatures have been studied since a long time ago. Nevertheless, there is no such extensive study of mechanical properties of foamed concrete at high temperatures.

Lin et al. (1996) conducted studies to investigate the microstructure of concrete exposed to high temperatures in both actual fire and laboratory conditions with the assistance of Scanning-Electron-Microscopy (SEM) and stereo microscopy. They found that the absorption of moisture from the surrounding medium provides a mechanism for the rehydration of calcium oxide and unhydrated cement grains that refilled the void spaces. They observed that long irregular fibers of C-S-H gel combined with ettringite and C-H crystals and formed as a result of rehydration.

In a study carried out by Schneider and Herbst (1989), chemical reactions and the behavior of calcium hydroxide, calcium carbonate, calcium silicate hydrate, non-evaporable water and micropores under various temperatures was examined. They found that the major increase of concrete permeability and porosity at high temperature was primarily produced by arising microcracks and by changes of material inner structure, as well as by crack opening due to high gas pressure values. As a result, the permeability of concrete depends not only on temperature levels, moisture content and gas pressure but also upon the degree of crack development.

It can be pointed out that the degradation mechanisms of cement-based material like foamed concrete upon exposure to high temperatures include chemical degradation and mechanical deterioration where each mechanism is dominant within a specific temperature range. The dehydration process in the cement paste becomes significant at temperatures above about 110 °C (Khoury et al., 2002) and diminishes the calcium silicate hydrate (C-S-H) links which provide the primary load-bearing formation in the hydrated cement. Furthermore, due to low permeability of the cement paste, internal water pressure built up during dehydration of the hydrated C-S-H, which increases internal stresses and induce microcracks in the material from about 300°C, resulting in decreased strength and stiffness of the material (Hertz, 2005). At higher temperatures around 450°C, calcium hydroxide ($Ca(OH)_2$), which is one of the most vital compounds in cement paste, dissociates, resulting in the shrinkage of foamed concrete.

A variety of test methods may be used to obtain different aspects of mechanical properties of materials at high temperatures, including the stressed test, the unstressed test, and the unstressed residual strength test (Phan and Carino, 2003). In this research, the unstressed test method was adopted for convenience. In the unstressed test, the sample was heated, without preload, at a steady rate to the predetermined temperature. While maintaining the target temperature, load was applied at a prescribed rate until sample failure. Because the temperature is unchanged, the test is also referred to as steady state test, as opposed to transient test in which the specimen temperature changes with time.

2. SET-UP FOR COMPRESSION TESTS

Figure 1. *High temperature electric furnace with specimens*

Two different electric furnaces were used for heating the foamed concrete specimens to the various steady-state temperatures. One furnace had a maximum operating temperature of 450°C (low temperature furnace), and the second furnace had a maximum operating temperature of 1000°C (high temperature furnace). Each of the furnaces was capable of holding three specimens. The low temperature furnace had a temperature range of 50°C to 450°C and was used for four of the reported thermal exposure conditions: 100°C, 200°C, 300°C and 400°C. The furnace temperature exposure profiles were produced by a programmable microprocessor temperature controller attached to the furnace power supply and monitored by a Type K thermocouple located in the furnace chamber. The high temperature furnace (Figure 1) had a maximum operating temperature of 1000°C. This furnace was used for exposing concrete specimens to 500°C and 600°C. This furnace was also controlled by a programmable microprocessor temperature controller attached to the furnace power supply based on feed-back temperature reading from a Type K thermocouple located in the furnace chamber. Pre-testing checking of the furnaces showed that both furnace controllers and furnace power system could keep furnace operating temperatures within ±1°C over the test range.

The compressive tests were carried out on 100 x 200 mm cylinders. The specimens were removed from moulds after 24 hours of casting and then cured in a water tank at 20 ± 2°C for 28 days. Prior to testing, the specimens were removed from the curing tank and put in the oven for 24 hours at 105°C. After 24 hours, all the specimens were removed from the oven and their ends were ground flat. To monitor the strain behaviour at ambient temperature during loading, two strain gauges was fitted on each sample for the ambient test only. These ambient temperature strain measurements were used to confirm that the strain calculated based on the displacement of the loading platen was of sufficient accuracy. Since it was difficult to measure strain at high temperatures, the displacement of the loading platen was used to calculate the strain for the high temperature tests. Four Type K thermocouples were installed in the central plane of each cylinder specimen to measure the specimen temperature, as shown in Figure 2.

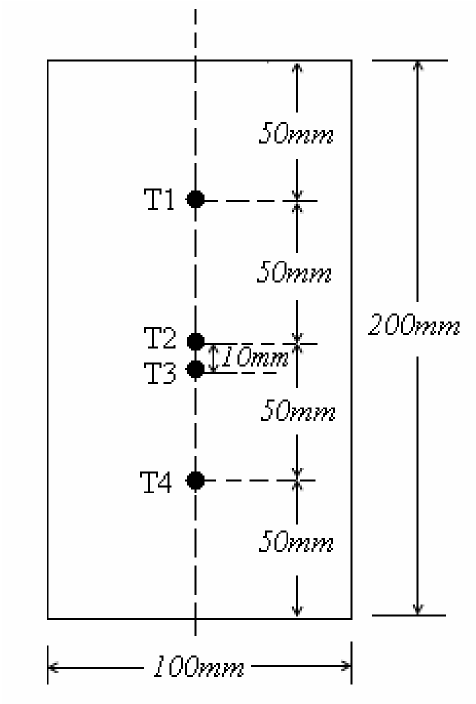

Figure 2. *Typical 100 x 200 mm cylinder specimen with thermocouples arrangement*

Loading was applied using an ambient temperature machine after removing the test samples from the furnace. Each specimen was wrapped with insulation sheets immediately after being removed from the electric furnace to minimise heat loss from the specimen to atmosphere. For each set, three replicate tests were carried out to check consistency of results. The target temperatures were 20°C (room temperature), 100, 200, 300, 400, 500, and 600°C.

During the loading process, the temperature of each sample (thermocouple T3) was measured and it was found that the temperature was stable throughout the testing period. Figure 3 shows typical temperature variations throughout the loading phase for specimens of 1000 kg/m^3 density. As can be seen, because the duration of loading was short (just over one minute), there was very little heat loss and the temperature change was less than 0.5^{o}C.

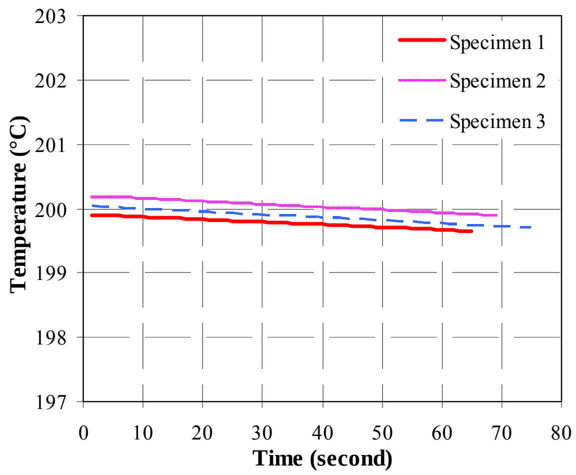

Figure 3. *Temperature change during test of specimens of 1000 kg/m^3 density at target temperature of 200°C*

3. RESULTS AND DISCUSSIONS

The engineering stress-strain relationships of foamed concrete were determined from the measured load and deflection results using the original specimen cross-sectional area A_o and length L_o. Due to difficulty of using strain gauges at high temperatures, the deflection used to calculate the strain was that of the movement of the loading platen. Strains were measured at ambient temperature to confirm this method. Figure 4 compares the measured strain and that calculated using the displacement of the loading platen for the ambient temperature test. This comparison demonstrates that it is sufficiently accurate to use the loading platen displacement to calculate the axial strain of the test specimen.

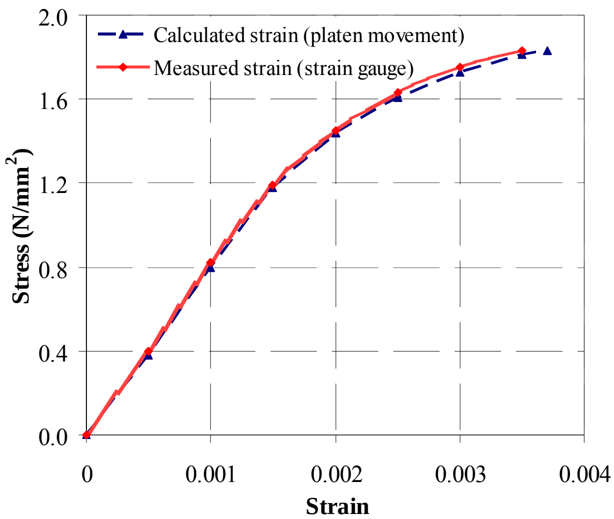

Figure 4. *Comparison of measured strain and calculated strain (based on movement of the loading platen) for foamed concrete of 650 kg/m^3 density at ambient temperature*

The tests were displacement controlled where the crack continue to develop and grow after the peak load is reached. However, since the test specimens failed in a brittle manner after reaching the peak stress, it was not possible to obtain the descending branch of the stress-strain relationship. Figures 5-8 present typical stress-strain relationships for the three duplicate samples at different temperatures for the 650 kg/m^3 density specimens and Figures 9-12 are for the 1000 kg/m^3 density specimens. It was clear that all the three duplicate samples produced very consistent results. Figures 13 and 14 present the average stress-strain curves at all different testing temperatures for the two densities.

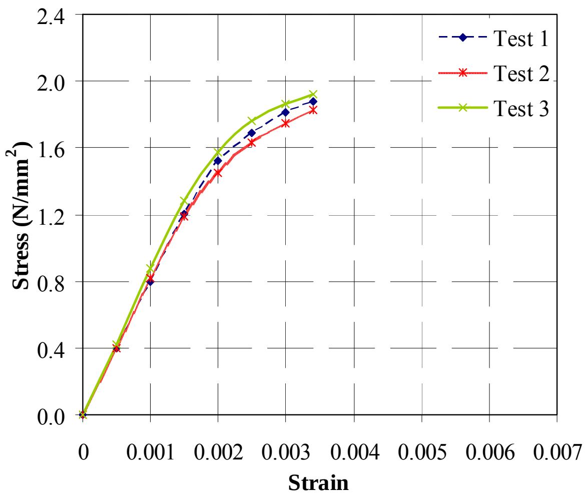

Figure 5. *Stress-strain relationship for foamed concrete of 650 kg/m^3 at ambient temperature*

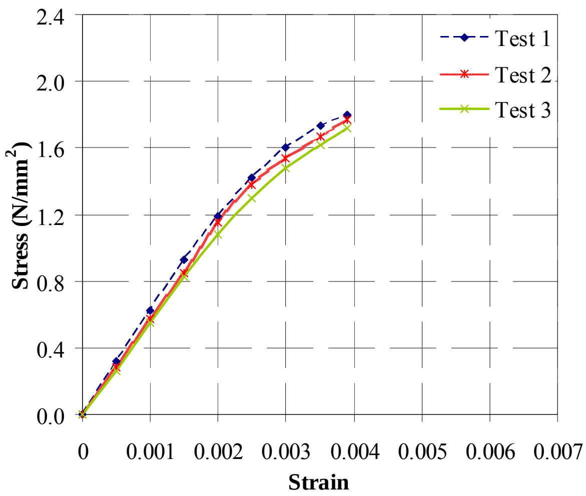

Figure 6. *Stress-strain relationship for foamed concrete of 650 kg/m^3 at 200°C*

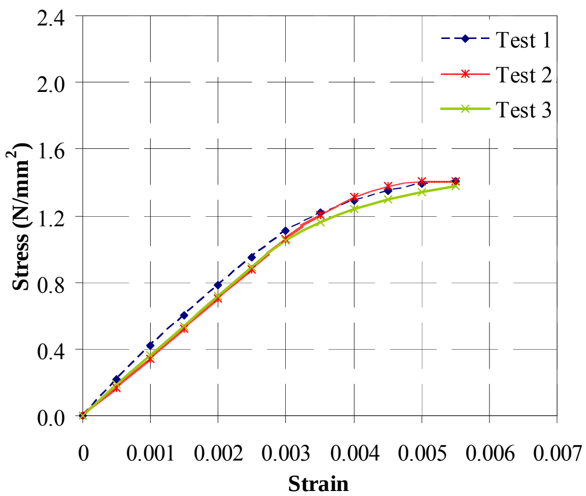

Figure 7. *Stress-strain relationship for foamed concrete of 650 kg/m^3 at 400°C*

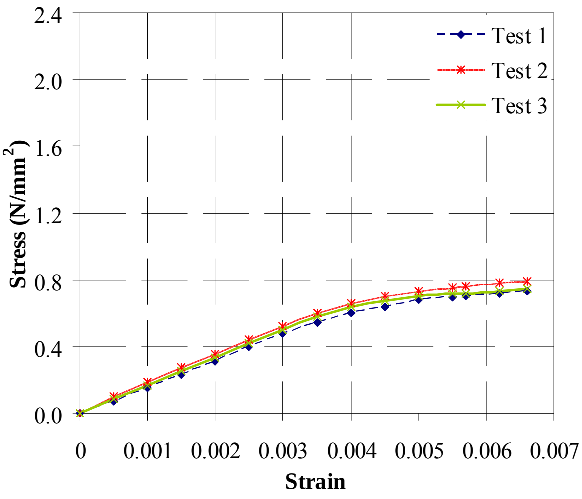

Figure 8. *Stress-strain relationship for foamed concrete of 650 kg/m^3 at 600°C*

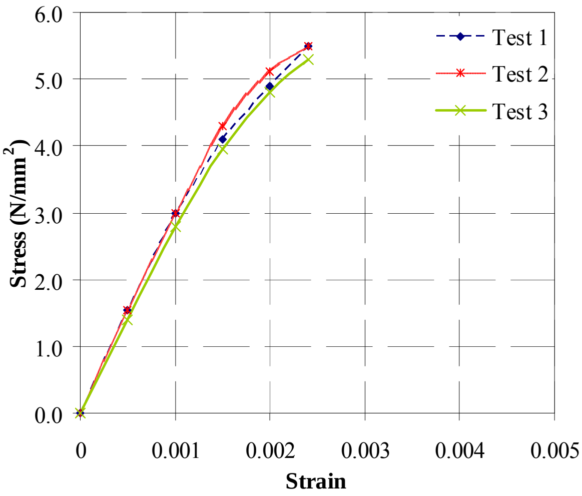

Figure 9. *Stress-strain relationship for foamed concrete of 1000 kg/m^3 at ambient temperature*

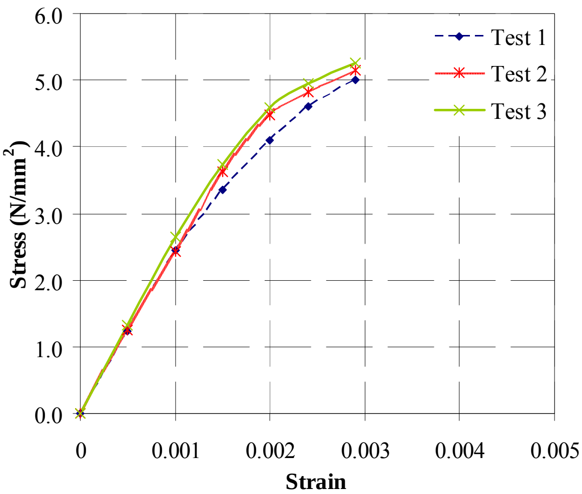

Figure 10. *Stress-strain relationship for foamed concrete of 1000 kg/m^3 at 200°C*

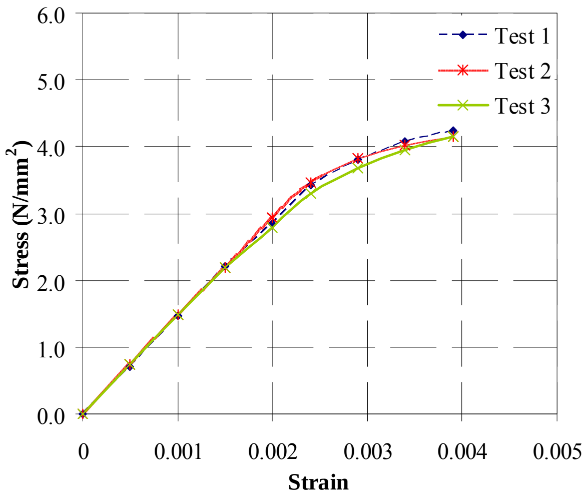

Figure 11. *Stress-strain relationship for foamed concrete of 1000 kg/m^3 at 400°C*

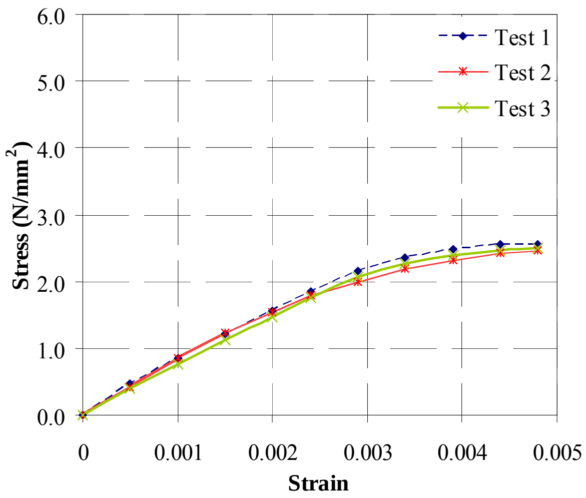

Figure 12. *Stress-strain relationship for foamed concrete of 1000 kg/m^3 at 600°C*

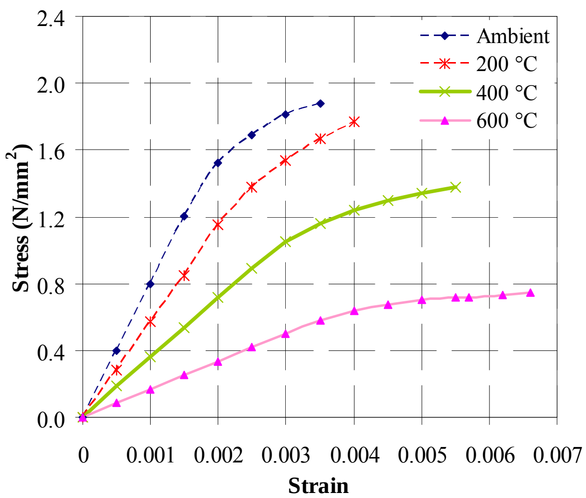

Figure 13. *Average stress-strain relationships for foamed concrete of 650 kg/m^3 at different temperatures*

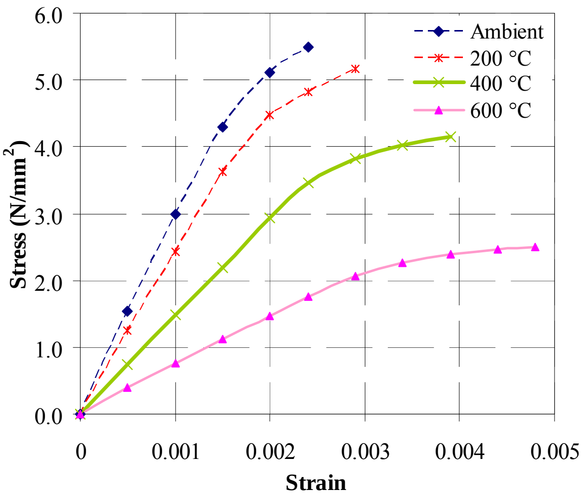

Figure 14. *Average stress-strain relationships for foamed concrete of 1000 kg/m^3 at different temperatures*

For both densities at all temperature levels, the ascending branch was linear for stress up to 75% of the peak strength. The strain corresponding to the peak strength increased at increasing temperatures. For foamed concrete of 650 kg/m^3 density, the maximum strains were 0.0034, 0.0039, 0.0055 and 0.0066 at ambient temperature, 200°C, 400°C and 600°C respectively; for the 1000 kg/m^3 density, the corresponding values were 0.0024, 0.0029, 0.0039 and 0.0048 at ambient, 200°C, 400°C and 600°C respectively. The increase in strain results from opening of cracks initiated by the heating at higher temperatures. Table 1 shows, for both densities and all temperatures, the elastic strain at the maximum stress, the maximum strain at the maximum stress and the ratio of these two strains. It appears that an average constant ratio of about 1.78 may be used for all cases.

Table 1. *Elastic strain at the maximum stress, maximum strain at the maximum stress and the ratio of these two strains for both densities at different temperatures*

Density (kg/m^3)	Temperature (°C)	Elastic strain at maximum stress	Maximum strain at maximum stress	Ratio of maximum strain at peak stress to elastic strain at peak stress
650	Ambient	0.0019	0.0034	1.79
	200	0.0022	0.0039	1.77
	400	0.0030	0.0055	1.83
	600	0.0037	0.0066	1.78
1000	Ambient	0.0013	0.0024	1.85
	200	0.0016	0.0029	1.81
	400	0.0023	0.0039	1.70
	600	0.0028	0.0048	1.71

4. CONCLUSIONS

This paper has presented the results of an extensive series of experimental studies to attain stress-strain relationship of foamed concrete at high temperatures. Compressive cylinder tests were carried out for a range of foamed concrete densities at different temperatures from ambient up to 600^{o}C. The primary mechanism causing strength and stiffness degradation is microcracking, which occurs as water expands and evaporates from the porous body. As expected, reducing the density of foamed concrete reduces its strength. For both densities at all temperature levels, the ascending branch was linear for stress up to 75% of the peak strength. The strain equivalent to the peak strength increased at increasing temperatures. The increase in strain results from opening of cracks instigated by the heating at higher temperatures.

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ASSESSING THE GREEN ROOF TECHNOLOGY IN GREEN BUILDING RATING SYSTEMS

Lee Xia Sheng[1], Ati Rosemary Binti Mohd Ariffin[2], Hazreena Binti Hussein[3]

[1] School of Architecture, Building & Design, Taylor's University;

[2, 3] Faculty of Built Environment, University of Malaya

[1]leexiasheng@gmail.com , [2]atiarchitectureum@gmail.com, [3]reenahussein@googlemail.com

ABSTRACT: Green roofs or vegetated roofs are becoming popular for sustainable development. On one hand, research shows that green roofs have numerous environmental benefits such as reduce flood risk, improve rainwater runoff quality, mitigate urban heat island, building energy saving and provide urban wildlife habitat. On the other hand, the development and utilisation of green building rating systems are crucial to appraise existing and new green buildings. The research was conducted foreseeing that the criteria and weight of scoring in such rating systems certainly will influence built environment stakeholders' decision in pursuing green building technologies. A sufficient weight of scoring in the rating system will stimulate the implementation of green roof technology for the construction projects that are adopting that particular green building rating system. However, insufficient weight of scoring will make green roof technology less attractive and eventually being left out in the sustainable development process even though this technology have the potential to mitigate current environmental issues. This paper explores the approach of assessing green building technologies based on rating systems to measure its performance and potential, with an aim to clarify the role and position of green roof technology in various green building rating systems including Leadership in Energy and Environmental Design (LEED), Green Building Index (GBI) and Singapore Building Construction Authority Green Mark. The research methodology involves analysing the relevance and scoring performance of green roof based on the criteria stated in green building rating systems such as sustainable site planning and management, materials and resources, water efficiency and innovation. This paper will also explore the approach of assessing green building technologies based on rating systems to measure its performance and potential.

Keywords: green roof, vegetated roof, Green Building Rating Systems, Green Building Index, LEED, BCA Green Mark, Sustainable Development

1. INTRODUCTION

The scoring performance of green roof technology in various green building rating systems is an important factor influencing built environment stakeholders' decision in whether to pursue this particular green building technology for sustainable development.

1.1 Green Roof's Role in Sustainable Development

The flood risk in Malaysia is increasing due to rapid urbanization of catchments for example urbanized areas. Georgetown and many parts of the places in Kuala Lumpur are so frequently inundated is because of over-development of river valleys (Ngai, 1997). Impermeable surface such as hard roof and pavement are replacing green areas, accelerating the rainwater peak runoff that increase the flood risk in the event of heavy monsoonal and convectional rainfall. Table 1 show that the runoff for conventional roof is very high ranging 0.9-0.95. However, implementation of the green roofs resembling the flat soil with vegetation category ranging 0.1-0.6 and flat lawns with heavy soil category ranging 0.13-0.17 on certain extend will greatly reduce the peak rainwater runoff and flood risk. Therefore green roofs can be very effective tools to mitigate rainwater runoff and without the construction of large capacity and costly drainage system.

Table 1. *Typical Values of Runoff Coefficients (Waterfall, 1998)*

Type of Surface	High	Low
Roof: Metal, gravel, fiber glass, mineral	 0.95	 0.9
Paving: Concrete, asphalt	 1.00	 0.9
Gravel	0.7	0.25
Soil: Flat, bare Flat with vegetation	 0.75 0.6	 0.2 0.1
Lawns: Flat, sandy soil Flat, heavy soil	 0.10 0.17	 0.05 0.13

Urban Heat Island (UHI) effect is the phenomena whereby the city centre area exhibits higher temperature as compared to rural or suburban areas. The increase in anthropogenic heat emissions from combustion of fuel, vehicle emissions and air conditioning, the decrease in green spaces and water; and increase in manmade structures and pavements are the main causes of UHI (Wong, 2002). Cities in Malaysia are facing UHI effect as more and more green spaces are taken up for development. Given the hot and humid condition in Malaysia, green roofs can provide consistant passive thermal protection for buildings and environment. If green roofs are being implemented on a sufficient scale, they have the potential to help mitigating the uprising global warming effects.

Generally, More than half of solar gain by low height building like a typical terraced house is through its roof, as the roof plane is the part of a building that receive the most solar radiation and for the longest duration through the day. Research shows that buildings built not complying to the 2006 UK building regulations will have much lower U-Values associated with poor roof insulation. These findings encourage retrofitting old buildings without good insulation in Malaysia with green roofs. New construction should also consider green roof as a green building design approach at the same time saving the cost for conventional roof insulation (Cartleton et al, 2010). The energy benefits provided by the green roof options also make a noteworthy impact in the life cycle assessment (Kosareo, 2007).

Green roofs also have the potential to be a wild life habitat in urban area. Iinvestigations have indicated that green roof technology may lead to significant gains in biodiversity. Research shows that numerous species of spiders and beetles in the International Union for Conservation of Nature (IUCN) Red List have been found on green flat-roof habitats in Europe (Brenneisen, 2003). There is also evidence for the habitat potential of green roofs for endangered bird species (Brenneisen, 2003; Baumann, 2006). Until now, little consideration has been given to the intangible ecological functions that green roofs may perform as wild life habitat, this maybe due to the current green building technologies that are economically driven.

Figure 1. *A northern lapwing (Vanellus vanellus) on the green roof in Steinhausen,Canton Zoug. (Photo by A. Kaufmann)*

Other research shows that green roofs house a large swathe of invertebrates in London, where at least 10% of which are rare or scarce, indicating the potential of green roofs' as artificial habitats is vast (Kadas, 2006).

From an aesthetic perspective the primary application of green roof is to provide a visually interesting vegetation layer of diverse texture and seasonal colour replacing a rock ballast or dark surface (Weiler, 2009). In fact the deep aesthetic discontent of concerned citizens and environmental activist with the status quo of the built environment is the trigger for Germany's green roof movement (Werthmann, 2007).

1.2 Green Building Rating Systems

Some expert has strong argument that green building is or will soon become a megatrend and building rating systems add objectivity and credibility to the process by offering standards and certification (Rock, 2010) . For credibility, a business must have clearly articulated, independently quantifiable and verifiable standards, and it is not enough to have a set of published standard, and at same time self-evaluate and self-monitor the standards put in place. (Melaver; Mueller, 2009). Therefore a reliable rating system must have a standard by which success is defined and by having quantifiable components, and its components must be reviewed and verified by a third party. Only by doing this, people can compare and assess green buildings on a reliable and independent platform.

In the US and Canada, a commercial green building is generally considered to be the one certified by the LEED green building rating system of the US Green Building Council (USGBC) or Canada Green Building Council. More than 98 percent of the certified green buildings in both countries come from this system (Yudelson, 2008). It is foreseeable that the similar will happen in Malaysia where people will recognize a building as green building only if the building is evaluated and certified by similar green building rating system.

Knowing that a reliable and independent platform is needed, Malaysia's construction industry players are absorbing other countries' experience and at the same time developing elements that adapt local situations. The Green Building Index (GBI) is one of the myriad green rating system that has already been adopted by the UK (BREAM), US (LEED), Singapore (Green Mark), Japan (CASBEE), Australia and New Zealand (Green Star). The GBI and as well as Green Mark are pioneering systems for measuring sustainability levels of buildings in a tropical zone.These evaluation and certification with these two rating systems come with cost just like other green rating systems.

Comparison study between green building rating systems shows that one important gap appears to exist is many of the rating criteria are independently rated by cut-off values lacking an assessment of the tradeoffs between them. As a result, one may find two different combinations of scores that leading to a fulfillment of the same requirement (Smith et al, 2006). This study acknowledges such important gap and explores the impact of it to green roof technology and other green building technologies.

1.3 Impact of Green Building Rating System

As some green building rating system are put into implementation for years and steadily evolve into a mature system, researchers start to study about the various impacts of such rating system on constructors(Mago, 2007), small to medium sized enterprises(Lisowski, 2006) and higher education institutions (Chance, 2010). Researcher also highlight about the emerging significance of green building rating system for example LEED but little is known of the changes that have taken place from such implementation. This raise the question of some professionals adopting green building rating system as an article of faith when critical assessment seldom being carry out to evaluate the actual effectiveness of the green building rating system criteria (Zukowski, 2005). Therefore researchers and built environment stake holders should assess and improve the green building rating systems, considering that these systems will have significant influence on the direction of green building implementation and sustainable development as a whole.

The criteria and weight of scoring in green building rating systems certainly will influence built environment stakeholders' decision in pursuing green building technologies. A sufficient weight of scoring in the rating system will stimulate the implementation of green roof technology for the construction projects that are adopting that particular green building rating system. However, insufficient weight of scoring will make green roof technology less attractive and eventually being left out in the sustainable development process even though this technology have the potential to mitigate current environmental issues.

2. METHODOLOGY

The study methodology is partially adopted from a research exploring outer space technologies for sustainable buildings (Low, 2009) and will assess the potential score that can be achieved by the implementation of green roof technology based on various criteria and items in 3 different green building rating systems. The study is a simplified assessment just to explore the scoring performance of green roof technology under green building rating systems. The 3 selected rating systems are Leadership in Energy and Environmental Design (LEED), Green Building Index (GBI) and Singapore Building Construction Authority Green Mark and the assessment will based on the Non-Residential New Construction category. The potential scores will be compared in terms of the scoring in various criteria and also the maximum potential scoring weight.

2.1 Criteria in LEED, GBI and Green Mark

The three selected green building rating systems have varies criteria and weight of scoring.

***Table 2.1a**. Leadership in Energy and Environmental Design-LEED 2009 for New Construction and Major Renovations Criteria*

	Criteria	Possible Points
1	Sustainable Sites (SS)	26
2	Water Efficiency (WE)	10
3	Energy and Atmosphere (EA)	35
4	Materials and Resources (MR)	14
5	Indoor Environmental Quality (IEQ)	15
6	Regional Priority Credits (RP)	6
7	Innovation and Design Process (ID)	4
	Total	110

***Table 2.1b.** Green Building Index Non-Residential New Construction Criteria*

	Criteria	Possible Points
1	Energy Efficiency (EE)	35
2	Indoor Environmental Quality (EQ)	21
3	Sustainable Site Planning & Management (SM)	16
4	Materials and Resources (MR)	11
5	Water Efficiency (WE)	10
6	Innovation (IN)	7
	Total	100

***Table 2.1c.** Singapore Building Construction Authority Green Mark*

New Non Residential Building Criteria

	Criteria	Possible Points
1	Part 1 Energy Efficiency	116
2	Part 2 Water Efficiency	17
3	Part 3 Environmental Protection	42
4	Part 4 Indoor Environmental Quality	8
5	Part 5 Other Green Features	7
	Total	190

The tables above will be explained in detail in section 2.2.

2.2 Green Roof Technology Potential Scoring in Green Building Rating System

The assessment of green roof technology will base on the requirements of LEED, GBI and Green Mark stated in their criteria and specific items. This study acknowledge the complex assessment process for each rating system, uniqueness of every green building project and also different possible outcome due to different opinion of assessor. Therefore, the focus of this study is not to do a detail evaluation on particular green building projects but to explore the scoring performance of green roof technology under different system. The scoring performance will have a significant impact influencing built environment stakeholders' decision in pursuing green roof technology.

Whenever green roof technology is stated as a criteria requirement fulfilment option, the assessment will consider green roof technology as the primary technology and award the potential maximum scoring points. In situations where green roof technology potentially serves only as a supporting technology in certain criteria and specific items, a pre-set of points will be deem as the contribution from green roof technology.

The potential scores are summarised and put in tables as follow:

***Table 2.2a**. Green Roof Technology Potential Score in Leadership in Energy and Environmental Design-LEED 2009 New Construction and Major Renovations*

	Criteria / Items	Points
	Sustainable Sites (SS)	
1.	SS Credit 5.1: Site Development—Protect or Restore Habitat	1
	SS Credit 5.2: Site Development—Maximize Open Space	1
	SS Credit 6.1: Stormwater Design—Quantity Control	1
	SS Credit 6.2: Stormwater Design—Quality Control	1
	SS Credit 7.2: Heat Island Effect—Roof	1
	Water Efficiency (WE)	
	WE Credit 1: Water Efficient Landscaping	2
	Energy and Atmosphere (EA)	-
	Materials and Resources (MR)	
	MR Credit 4: Recycled Content	0.5*
	Innovation and Design Process (ID)	
	ID Credit 1: Innovation in Design	1
	Regional Priority Credits (RP)	-
	Total	**8.5 (8.0**)**

*Green roof as supporting tool, **Points excluding Green roof as supporting tool

***Table 2.2b.** Green Roof Technology Potential Score in Green Building Index Version 1.0 Non-Residential New Construction (NRNC)*

	Criteria / Items	Points
	Energy Efficiency (EE)	
1.	EE1 Minimum EE Performance	0.5*
	Indoor Environmental Quality (EQ)	
	EQ12 External View	0.5*
	Sustainable Site Planning & Management (SM)	
	SM4 Environment Management: (A)Conservation (B)Open Space	2
	SM11 Stormwater Design – Quantity & Quality Control	1
	SM12 Greenery & Roof	2
	Materials and Resources (MR)	
	MR2 Recycled Content Material	0.5*
	MR3 Regional Material	0.5*
	Water Efficiency (WE)	
	WE3 Water Efficient Landscaping	0.5*
	Innovation (IN)	
	IN1 Innovation	1
	Total	**8.5 (6.0**)**

*Green roof as supporting tool, **Points excluding Green roof as supporting tool

Table 2.2b. *Green Roof Technology Potential Score in - BCA Green Mark for New Non-Residential Buildings (Version NRB/4.0)*

	Criteria / Items	Points
1.	Part 1 Energy Efficiency NRB 1-1 Thermal Performance of Building Envelope Part 2 Water Efficiency NRB 2-3 Irrigation System and Landscaping Part 3 Environmental Protection NRB 3-3 Greenery Provision NRB 3-7 Stormwater Management Part 4 Indoor Environmental Quality Part 5 Other Green Features NRB 5-1 Green Features and Innovations	 8 (2*) 0.5* 3 (1*) 3 (1*) - 1
	Total	**15.5 (11.5**)**

*Green roof as supporting tool, **Points excluding Green roof as supporting tool

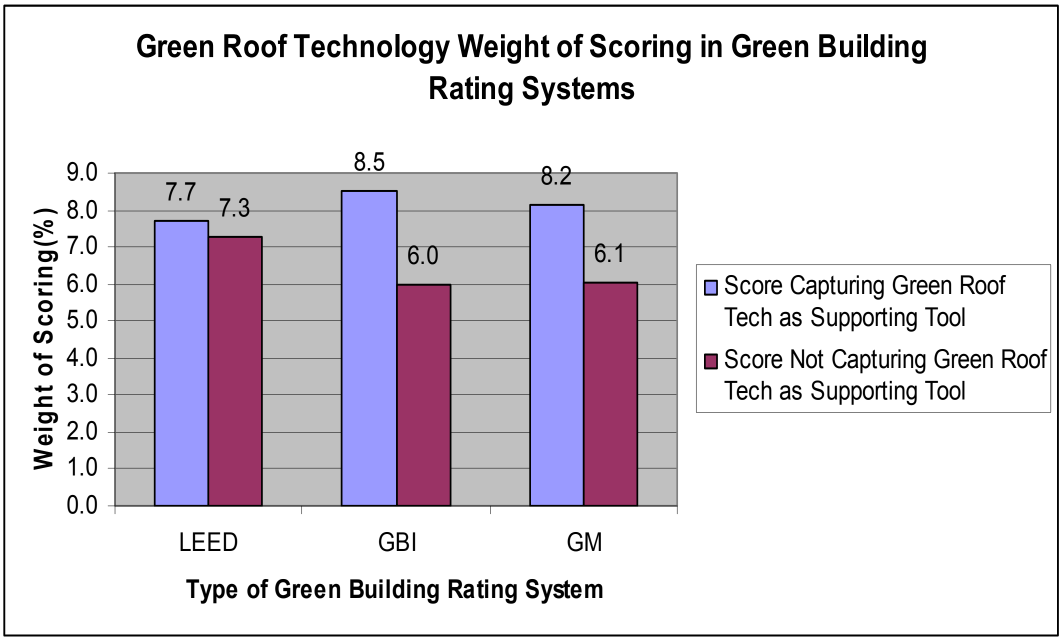

Figure 2. *Green Roof Technology Potential Weight of Scoring in LEED, GBI and GM*

Generally green roof technology have a weight of scoring ranging from 7.7% - 8.5% if the rating systems able to capture the role of green roof technology as a supporting tool, and a weight of scoring ranging from 6.0% - 7.3% if the rating systems fail to capture the role of green roof technology as a supporting tool. LEED scoring criteria is the rating system that most unlikely to overlook or missing out green roof technology as supporting tool in green building assessment with a difference of 0.1%. GBI scoring criteria is the rating system that most likely to overlook or missing out green roof technology as supporting tool in green building assessment with a difference of 2.5%.

3. CONCLUSIONS

The study is a simplified assessment to explore the possibility of using green building rating systems to gauge green roof technology performance. This assessment can be done on other green building technologies such as rainwater harvesting system (Mohammed, T. A. et al, 2007), and the weight of scoring of different technologies can be used as an initial indication of built environment stakeholders' decision in pursuing green building technologies. Built environment stake holders can use this simple assessment to monitor the influence and impact of a particular green building technology. Green building rating system researchers and developers can also utilised this simple assessment to improve the current rating system on capturing green building technology as a supporting tool for other sustainable development approach.

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SUITABLE CRITERIA FOR THE LOCATION OF GATED COMMUNITY HOUSING

Zurinah Binti Tahir[1] and Khadijah Binti Hussin[2]

[1,2]Department of Land Administration and Development, Universiti Teknologi Malaysia, Skudai Malaysia

[1]zurinah2live@utm.my , [2]khadijah@utm.my

ABSTRACT: Gated Communities set up without proper planning in housing developments could have a negative impact on urban planning and management. Issues arising from Gated Communities include matters relating to congestion, town management and the role of the local authorities. Looking at the current scenario, the planning that goes into determining suitable locations for housing development projects has not made the best use of technological tools. Consequently, decisions that are made are often unsuitable due to the lack of data or information that reflects the true spatial orientation and conditions of the selected location. This paper discusses the criteria that have been adopted in approving the development of existing Gated Communities. As a basis for developing analytical models for residential space usage and occupancy, such criteria would serve to identify areas suitable for the development of Gated Communities through the application of the Geographic Information System (GIS). The system of 'location information' being developed is very important to the Government, especially to urban planners in obtaining information related to location, area, and address displayed in the form of maps. Such mapping information is very useful, especially when presented in a simplified format and shared by many users. The housing industry in Malaysia would greatly benefit by the application of such a system in the planning of gated communities.

Keywords: analytic hierarchy process, approval criteria, gated community, Geographic Information System, housing, Multi-Criteria Analysis, Urban Planning, Weighted Linear Combination

1. INTRODUCTION

Modern urban planning must incorporate a long-term programme of planned housing development. To ensure that housing development is properly planned, especially with regard to the development of the gated communities, the government and the private sector need to develop an appropriate management system to enable its efficient operation and supervision. Rapid progress in information technology has brought many advantages and benefits in the areas of urban planning, for example, in the Geographic Information Systems (GIS) that has changed the way planning decisions are made.

In recent years, the 'Gated Community' housing concept is becoming increasingly well-received as 'landed gated communities' around the world, including the United States. This trend has extended to various parts of Europe, such as Portugal, Poland and Scandinavia (Nicholls, 2006). As the Gated Community housing concept is still new in Malaysia, there are no accurate statistics for the number of such communities in this country. Nevertheless, the development of these communities in Malaysia is gaining in popularity, especially in big cities like Kuala Lumpur, Penang and Johor Bahru. This is true also in other areas of high property value (Ismar M.S. Usman, Nur Akmal Goh Abdullah, Zuhairuse Md. Darus, Nik Lukman Nik Ibrahim, Mazlan Mohd Tahir, Abdul Halim Ismail, Azimin Samsul Tazilan, 2006).

There is presently no comprehensive legislation regulating gated housing communities in the country. There are even residential areas that exist not only as gated communities, but as 'double gated communities'. The concept of the double gated community falls outside existing provisions governing gated communities. Many Gated Development Communities that have been planned primarily for enhanced security have in fact strayed from this original planning concept. If a gated community development is approved without proper study and regulation, development in the surrounding areas are likely to be affected in the future. This could have serious implications for subsequent development in the adjacent areas. The unregulated approval of Gated Communities could therefore have negative consequences for the long-term planning and management of the city.

Comprehensive planning is needed before any decision is made, taking into account the many relevant factors and sensitivities, as well as the views of all the parties involved (Mohd Sanusi, 2007). Establishing a suitable location for the development of a Gated Community requires a thorough review and analysis. Such planning is necessary to ensure that the area identified meets planning requirements in terms of the economic, social and environmental aspects.

2. METHODOLOGY

A study to assess the suitability of a location for a Gated Community entails several implementation stages that involve data acquisition, modeling and analysis (Figure 1). The GIS system, a key tool in this study, will be employed in the development of databases and in spatial analysis. In the analysis stage, two techniques, *viz.* the Multi-Criteria Analysis (MCA) and the Weighted Linear Combination (WLC), will principally be employed in ascertaining the suitability of a proposed location for the development of a Gated Community.

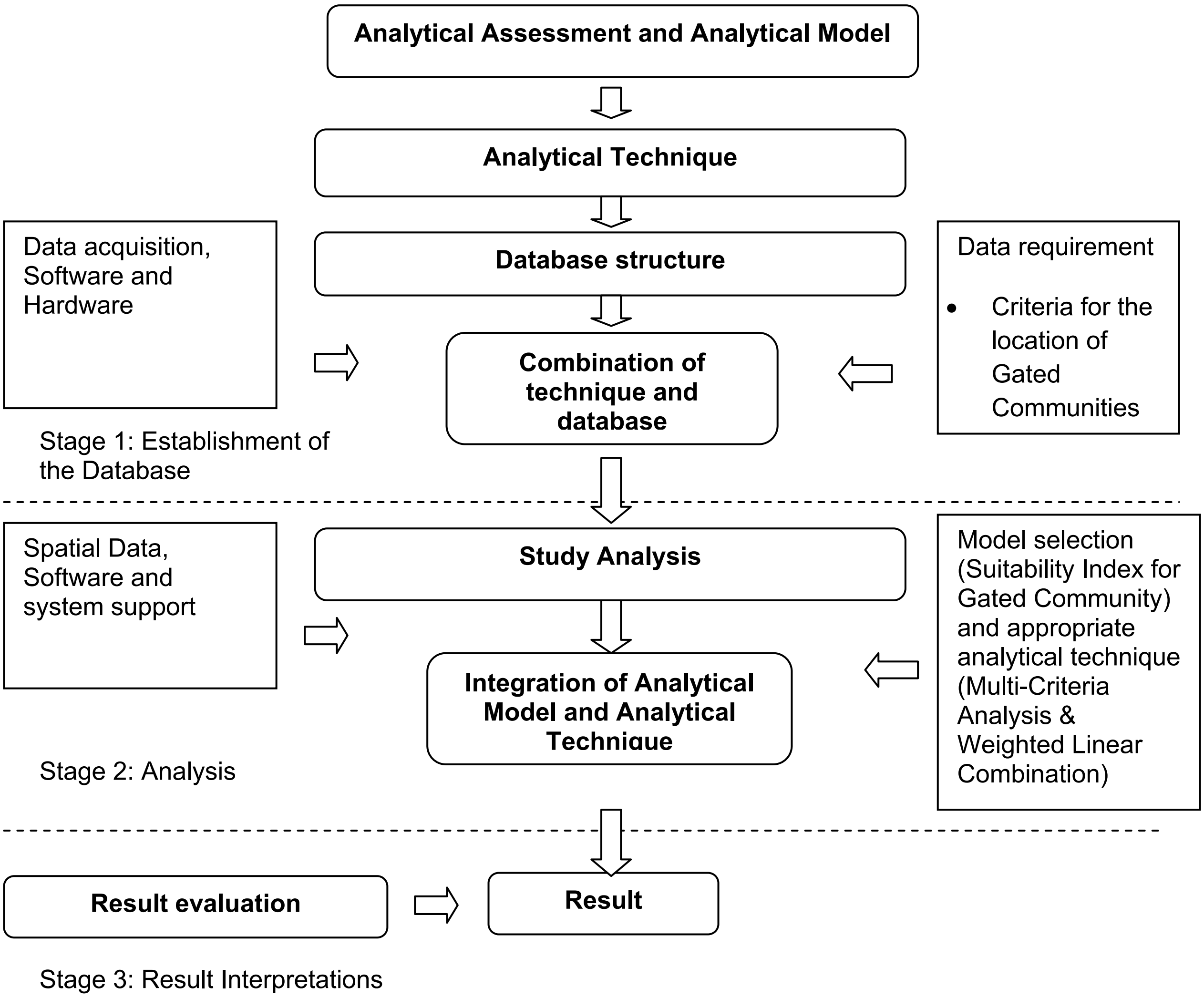

Figure 1. *The Methodology of the study*

3. EXPERIMENTAL PROGRAMMES

The developmental model uses the GIS software to identify the selection criteria to be used to assess the suitability of a proposed Gated Housing Community. Studies that determine appropriate selection criteria for sustainable development will be prepared in accordance with current principles and concepts in urban planning. The present study focuses on analyzing various physical aspects, including land suitability, to identify appropriate locations for the development of Gated Housing Communities in the District of Johor Bharu (Figure 2).

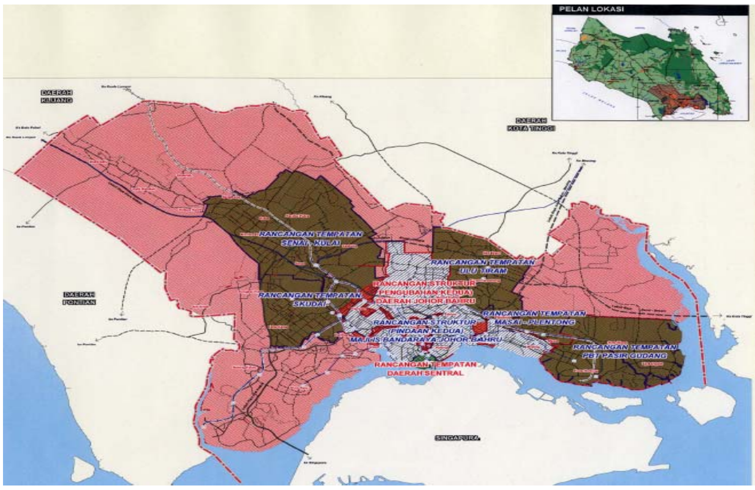

Figure 2: *Location plan of the Johor Bharu District*

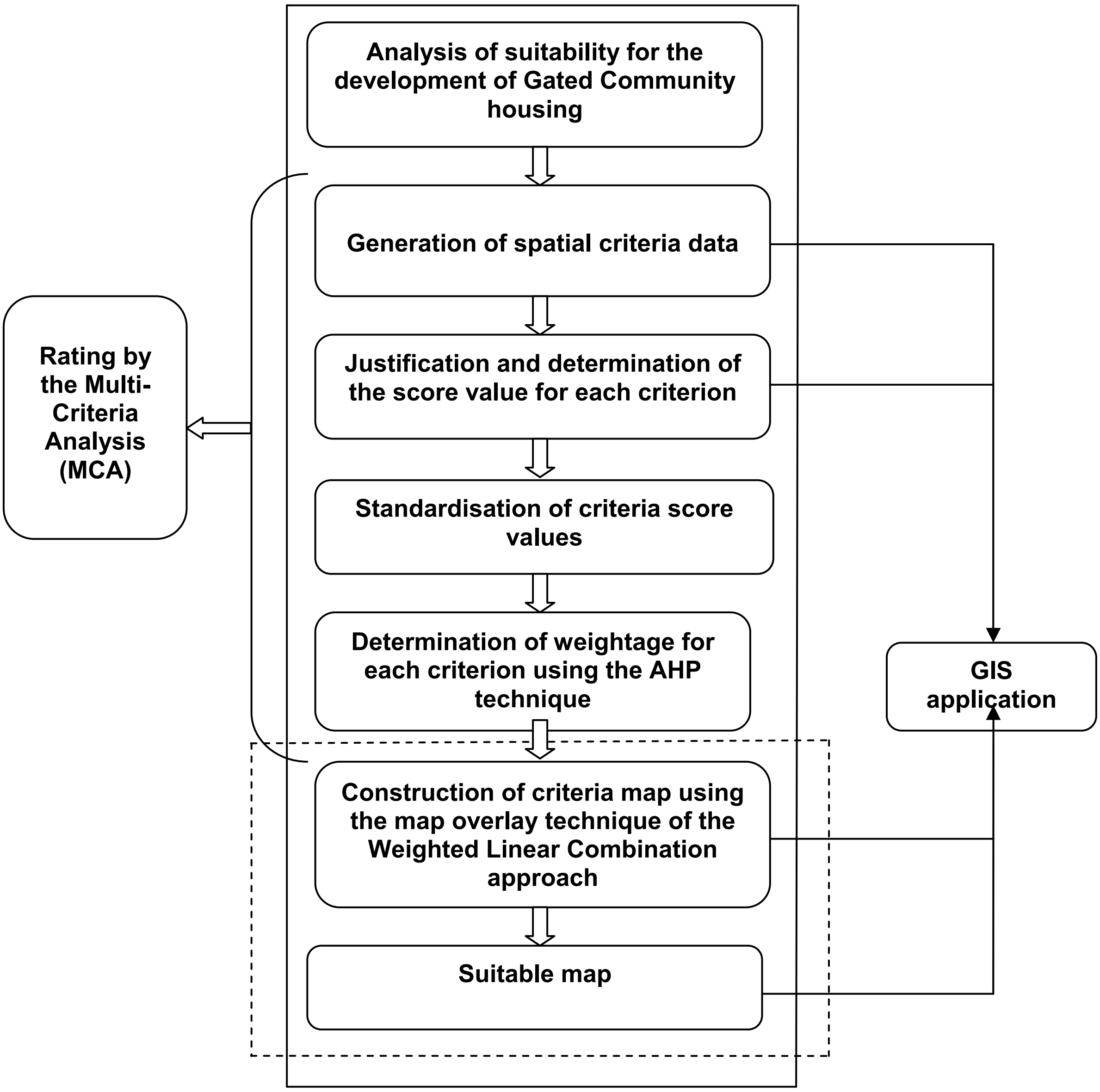

Figure 3. *Multi-Criteria Analysis (MCA) to determine the suitability of Gated Community locations*

3.1 Multi-Criteria Analysis (MCA)

The multi-criteria assessment of the GIS raster model is a decision-making process that combines spatial data in accordance with their importance. This procedure involves the production of maps of the relevant criteria (including constraining factors) used to identify suitable locations for Gated Community housing developments. The availability of relevant information is crucial to rational decision making and lays the foundation for better management efficiency in the future.

The approach for this analysis is based on the Multi-Criteria Analysis (MCA) concept while the Analytic Hierarchy Process (AHP) technique will be used in its implementation. This technique links with the GIS to produce a weightage for each selected criterion (Figure 3). The AHP technique enhances the rationality and objectivity behind the decisions taken. To obtain a meaningful analysis that would be useful for planning purposes, it is essential that the MCA results are accurate before they are incorporated into the GIS database. Selection of the criteria for the suitability of a Gated Community location will also depend on the developmental status and topography of the particular housing area. Thirteen such criteria have been identified for Gated Community housing developments in Johor Bharu, namely:

i) Crime rates
ii) Urban areas
iii) Network of roads
iv) Proximity to villages
v) Populated areas
vi) Existing facilities (utilities)
vii) Neighbourhood and surroundings
viii) Appropriate land use
ix) Accessibility (distance and cost)
x) Type of Gated Community housing
xi) Tenure
xii) Environmentally sensitive areas
xiii) Topography

3.2 Suitability Index Model

A Suitability Index model can be constructed using the various functions of the GIS system as shown in Figure 4. These functions include input, display access, update and output. An appropriate Suitability Index model will be established using the Multiple Criteria Evaluation (MCE) technique in conjunction with the Weighted Linear Combination.

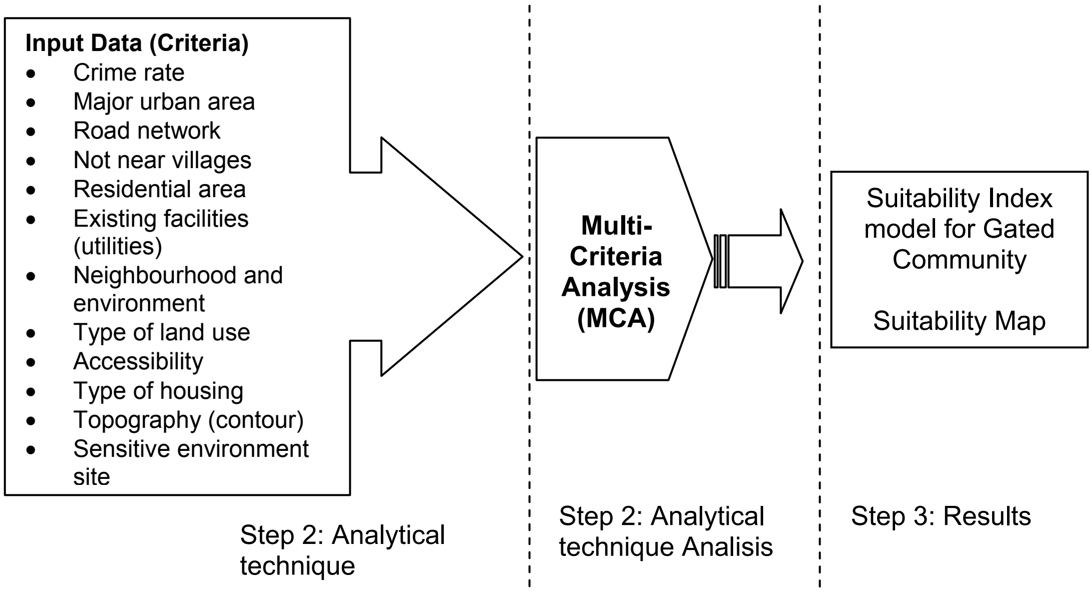

Figure 4. *Analysis of the Suitability Index model to determine the suitability of a proposed Gated Community location*

The Suitability Index model is constructed by taking into consideration the combination of factors based on 12 criteria as shown in Figure 5. This analytical model will be developed conceptually to reflect the final map overlay. The results of the GIS analysis on the criteria will be presented in the form of standardised factor maps and maps depicting constraints. Integrating the multi-criteria evaluation techniques with the GIS enables decision-makers to differentiate or to change the weightage of the attribute scores of a factor map in assessing the suitability of a location

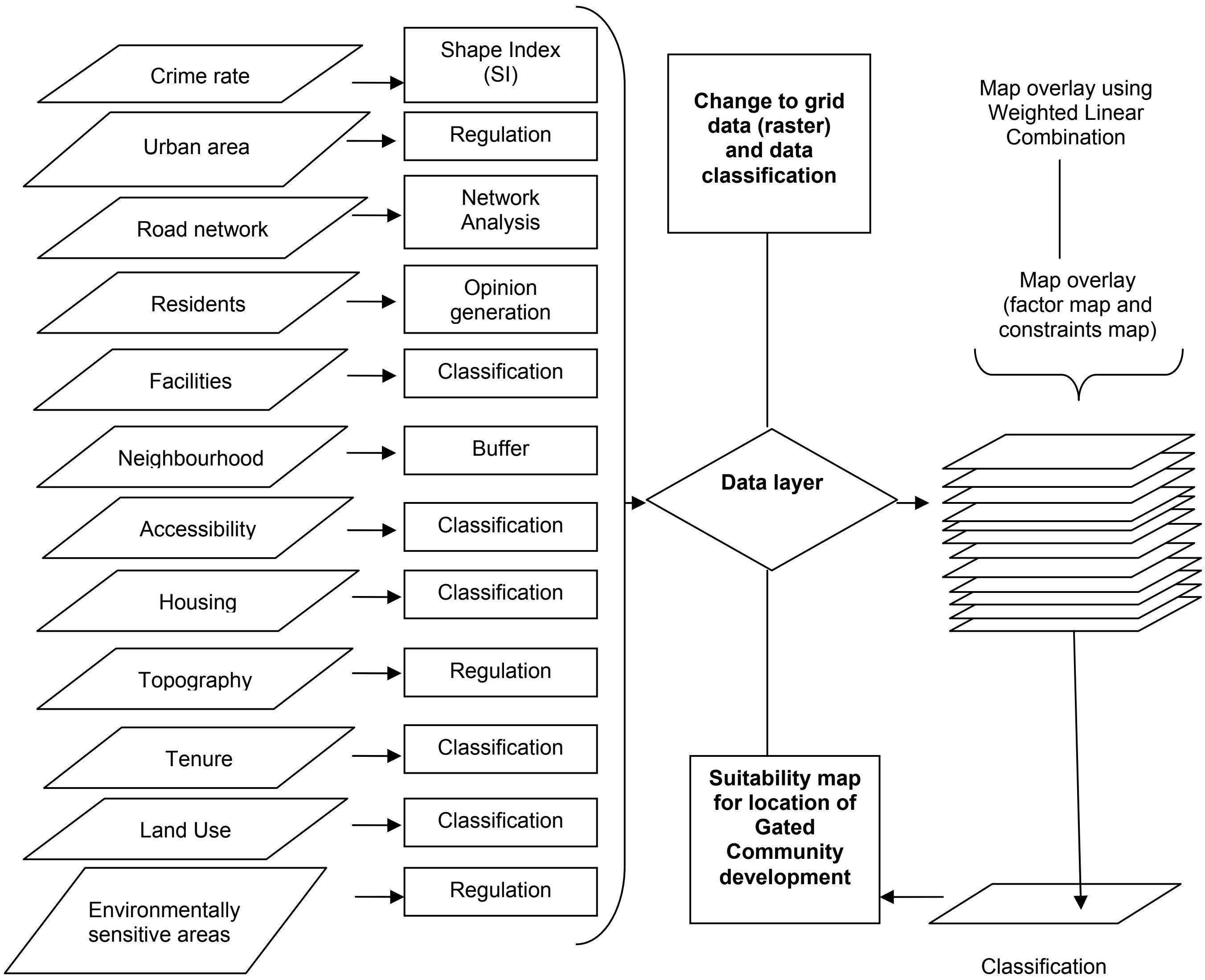

Figure 5: *Suitability Index model for the development of Gated Community housing*

4. CONCLUSIONS

The results of this study will provide an analysis of the physical aspects of the proposed location, including soil suitability, and identify criteria for acceptable locations for developing Gated Communities. The proposed criteria identified from such models will assist the local authorities in evaluating and monitoring the implementation of 'gated and guarded communities' (GC) in Johor Bharu. This will ensure that such developments do not conflict with the current tenets and practice of urban planning. At the same time, basic directives will be available to developers to design and build 'gated and guarded communities' in compliance with the guidelines and standards adopted. The ‘location information’ system being developed will assist urban planners make rational decisions. The system will yield relevant information based on set criteria for the development of future Gated Communities. Such information can be presented in a simplified map format and shared by many users. Hence urban planning, especially with regard to gated communities, will be greatly enhanced. This would augur well for the housing industry in Malaysia.

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STRESS-STRAIN RELATIONSHIP FOR MASONRY MODELLING

Ahmed Hasan Alwathaf [1], Mohd Saleh Jaafar [2],

Waleed A. Thanoon [3], Jamaloddin Noorzaei [4]

[1] Civil Engineering Department, Faculty of Engineering,

Sana'a University, Sana'a, Yemen, P. O. Box. 12544

[2, 4] Civil Engineering Department, Faculty of Engineering, University Putra Malaysia, 43400 UPM-Serdang, Malaysia

[3] College of Engineering and Architecture, University of Nizwa, Nizwa, Oman

[1]aalwathaf@yahoo.com , [1]aalwathaf@suye.ac

ABSTRACT: Stress-strain relationship of masonry material is essential to predict strength and deformation of masonry structures in analytical modelling. In this study, the best fit equation of experimental data for masonry block and grout under uniaxial compression test is proposed. Numerical technique is proposed to obtain appropriate material parameter for both ascending and descending parts. Accurate simulation for the test results is attained by the proposed equation.

Keywords: stress-strain, masonry, compression, modelling, numerical technique.

1. INTRODUCTION

Using stress-strain relationship that can describe accurately the behaviour of the masonry materials is essential to predict the actual strength and deformation of the masonry structure. This fact is more significant when the system consists of different constituents (block units, grout, mortar (if any)) in which every material has different stress-strain relation and required to be discretized separately in the finite element model to incorporate the effects of interactions of these constituents.

Several mathematical models have been developed for simulating the experimental stress-strain relation for concrete under uniaxial compression. Some of these models developed for plain concrete and other models developed for concrete under different condition such as confinement by reinforcement (Park and Paulay 1975).

In view of the fact that plain concrete is the actual status of the masonry concrete block and grout, the stress-strain relations reviewed in this section have been developed for plain concrete which is also applicable for concrete masonry block (Alwathaf, 2006). The reviewed stress-strain models are shown in Table 1 with the important features. The coefficients in the table were recommended by the respective authors in their studies (Alwathaf, 2006).

Table 1. *Reviewed Stress-Strain Relations*

Reference	Equation	Feature	Remarks
Hognestad (1951)	$\sigma = \sigma_o \left[2\frac{\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o} \right)^2 \right]$ $\sigma = \sigma_o \left[1 - 0.15 \left(\frac{\varepsilon - \varepsilon_o}{\varepsilon_u - \varepsilon_o} \right) \right]$	For ascending For descending	
Desayi & Krishnan (1964)	$\sigma = \frac{E_o \varepsilon}{1 + (\varepsilon / \varepsilon_o)^2}$	For ascending & descending	$E_{s=}\ \sigma_o / \varepsilon_o$ should be equal to 2
Saenz (1964)	$\sigma = \frac{E_o \varepsilon}{1 + (E_o / E_s - 2)(\varepsilon / \varepsilon_o) + (\varepsilon / \varepsilon_o)^2}$		(E_o/E_s) should be equal or greater than 2
Wang et al. (1978)	$\sigma = \sigma_o \left[\frac{A(\varepsilon / \varepsilon_o) + B(\varepsilon / \varepsilon_o)^2}{1 + C(\varepsilon / \varepsilon_o) + D(\varepsilon / \varepsilon_o)^2} \right]$	For ascending A=1.300501 B= -0.835818 C=-0.699498 D=0.1641812	For descending A=0.349777 B= -0.104963 C=-1.650222 D=0.895036
Carreira & Chu 1985)	$\sigma = \frac{A(\varepsilon/\varepsilon_o)\sigma_o}{A - 1 + (\varepsilon/\varepsilon_o)^A}$	For ascending & descending	$A = \frac{1}{1 - (E_s/E_o)}$
Thanoon (1997)	$\sigma = \sigma_o \left[\frac{A(\varepsilon / \varepsilon_o)}{(\varepsilon / \varepsilon_o)^3 + B(\varepsilon / \varepsilon_o)^2 + C(\varepsilon / \varepsilon_o) + D} \right]$	For ascending & descending	For plain concrete: A=1.1 B=-1.3 C= 0.75 D=0.65

2. EXPERIMENTAL TEST

2.1 Compression test of block and grout

The objectives of these tests are to find stress-strain relation and compressive strength of the interlocking block masonry constituents. Specimens of these tests are the individual concrete block units and grout cylinders. The block units used in the test specimens have been produced especially for the current study using a semi automatic block making machine.

2.2 Compression test of block and grout results

Figure 1 shows typical stress-strain curves obtained from the tests. The curves indicate that stronger material is more brittle than weaker material. This is consistent with the observed test phenomenon where the failure was quite sudden for blocks having high compressive strength. On the other hand, load decreased gradually after the peak in the grout that having low strength. First sign of failure was observed approximately at 80% of the maximum load as pulling out small parts from the block. It can be also observed that the modulus of elasticity increases with compressive strength and the descending parts of these curves have high negative slopes after the peak. In contrast to block stress-strain curves, the peak of the grout cylinders are flatter and descending portion decreases gradually with small negative slope.

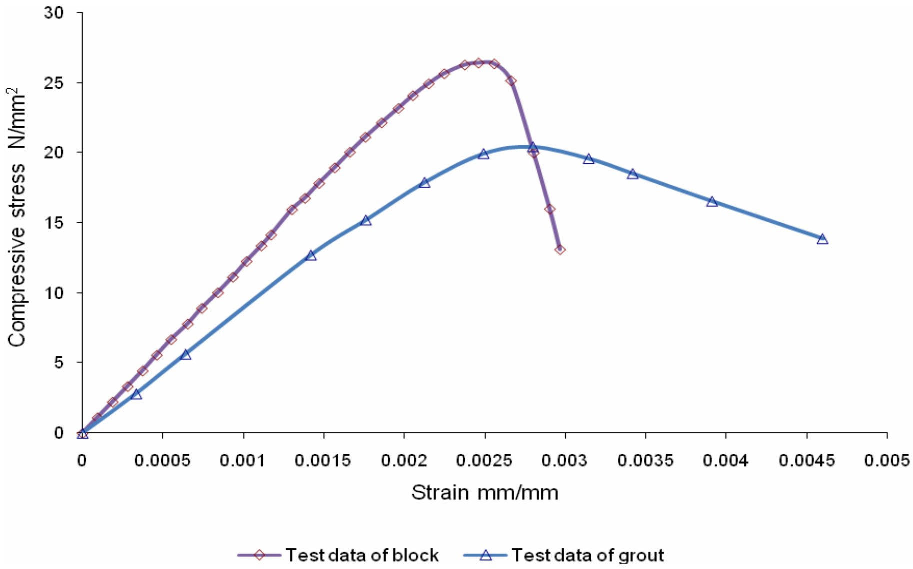

***Figure1**. Typical Stress-Strain Curve of Block and Grout under Compression Test*

3. PROPOSED STRESS-STRAIN RELATION

In this study, the best fit equation of the experimental data of masonry block and grout under uniaxial compression test for both ascending and descending parts is adopted. It can be expressed as:

$$\sigma = \frac{A(\varepsilon/\varepsilon_o)\sigma_o}{A-1+(\varepsilon/\varepsilon_o)^A} \qquad (1)$$

where

σ , ε instantaneous values of the stress and the strain respectively

σ_o , ε_o the ultimate stress (peak) and the corresponding strain respectively

A a coefficient called material parameter depends on the shape of the stress- strain diagrams.

Equation 1 is capable of simulating the stress-strain relation for different masonry materials and can be incorporated efficiently into the biaxial stress model. The material parameter, *A,* depends on the material stress-strain behaviour and takes a constant value for each material.

An accurate numerical technique has been proposed in this study to find the suitable material parameter (*A*). This technique improves the ability of Eq. 1 to describe different curvature in the ascending and also in the descending part. In this study, nonlinear regression analysis is used to obtain the material parameter (*A*) of the selected stress-strain relation based on the entire diagram test data not only the data at the origin (Carreira and Chu, 1985). Nonlinear regression is a method of finding a nonlinear model of the relationship between the dependent variable and a set of independent variables (Bates and Watts, 1998). The dependent variable in Eq. 1 is the stress ratio (σ/σ_o) after normalizing the stress using the maximum stress at the peak, σ_o. Furthermore, the independent variable is the strain ratio ($\varepsilon/\varepsilon_o$).

Figure 2 shows the experimental test data and the best fit curve drawn by Eq. 1. A comparison with the well-known formula suggested by Saenz (1964), which is frequently used for modelling of compressive stress-strain curves of masonry and concrete under biaxial stress state (Cerioni and Doinda, 1994; Chen, 1982), is also shown in Fig. 2. Unlike Eq. 1, Saenz's formula fails to represent the variation of curvatures of the stress-strain relations for different materials which in turn makes Saenz's equation more suitable for masonry macro-model as used by some researchers (Cerioni and Doinda, 1994). Another condition that restricts using Saenz's formula is the ratio of the tangential modulus of elasticity at the origin to the secant modulus at the peak which should be more than or equal to 2, otherwise concave curve at the low stress level will be produced.

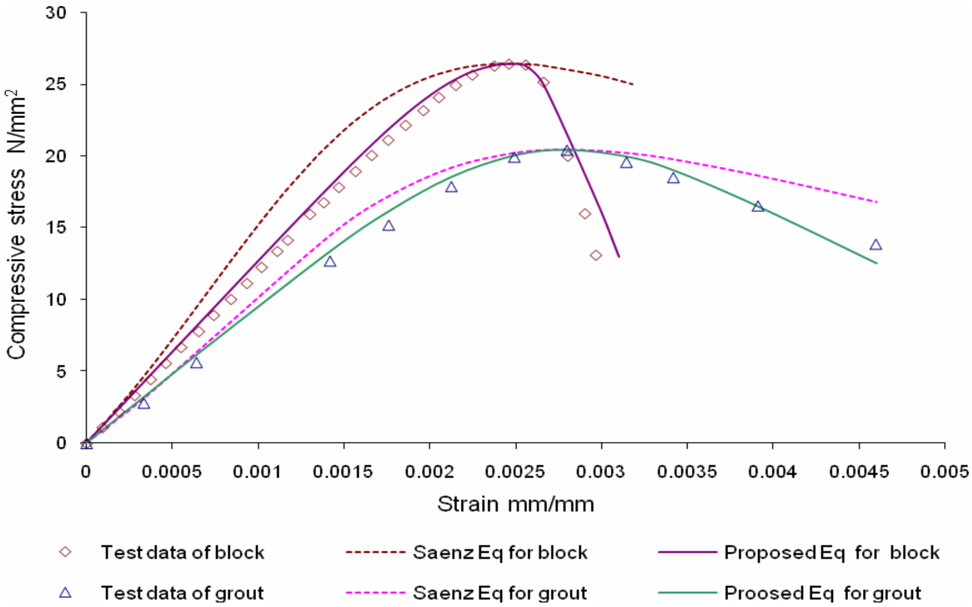

***Figure 2**. Comparison of Test Data and the Best Fit Relation*

4. CONCLUSIONS

It has been found that the stress-strain data of the masonry materials (block and grout) has been accurately fitted by the employing stress-strain equation after using the proposed numerical technique by means of the whole diagram data of the compression test. The proposed numerical technique for the material parameter yields accurate simulation for the stress-strain behaviour of the masonry material under compression for ascending and descending parts. The proposed stress-strain relational is easily incorporated in the masonry modelling for any numerical investigation.

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THE NEEDS OF INDUSTRIALISED BUILDING SYSTEM IN MALAYSIA

Muhamad Azani Yahya[1], Suriyadi Sojipto[2], Ahmad Shahrir Ismail[3]

[1, 2, 3] Civil Engineering Department, Faculty of Engineering

Universiti Pertahanan Nasional Malaysia,

57000 Kem Sungai Besi, Kuala Lumpur

[1]azani@upnm.edu.my, [2]suriyadi@upnm.edu.my, [3]shahrir@upnm.edu.my

ABSTRACT: The new approach in changes to new perspective of construction industry depending on the concept those focus on improving construction efficiency. Excellence in construction phase can help raise productivity and change the paradigm of current lacking view. The needs to be a greater concentration on achieving a better construction which meets the needs of the end user have to be focus as an improvement aspect on conventional construction process. Modernizing construction using Industrialised Building System (IBS) seems to be one of construction improvement aspect in Malaysia. IBS is a construction system that is built using pre-fabricated components. Questionnaire surveys have been distributed to 30 respondents which involved in construction phase to see the perception on IBS implementation in Malaysia. To validate the findings, schedule simulation was done on replacement elements of conventional construction process using IBS elements and the result shown IBS offer a principle of schedule compression. Construction players agreed that this method offer reduction of construction time, enhance construction efficiency, reduce construction wastage, decreased complexity and ensure the successes of project delivery.

Key Words: Industrialised Building System, schedule simulation, modernizing construction

1. INTRODUCTION

Construction is a hugely important industry. Excellence in construction phase can help raise productivity and change the paradigm of current lacking view. The needs to be a greater concentration on achieving a better construction which meets the needs of the end user have to be focus as an improvement aspect on conventional construction process. Modernizing construction using Industrialised Building System (IBS) seem to be one of construction improvement aspect in Malaysia. IBS is a construction system that is built using pre-fabricated components. The general view of the construction process is that it is in ordered, linear phenomenon, which can be organized, planned and managed top down. The frequent failures to complete construction projects on budget and schedule give rise to thinking that the process maybe not is as ordered and predictable in its nature as it may look (Bertelsen, 2002). A closer examination reveals that construction is indeed a complex, non linear and dynamic phenomenon which often exists on the edge of chaos. Construction projects are in general rich in plan failure, delays, cost overruns and grief, indeed more so than successes. The project itself as an assembly-like process was often more complicated, parallel and dynamic. The mistake is the ordered view of the surrounding world. All supply chains are believed to be made in accordance with the project's schedule. All resources such as equipments and workers are supposed to standby, ready for the project's beck and call. However, this is not the way the world operates. This happened because of the concept of site production. Every item seems to be done in-situ. Since 1998, the Construction Industry Development Board of Malaysia has been actively promoting the use of Industrialised Building System (IBS) in the Malaysian construction industry. The existing of IBS is to realign and change the paradigm of the conventional construction process which was lack of efficiency.

2. BACKGROUND OF INDUSTRIALISED BUILDING SYSTEM (IBS)

Industrialsed Building System (IBS) has already utilized in Malaysia since 1960's (IBS Digest, 2005). IBS is the way forward for construction players to make leaps and bounds progress in the construction industry. To date there has been no one commonly-accepted or agreed definition of IBS. However, there are a few definitions by researchers who studied into this area previously were found through literature. Rahman and Omar (2006) defined IBS as a construction system that is built using pre-fabricated components. The manufacturing of the components is systematically done using machine, formworks and other forms of mechanical equipment. The components are manufactured offsite and once completed will be delivered to construction sites for assembly and erection. Parid (1997) defined IBS as a system which use industrialised production technique either in the production of component or assembly of the building or both. Lessing, *et. al.* (2005) defined IBS as an integrated manufacturing and construction process with well planned organisation for efficient management, preparation and control over resources used, activities and results supported by the used of highly developed components. Trikha (1999) defined IBS as a system in which concrete components prefabricated at site or in factory are assembly to form the structure with minimum in situ construction. IBS was also defined as a set of interrelated element that act together to enable the designated performance of the building (Warszawski, 1999). Esa and Nurudin (1998) defined IBS as continuum beginning from utilising craftsmen for every aspect of construction to a system that make use of manufacturing production in order to minimise resource wastage and enhance value end users. Junid (1986) defined IBS as process by which components of building are conceived, planned and fabricated, transported and erected at site. The system includes balance combination between software and hardware component. The software element include system design, which is complex process of studying the requirement of the end user, market analysis and the development of standardize component. Chung and Kadir (2007) defined IBS as a mass production of building components either in factory or at site according to the specification with standard shape and dimensions and transport to the construction site to be re-arranged with certain standard to form a building. All the above definitions emphasized on prefabrication, off-site production and mass production of building components as the main characteristic of IBS. The focal of discussion in the study synergizes the key concept of IBS as defined as follows: A construction technique in which components are manufactured in a controlled environment (on or off site), transported, positioned and assembled into a structure with minimal additional site works (CIDB, 2003).

2.1 IBS ENVIRONMENT IN MALAYSIA

The usage of IBS has been implemented stage by stage in the building industry (Harun et. al., 2009). The construction industry and private sector assume an important role in generating wealth and improving the quality of life for Malaysians through the translation of the government’s socio-economic policies into social and economic infrastructures and building (CIDB, 2007). Meaning to say that construction industry in Malaysia was not stand alone sector but it brings together many others related sector including manufacturing. That’s why the usage of IBS should tremendously been implement to change the norm of practiced. Figure 1 defines the construction industry value chain, as it involves multiple stakeholders at different stages of the value chain and key sectors that could leverage on the construction industry.

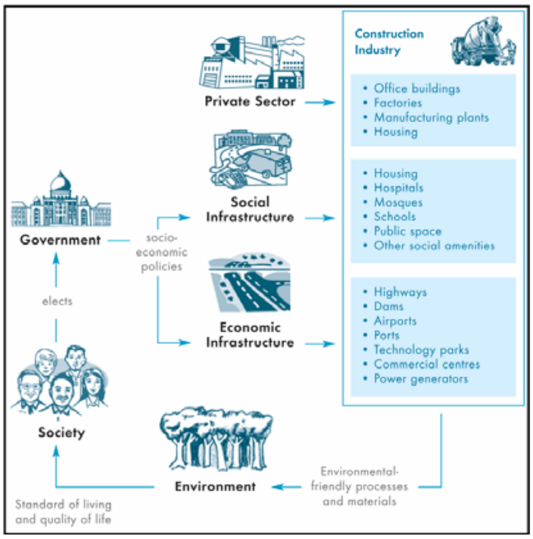

***Figure 1**. Enabler of Government’s Socio-Economic Policies.*
Source: CIMP 2006-2001, CIBD (2007)

In Malaysia, the usage of IBS gives the opportunity to the construction industry to be more matured and it can be consider as modernizing method. The IBS, which enables on-site prefabricated or building components manufactured at factories, will enable cost saving and quality improvement through the reduction of labour intensity and construction standardization. Apart from this, it offers minimal wastage, less site materials, cleaner and neater environment, controlled quality and lower construction cost. Table 1 show the benefits of IBS compare from conventional construction system.

***Table 1**. Benefits of IBS From Literature*

Item	Benefits	Supporting Statement
1	Reduce remittances by foreign workers	The Malaysian government aimed to achieve 100 percent usage of IBS and to reduce 15% of foreign workers in the construction industry (Bernama, 2006)
2	Enhance efficiency	Construction industry is driven towards the adoption of an integrated and encouraging key players in the construction industry to produce and utilize pre-fabricated and mass production of the building at their work sites. This will help to enhance the efficiency of construction process, allowing a higher productivity, quality, time and cost saving (CIDB, 2004)
3	Produce better product	IBS promises elevated levels of expertise throughout the industry, from manufacturers, installers, engineers, planners, designers and developers. The benefits of IBS will ultimately produce better products for the population (CIDB, 2003)
4	Reduce wastage, less site materials, costs, cleaner and neater environment	Enables on-site prefabricated components manufactured at factories offer that benefits (CIDB, 2003).
5	Higher quality of component	Attainable through careful selection of materials, use of advanced technology and strict quality assurance control.
6	Reduce labour at site	Prefabrication takes place at a centralized factory, thus reducing labour requirement at site. This is true when high degree of mechanization is involved (Warszawski, 1999)
7	Faster completion	Casting of precast element at factory and foundation work at site can occur simultaneously. This provides earlier occupation of the building, thus reducing interest payment or capital outlays (Waleed et. al., 2003)
8	Not affected by adverse weather condition	Construction operation is not affected by adverse weather condition because prefabricated component is done in a factory controlled environment (Waleed et. al., 2003)
9	Flexible design	Allows flexibility in architectural design in order to minimize the monotomy of repetitive facades (Warszawski, 1999). IBS provides flexibility in the design of precast element as well as in construction so that different systems may produce their own unique prefabrication construction methods (Zaini, 2000)

*Extract literature from Haron et. al., 2009

2.2 IBS Barriers in Malaysia

The building industry is often described as an industry with many problems and lack of efficiency (Alinaitwe, 2009). The common consensus of all the stakeholders of construction in Malaysia is that the IBS implementation in construction industry is still very low compared to the conventional methods. This is due to several reasons as in Table 2.

***Table 2**. Summary of barriers to adoption of IBS in Malaysia*

Item	Barriers	Supporting Statement
1	Return investment	Wide swings in building demand, high interest rate and cheap labour cost, make it difficult to justify large capital investment. At present there is an abundance of cheap foreign workers in Malaysia and contractors prefer to use labour intensive conventional building system because it is far easier to lay off workers during slack period.
2	Lack of manpower skilled	Fully prefabricated construction system requires high construction precision. Malaysian labour force still lack of skilled workers in IBS implementation.
3	Nature practices	The construction industry is very fragmented, diverse and involves many parties. Consensus is required in the use of IBS during planning stage.
4	Knowledge based	Lack of R&D in the area of novel building system that uses local materials. Majorities of IBS in Malaysia are imported from developed countries, thus driving up the construction cost. Engineering degrees in local universities seldom teach about the design and construction of IBS.
5	Low quality	The use of IBS in Japan and Sweden are so successful due to high quality and high productivity but it is the opposite in Malaysia. Previous projects constructed with IBS concept were of low quality and high construction cost.
6	Lack of incentive and awareness	Due to lack of incentive and promotion from government in the use of IBS, many architects and engineers are still unaware of the basic elements of IBS such as modular coordination.
7	Lack of scientific information	An IBS can only be accepted to practitioners if its major advantages are valuable compared to the conventional system. However, up to date, there is inadequate corroborative evidence to substantiate the benefits of IBS system. It is therefore, arguable that the implementation of IBS is particularly hindered by lack of scientific information (Badir et. al., 2002)
8	Wastage of material	Standardization of building agreements faces resistance from the construction industry due to aesthetic reservation and economic reason. One good example of this is when a 300mm thick modular standardized floor slab has to be used although a 260mm thick floor slab can achieved the similar structural performance. This results wastage of material (Waleed et. al., 2003)

*Extract literature from Haron et. al., 2009

3. OBJECTIVES

The main aim of this paper is to establish the usage of Industrialised Building System (IBS) as a modernizing method in Malaysian construction industry. To achieve this aim, the paper will flow by the objectives of:

1. To identify the implementation of IBS in construction industry;
2. To identify the perception from construction players regarding the improvement aspects by implementing IBS; and
3. To evaluate the concept of IBS to achieved time reduction in construction phase by scheduling simulation.

4. METHODOLOGY

The research methodology in this paper serves as a guide in achieving the objectives of the study and discusses in details the research procedures, from how the data is collected till how it is processed and analyzed to achieve the objectives and scopes of the study. It involves the identification and further understanding of the research topic, which consists of problem statement and research objectives. Literature review has been done on several references, either from electronic journals, books, magazines, articles and so on to further enhance the understanding on the research topic. The assembly and erection of the conceptual high rise residential building has been modelled using computer software and analysed. More emphasize is on how the scheduling on assembly time is carried out efficiently at a site using IBS components for the proposed conceptual high rise residential building. Therefore, a comparison between the work breakdown structure (WBS) between a typical high rise residential building construction using conventional construction method and the other using Industrialised Building System (IBS) components for the high-rise residential building been further analysed using Microsoft Project for Window. Finally, a conclusion will be drawn out based on the results of the analysis obtained.

5. ANALYSIS AND RESULT

The respondents that involves in this study are the individual that related to construction industry. They were called as construction player. Figure 2 show the percentage of respondent's position that involve in the questionnaire survey.

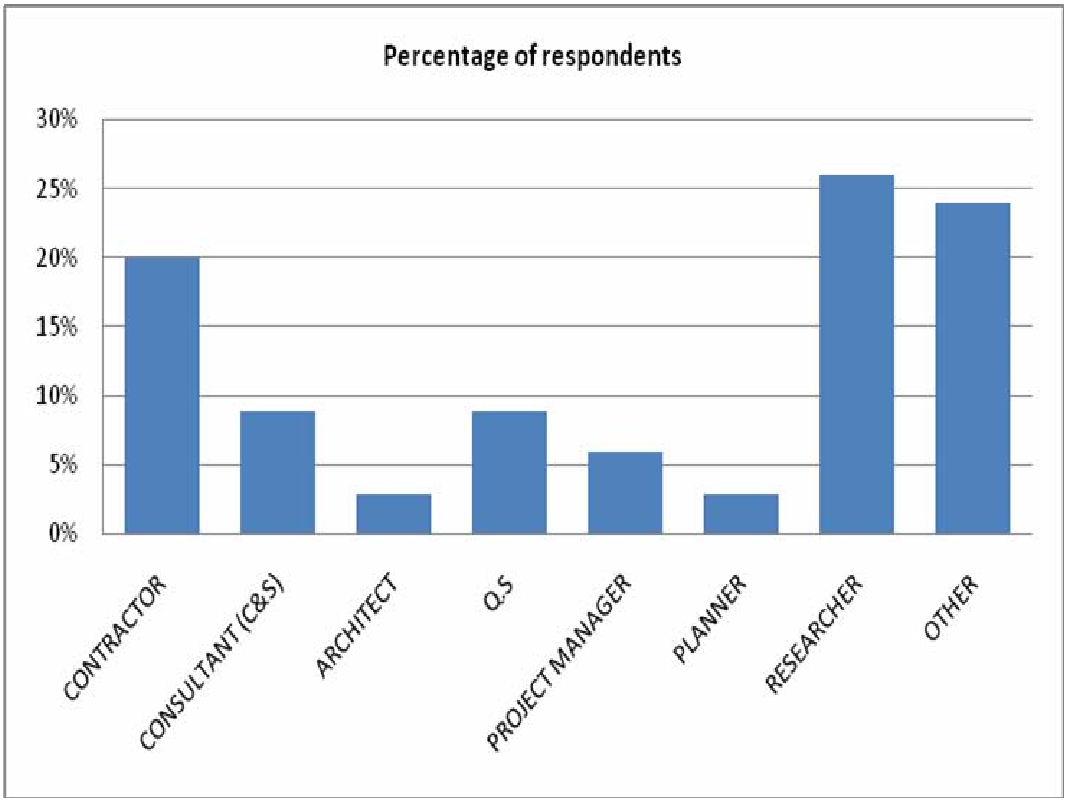

***Figure 2**. Percentage of respondent's positions than involve in questionnaire survey*

An open ended question have been ask to see the perception from respondents about the credits given to the construction industry when implementing IBS and the results summarized as in table 3.

***Table 3**. Perception of IBS on contribution to construction process flow*

IBS IMPLEMENTATION IN MALAYSIA	Reduce construction time	Less platform preparation
		Easy to install
		Involving off site manufacturing
		Suitable for large scale project
	Enhance construction efficiency	Less workers
		Resources efficient
		Technology involvement
		Should be implemented by all parties
	Decreased complexity in construction phase	Similar contribution of lean construction
		Less flexibility and quite straight forward
		Less wet works at site
		Less stockpile on site (Reduce site congestion)
	Ensure the successes of project delivery	Modernizing construction
		Changing the paradigm of conventional project delivery
		Less defects
		Easy to define timescale

Figure 4 define that majority construction player agreed the usage of IBS can offer the element of time reduction. Therefore, it can ensure the successful of project delivery which is the goal of every construction project.

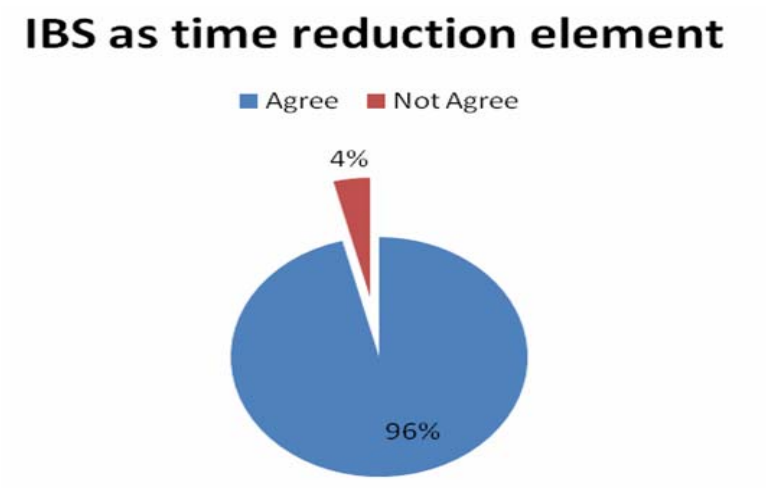

***Figure 4.** Percentage of respondents on a element of time reduction for IBS*

Finally, to validate and proof that IBS offer the time reduction factor, a schedule simulation was done to the project located in the central region of Malaysia. This construction flow of 10 storeys building using conventional construction methods as in figure 5.

According to this construction process flow, the activities for 10 storeys (building work only) were 470 tasks. It is fully using traditional method of construction. By using IBS, some activities can be eliminate. The elimination could be as:
Formwork activities such as preparation, installation and dismantle;
1. Cutting, bending and installation of high tensile steel; and Concreting works

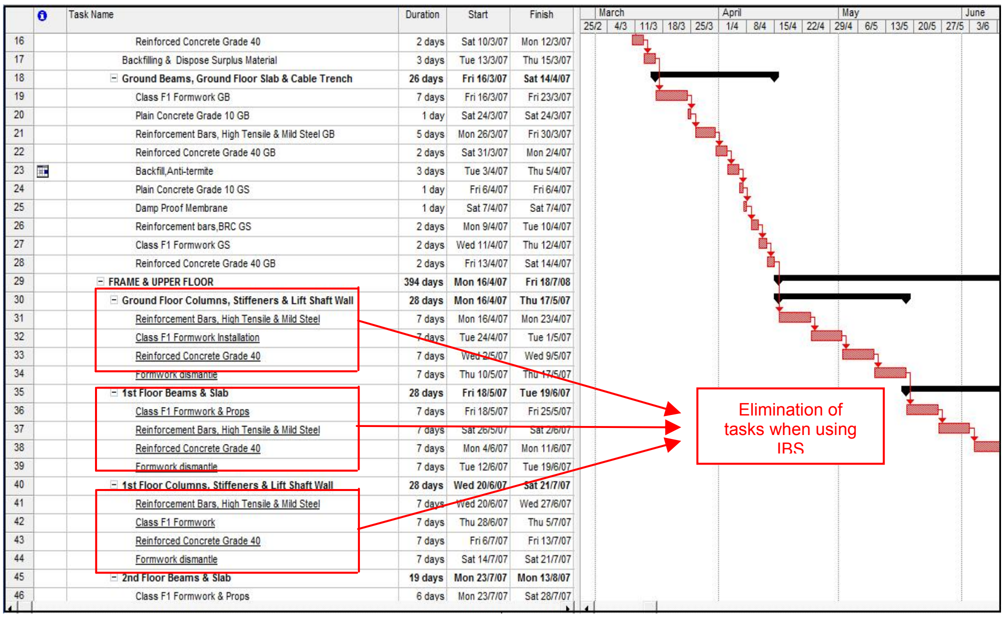

Figure 5. *Gantt Chart for normal conventional construction method in Malaysia*

For this particular project, the elimination count as 128 activities. Mean that, by using IBS, it could reduce 25% of total activities. When replacing the element of IBS in the schedule, the construction process flow should be added the activity of supply chain and installation. The Gantt chart and the process flow becoming like figure 6. This simulation of schedule show that by using the IBS system, duration decrease tremendously. Without considering the period of supply chain like production and transport, the duration decrease as 45% comparing to normal conventional method. But if the supply chain time included, it is probably take around 10% of construction time if the construction manager didn't well manage and forget about just in time concept. But in this implementation of IBS system, it is still reduce the construction time. The situation in construction differs in many ways from the typical situation in manufacturing. Thus, by implementing IBS system, it can reduce the construction time by:

1. Increasing the speed tasks;
2. Reducing the buffer between consecutive tasks; and
3. Reducing the number of tasks.

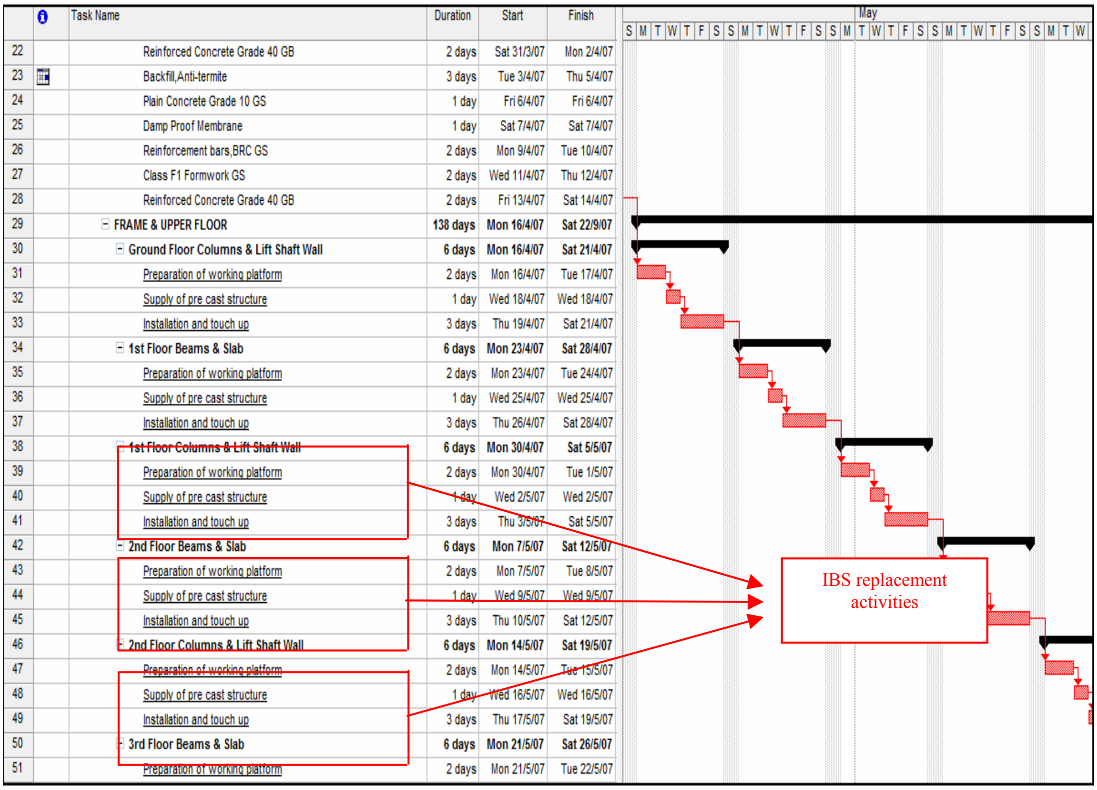

***Figure 6.** Gantt Chart for IBS construction method in Malaysia*

6. CONCLUSIONS

The positive changes include creating a healthy working environment among those involved directly in the construction industry. The major players are architects, engineers, town planner, developer, contractor and the supplier or manufacturer have to play their roles in enhancing their working system, management and administration to enable the modernization in the industry. Although the long-introduced IBS has promised to solve and improved the current construction method and scenario, but the IBS method has been low in gaining popularity, partly due to lack of awareness and coordination among the relevant parties. Currently, the level of IBS usage method is very low as compared to the conventional methods in building construction. In spite of its many benefits, the different perceptions among the construction players and practitioners towards its application in construction industry has led to the low usage of IBS components in the construction industry. Nonetheless, there are still some areas in the IBS management that can be look to conduct a research for further studies and improvement. The finding of this study gives an effective strategy to implement IBS in the current state of the construction industry. The application of the IBS in the construction will eventually speed up the process of nation in achieving Vision 2020 and to become part of the developed nation. Therefore, by applying the IBS concept, it will improve the construction process flow and modernizing construction in Malaysia.

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A LITERATURE REVIEW ON THE STATE AND PRACTICE OF LCC IN MALAYSIA

Mohd Fairullazi Ayob[1] and Khairuddin Abdul Rashid[2]

[1, 2]International Islamic University Malaysia

[1]fairullazi@yahoo.com , [2]khairuddin@iium.edu.my

ABSTRACT: Commentators suggest that most clients focus only on initial or capital costs but very seldom they consider future costs (maintenance, replacement, etc) when they initiate and procure building and infrastructure facilities. In addition, techniques including Life Cycle Cost (LCC) were seldom applied; these despite the fact that past research indicated that the application of such techniques could provide clients with a holistic view on the total costs (initial, maintenance, replacement, etc) of their proposed development. Consequently, a research project aimed at investigating the state and practice of LCC in Malaysia was initiated and carried out. This paper reports on the initial stage of the study and its outcome. The initial study was conducted through a comprehensive review of literature on LCC in general and specifically on the practice of LCC in Malaysia. The outcome of the initial study suggests that among others; most clients and their consultants are aware of the concept of and practice of LCC, there exist several models and techniques on how to apply LCC for building and infrastructure projects and yet LCC is seldom practiced in Malaysia. The findings provide current information on the state and practice of LCC in Malaysia.

Keywords: life cycle cost (LCC), initial and future costs, building, construction and clients

1. INTRODUCTION

In this paper, our aim is to report the state and practice of LCC in the Malaysian construction industry. In preparing this paper, a comprehensive review of literature on the practice of LCC related to Malaysian construction industry were conducted. The outcomes of the review will be reported as an introduction of the research.

The LCC assessment can be divided into three categories which are data inputs, conversion and outputs (BS ISO 15686, 2008; Kelly et al., 2009; RTO-SAS-069, 2009). Nevertheless, it was observed from the literature review the main focus on the LCC practice in Malaysia is considerably on the conversion of LCC analysis. Most of the literatures written on LCC give emphasis to conversion, but less focus is given to the data input and outputs of LCC analysis. Indeed, there is no research has been carried out to identify of how the data inputs are utilized for conversion in producing accurate and reliable outputs of the LCC analysis. Besides, there is no research has been performed to ascertain whether the local data inputs are appropriate to be used in monitoring the outputs of LCC analysis.

2. THE DEFINITION AND CONCEPT OF LIFE CYCLE COST (LCC)

There are many different expressions were used to refer to the ultimate cost or total cost of a building ownership (Schade, 2007; Khairani, 2009; Kirk et al., 1995; Brandon, 1995; Mohammed Kishk et al., 2003a, 2003b; Boussabaine et al., 2006; Kirkham, 2007; Levander et al., 2009; Zainal Abidin et al., 2010). The difference in wordings and expressions to symbolize total cost of building ownership have caused ambiguity and confusion to the practitioners to select an appropriate technique to compute total cost of building ownership in the industry. Although the expressions used are interchangeably, the Life Cycle Cost (LCC) is preferred as it is widely used in the practice and better known to represent the total cost related to the building ownership throughout the study life.

There are many different definitions of Life Cycle Cost (LCC) found in the literature. The most basic definition is the total discounted cost of building ownership throughout the study life. It represents an economic assessment of an asset, expressed in terms of total costs related to the building ownership which connects the initial capital cost with operating and maintenance costs, replacement costs, financing costs, and the cost or benefit of the building at the end of the study life.

The LCC studies are performed to provide major cost factors and potential cost saving. The outputs of LCC studies are used to advise the clients or building owners in determining the most optimum cost of asset ownership, and comparing cost-effectiveness of mutually exclusive alternatives (Mohammed Kishk et al., 2003a, 2003b; RTO-SAS-069, 2009). The LCC studies are not only a one-off task but they can be applied as an on-going activity to constantly assess cost implication at any stage of procurement activity in the effort towards achieving value for money (Mohammed Kishk et al., 2003a, 2003b; Bakis et al., 2003). Nevertheless, it is important the data inputs procured for producing a LCC analysis are reliable, comprehensiveness, updated, accessible and available in order to ensure the estimated results of LCC analysis are appropriate and accurate with minimum erratum.

3. REVIEW

3.1 The application of LCC in the Public Private Partnership (PPP) program

Public Private Partnership (PPP) is a new procurement strategy in Malaysia that refers to a working relationship between a Government agency and a private agency in meeting a common goal to deliver public infrastructures and services (Khairuddin, 2010a, 2009; Ahmad Zamri, 2010). PPP focuses on efficient allocation of risk, life cycle cost, service approach, private sector innovation and management skills on the basis of long term partnership between public and private sector to attain value for money (Khairuddin, 2010a, 2009; Nor Hizam, 2010).

The LCC application was adopted by the Government into PPP procurement approach as a technical instrument and cost effectiveness tool to drive the project activities in meeting the Government objectives (3PU, 2009a:p6; 3PU, 2009b:p5; EPU 2010a, 2010b). The Public Private Partnership Unit of Prime Minister's Department (3PU) was formed by the Government to undertake effective planning and management in the implementation of PPP project programs in Malaysia (3PU, 2010; Khairuddin 2009:p80, 2010a; 3PU, 2010; Ahmad Zamri, 2009; Nik Nasir, 2009; 3PU, 2009a: pg.6; 3PU, 2009b: pg.5; Mariyam, 2010).

The market capitalisation for PPP (comprises of PPP/PFI/Privatisation projects) were extensively large, approximately RM182.45 billion achieved from 1983 till May 2010 (3PU, 2010a, 2010b). Unfortunately the concept and methodology of PPP approach was unknown to many. The concept and methodology of PPP is ambiguous and limited evidence is existed to ascertain how LCC is framed and used as a cost effectiveness initiative to produce reliable and accurate outputs in the PPP projects. The justifications were proven from survey conducted by Khairuddin (2008 cited in Khairuddin, 2010b) in identifying the key PPP competencies, and assessing the current level of PPP competencies on the group of construction key players with diversity of backgrounds (which include architects, engineer, quantity surveyors, contractors, financial management, financial and banking services). The survey reveals that 36 percent of the respondents were ambiguous and unclear with the PPP's concept and methodology. Lack of consistency on the level of PPP competencies due to a comparatively 'big' gap between the key areas of PPP competencies required and the availability of Malaysia experts in possessing the required competencies are some of the reasons found to the problem (Khairuddin, 2008 cited in Khairuddin, 2010b).

Based on the observation from the survey, it can be encapsulated the PPP's concept is still new in Malaysia and the adoption of LCC in the PPP context has been relatively new and limited. These setbacks should be treated effectively by enhancing more knowledge and competencies required through seminars, trainings and publications. These approaches can therefore accelerate resources, expertise, and professionals in fostering effective cooperation and working collaboration between the public and private sectors in successfully attaining the goals and objectives of the PPP programme.

3.2 The application of LCC in the Value Management (VM)

Value Management (VM) was first introduced in Malaysia in 1986 by Roy Barton through the Quantity Surveying Department at Universiti Teknologi Malaysia (Mohd. Mazlan, 2004, 2006, 2010; Mohd Azmi, 2004; Aini et al., 2006, Aini et al., 2009; Aini and Torrance, 2005 cited in Aini et al., 2008;; Olanrewaju et al., 2010).). Through the efforts carried out by Mohd Mazlan, VM was adopted as a mandatory requirement for the Government project (costing RM50 million and above), Malaysia Airport Holding

Berhad (for project costing above RM30 thousand), and Tenaga Nasional Berhad (for project costing above RM10 million). VM was also chosen as a cost prediction technique for several other local agencies like Public Work Department (PWD), Construction Industry Development Board (CIDB), Petronas, Ministry of Health (MOH) and etc. (Mohd. Mazlan, 2010). LCC is used to facilitate the agencies to search for eliminating unnecessary costs, increase cost saving in the building and infrastructure projects, optimise the overall life cycle cost, and simultaneously get the qualities standard required (Mohd. Mazlan, 2004; Mohd Azmi, 2004).

There are several alternatives of techniques can be chosen in producing a VM analysis (Nazirah et al., 2006). The LCC technique is a common used technique in the VM study that evaluates alternatives in selecting the most optimum solution for the purposes of removing unnecessary costs while increasing the cost savings, quality, reliability, performance, and other criteria to meet the client expectations (Dell'Isola, 1997 cited in Olanrewaju, 2010; Kirk et al., 1995; Mohd. Mazlan, 2010). VM takes account all the criteria requirements within the client value system. In the VM process, LCC is employed to select an alternative that has the lowest cost; whilst the other performance criteria like aesthetic (inspiring and harmonious), images (reputable and progressive), fitness for purpose, sustainability, buildability, maintainability, technology, quality, safety, reliability are evaluated through VM process (Olanrewaju, 2010).

In spite of the mandatory requirement by many agencies to exercise VM in the projects, there is unclear and ambiguous whether LCC was employed in the context of VM practice. It was observed that there is no standard approach or proper guidance of how LCC is employed in computing project cost for VM study in achieving a better value for money. It was contended by Aini et al. (2006), Aini and Torrance (2005) whether VM should be considered as a “cost cutting” tool as it could perhaps limit creativity and innovative thinking of designer to express state of art in a specific project. Nevertheless, several authors assert VM is not about a cutting cost as its main function is to enhance and improve value and function of the building designed in effective and efficient manner. As LCC is used to predict total cost of a building until the end of the service life, whilst VM acts for cost saving of the project up to the project completion stage, it is considerably essential to incorporate the functions of LCC and VM in eliminating unnecessary costs in the line with client objective to optimize the overall life cycle cost of the building, whilst at the same time to obtain the optimum function and value (Kelly et al., 1993; Nazzatul Shiha, 2009; Olanrewaju, 2010).

Based on the observation from the literature review, there is no evidence to indicate the practice of LCC in the context of VM in the Malaysian construction industry. It was found the main focus of VM in local practice is to attain optimum value in asset by undertaking necessary functions or alternatives at the lowest cost rather than emphasize on the importance aim to optimise the total cost of the asset throughout its study life. Without the adoption of LCC technique as a systematic framework in the VM exercise, it is uncertain to ascertain whether the outputs produced from VM analysis are undoubtedly reliable and accurate for sustainable development of an asset throughout a study life in attaining value for money.

3.2.1 LCC software for Value Management in Malaysia

A local web based environment software, LCCsoftonline (Version 4.0) was developed by Mohd Mazlan supported by MCM Value Sdn. Bhd. LCCsoftonline is developed to improve the practical use of LCC in the VM application, and facilitates the practitioners in making comparison of building cost in a meaningful and simplified approach. It is observed the software can only be used to convert the cost inputs to produce LCC outputs for the use in VM process. However, it does not provide method of how to acquire reliable and updated cost inputs for conversion. Moreover, it was observed that the literature information and data on LCCsoftonline is inadequate and limited pertaining to its use, principle and methodology in the VM studies. It was explained by the founder, Mohd Mazlan that the software is relatively new, and moreover it has not been launched yet by the Construction Industry Development Board (CIDB) as an online tool for the use of practitioners in exercising VM in the construction industry (Source: Mohd Mazlan; MCM Value, 2010; http://value.mcm-associates.com; http://lcc-soft.com/Info_aboutus.php). Based on the observation, hence it can be recapitulated the application of LCC in the VM exercise has not been well established and the use of LCC to provide information for VM application was not widely used by the practitioners in the construction industry.

3.3 The application of LCC in Facility Management (FM)

The Government has put high commitments by expanding huge amount of money into building management activities to ensure the potential use and performance of the buildings can be maximized throughout the entire service life span. Approximately RM1.08 billion in the Ninth Malaysia Plan budget (2005-2010) was expended for building management activities including maintenance works. The fundamental aims of the Government in preserving and maintaining the buildings are to enhance durability of the building performance, to deliver better return of investment and to increase cost saving through reduced life cycle costs (Puva et al., 2010; MBAM, 2007, Olanrewaju, 2008; NAFAM 2007; Utusan Malaysia, October 10, 2009; Azlan-Shah et al., 2010). The amount of money required for maintenance works increases in continuously regardless of size, type, location and ownership. The Government had attempted to reduce maintenance costs in the Budget 2008 by undertaking programs that were efficient and cost effectiveness in maintaining the building through reduced life cycle costs to meet the acceptable required performance (Azlan Shah et al., 2010; Mohd Sidek, 2007 cited in Olanrewaju, 2008; Puva et al., 2010; Ng, 2009).

The Government has organized a national convention, the National Asset and Facility Management (NAFAM) which aimed to set objectives and strategies in implementing the best practices in the field of facility management. The convention provides a platform where the Government and industry players are collaborated to formulate sustainable integration of asset management practices incorporated with life cycle cost (LCC), monetization, performance monitoring, good governance, asset management policy, public-private partnerships (PPP), and private finance initiative (PFI) (Puva et al., 2010; NAFAM 2007; Ng, 2009). The convention addresses several issues including LCC and proposes strategies to be formulated and executed to embark LCC as an integrated facility management approach technique in managing buildings and facilities in the Tenth Malaysia Plan (Shaziman, 2009; Ng, 2009).

Based on the observation from the literature review, it can be encapsulated the application of LCC in the context of facility management in the country has been relatively new. In the Tenth Malaysia Plan (2011-2015), the Government encourages LCC technique to embark as a part of development culture in maintaining and preserving the asset in efficient and holistic manner. The fundamental reason is the buildings in Malaysia that require maintenance are many and everlasting (Olanrewaju, 2008; Puva et al., 2010; NAFAM 2007; Utusan Malaysia Online, 2009; Shaziman, 2009; Ng, 2009; Judin, 2009;). Unfortunately, no research has been carried out to identify appropriate methodology for monitoring life cycle cost of the building, in order to prevent from being constructed for cheap and costly to maintain over the life span. A comprehensive assessment in the life cycle cost of the assets should be executed as part of facility management approaches to ensure the buildings and facilities can be operated with optimum maintenance and replacement costs, which can therefore facilitate the Government to attain the best value for money.

3.3.1 LCC software for Facility Management in Malaysia

A comprehensive engineering software application, the Life Cycle Cost for Multi-Storey Housing (LICCOMS) is developed to estimate life cycle cost of multi-storey low cost housing throughout the life span by performing the calculation through the Life Cycle Cost Analysis module incorporated with other features that accommodates by LICCOMS. LICCOMS is designed as a multifunction tool composes of several modules which include Life Cycle Cost Analysis, Module Builder, Service Life Prediction and Sustainable Rating (Siti Hamisah et al., 2005; Masoud, 2009, Masoud et al., 2010; MBAM, 2007; Business Times October 3, 2001; Berita Harian October 3, 2001). LICCOMS takes into account economic effects, the type of project, design factor, inflation rate, the service life of building and its components, operation costs, maintenance costs, occupancy costs, furnishings, energy costs, site cost, professional fees and capital cost, insurance, cleaning costs, replacement costs. (Zin et al., 2002 cited in Masoud, 2010; MBAM, 2007; Berita Harian October 3, 2001). LICCOMS facilitates users to compute the construction costs and operating costs of the building incorporated with two or more dimension of drawings (2d or 3D). LICCOMS converts the data which are procured from the environmental loads or performance rating, that produce from the past building assessments of more than 400 buildings and 7400 building components around Malaysia (Siti Hamisah et al., 2005; Business Times, 2001).

Despite of its multifunction, there are several setbacks. It was identified by Masoud et al. (2010) that LICCOMS may not provide accurate and reliable outputs for decision making as it is heavily depended on out of date data that were collected throughout 6 years research by Sustainable Construction Research Group (SCRG) of University Teknologi Malaysia. Real time data collections are important inputs of LICCOMS to ensure reliable and accurate outputs could be produced on the predicted cost. Unfortunately, the limitation of LICCOMS as a standalone software does not allow the system to be frequently updated with a real time data (Masoud et al., 2010; Olanrewaju, 2008; Berita Harian October 3, 2001). The availability of its use in the building industry is limited as the price of LICCOMS is considerably expensive to be purchased by the private building owners and individual users. To rectify these problems, Masoud (2009, et al., 2010) has proposed an efficient solution by shifting the LCC standalone paradigm to a web based environment that can allow the use of real-time databases. Hence, it could perhaps facilitate the users to easily update the data anytime and anywhere from appropriate resources. Based on the observation, it can be recapitulated the development of a web-based LCC tool will lead practitioners, users and analysts in Malaysia to practice LCC through an online calculation for making economic decision, and also facilitate them to procure data of reliable, accessible, updated and available from suitable resources within Malaysia. This obviously shows the availability of reliable data as input to a web-based LCC tool is essential in order to ensure an accurate LCC analysis can be produced.

3.4 The application of LCC in sustainable building

A “sustainable building” is defined as a building that is “designed, constructed and effectively managed by the users where the service life of the building they inhabit exceeding the design life, preserves the environment, meets ecological performance requirement, and notwithstanding surrounding environment and development the building able to meet the capabilities and needs of future generation” (Siti Hamisah et al., 2007). The benefits of a sustainable building are it can increases energy saving, encourages the use of recycled materials, and minimises the emission of toxic substances throughout the building life cycle. The Government through the prime minister had announced in the United Nation Climate Change Conference (COP 15) 2009 to execute effective strategies in reducing the emissions of greenhouses (carbon dioxide, CO^2) up to 40 percent by 2020 as compared with 2005 levels (The Star Online July 31, 2010). The LCC has been a major agenda of the Government as it will emphasize on sustainable development strategies in every aspect of a project development, energy efficiency in existing government buildings, managing solid waste production and renewable energy, in order to reduce 50 million tons of carbon dioxide per year (Judin, 2010). Public Works Department (PWD) was given important roles in monitoring and reducing carbon emission in the area of energy efficiency by fostering and formulating sustainable development strategies in the construction of Government buildings and facilities (Kannan, 2006).

In spite of the advancement progresses of the sustainable building and its essential to balance total economic cost, social life and ecological performance in Malaysia, there is no standard method has been formulated to quantify life cycle cost of a sustainable building (Shaziman, 2009). A consistent approach or standard procedure should be formulated and executed to break down the cost components and other cost parameters of the sustainable building to the life cycle cost estimation. This to ensure the procedures and methodology of the LCC analysis applied in the sustainable building development strategies will be well understood and comprehended by the practitioners, which can therefore be executed in producing reliable and accurate outputs (Kelly et al., 2009). Other major setbacks of LCC pertaining to sustainable building design are limited data inputs, unavailability of real time data, inadequate knowledge to predict total life cycle cost of a sustainable building, and etc. (Aun, 2010).

Although Malaysia Standard, MS 1525 has been produced to represent the minimum standards for energy efficiency in the design of new buildings and retrofitting of existing buildings, but it is observed that the standard does not demonstrate LCC as an economic decision making tool for energy efficiency and economic cost of the buildings. The standard mainly focuses on non-monetary factors, especially the design, operation and maintenance of the building envelop, lighting, electric power, mechanical ventilation, etc. (Kannan, 2006). As compared to other countries like Hong Kong, it owns a standard procedure or methodology that give emphasis to the employment of LCC technique in determining long term benefits of sustainable building designs and the items (Leong, 2009).

4. CONCLUSIONS

This paper explored the state and application of LCC in Malaysia, exclusively the adoption of LCC as a tactical tool for strategic advancement issues in the Malaysian construction industry. The application of LCC in the strategic advancement issues which include the Public Private Partnership (PPP), Value

Management (VM), facilities management (FM), and sustainable building were reviewed. The LCC technique has been considered as a useful tool to provide crucial information in advising the Government and clients to make the most cost-effectiveness decision on the investment and to attain the best value for money. Nevertheless, it was observed that most literatures put emphasis to conversion rather than the data inputs and outputs of LCC analysis. Therefore, it can be encapsulated the use of LCC in Malaysia is still at its infancy stage. To embark LCC as an effective tool in facilitating the Government and clients to attain the best value of money over the investment, the setbacks that hinder the application of LCC in the strategic advancement issues should be rectified effectively. The review is limited to the reported LCC applications only in the Malaysian construction industry. The research is on-going and further research is encouraged on the current level of data input requirements of availability, reliability, updated and accessibility in the effort towards producing an accurate LCC analysis.

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SUSTAINABLE HOUSING USING AN INNOVATIVE MORTARLESS INTERLOCKING BLOCKWORK SYSTEM – THE EFFECT OF PALM OIL FLY ASH (POFA) AS AN AGGREGATE REPLACEMENT

Nasly M.A[1], Abu Azam Md Yassin[2], Norhaiza Nordin[3],
Khairunisa Abdullah[4], Noram Irwan Ramli[5]

[1, 3, 4, 5]Universiti Malaysia Pahang, Kuantan, Malaysia

[2]CCA Systems Sdn Bhd, Johor, Malaysia

[1]nasly.ccas@gmail.com, [2]abuazam@gmail.com, [3]norhaiza@ump.edu.my

ABSTRACT: With the increase in material costs in the construction industry, there is a need to find more cost saving alternatives material so as to maintain the cost of constructing houses at prices affordable to clients. Universiti Malaysia Pahang in collaboration with industry partner, CCA Systems Sdn Bhd, has developed a building system which incorporates the interlocking load bearing block as its main feature, which consists of three different block types for the frame and wall system. Sustainability can be achieved by using Palm Oil Fly Ash (POFA) and recycled concrete and aggregate related construction waste as aggregates replacement in the interlocking load bearing block.This paper describes POFA as a replacement material in the production of the interlocking load bearing block. A number of mix proportions were tested and the compressive strength of the cube blocks on the 7th day and 28th day showed that POFA at the mix ratio of 1:6:2 and 1:5:3 (cement: sand: POFA) give optimum value of the compressive strength as replacing aggregates that satisfy the required strength to be used for construction of buildings. Using the formulated mixture, interlocking load bearing block is made using the CCA press, the resultant block is tested and the compressive strength is within the required specification for house construction. The design constraints and construction of a cafeteria building using the load bearing interlocking block is also presented.

Keywords: POFA, sustainability, aggregates replacement, load bearing interlocking block

1. INTRODUCTION

The most essential item to be considered in construction to realize the building structure is the materials for construction, together with other factors such as budget, site safety, tender documents preparation and so on. Construction materials including bricks, timber, cement, and steel are increasing in demand due to the rapid expansion of construction activities for housing, and other buildings. The current unrest in the middle eastern countries is causing rise in oil and petrol which will add to the world economic and financial upheaval resulting in the rising cost of construction materials. Hence there is the need to search alternative materials in order to fulfill the construction demand while maintaining cost at a reasonable level.

Concrete being one of the important materials in construction is produced from the mixture of cement and aggregates. Much innovation is being produced globally to face the growing demands of housing construction which involves new construction techniques and utilisation of waste materials for aggregate and cement replacement. POFA, a waste by product of the palm oil mill, could also be an alternative of aggregate replacement. The POFA usage in bricks material as aggregates replacement will reduce the cost of producing bricks. Achieving strength and durability are the targets in any concrete mix design. This paper will present the results of some experimental studies on obtaining the optimum mix ratio of compressed concrete using POFA as aggregate replacement with respect to the compressive strength for 7 and 28 days.

2. THE INTERLOCKING BLOCK TECHNIQUE

The construction method of using conventional bricks has been revolutionised by the development and usage of interlocking concrete masonry units (CMU) and lightweight concrete blocks. The tedious and time-consuming traditional brick-laying tasks are greatly simplified by the usage of these effective alternative solutions.

The interlocking blocks are different from conventional bricks since they do not require mortar for bricklaying work. Because of this characteristic, the process of building walls is faster and requires less skilled labour as the blocks are laid dry and lock into place.

Figure 1. *Aligning blocks*

Concrete blocks may be produced with hollow centres to reduce weight, avoid seepages or improve insulation. The holes inside the concrete block allow rebar and concreting (creating reinforced concrete) to run vertically through the block to compensate for the lack of tensile strength. Mild steel rebar can be used instead of the usual higher grade steel. Once a section of wall is built, grout holes are filled with a lean cement mixture to seal the wall and solid. The amount of grout used was calculated to be less than 7.5% of the mortar used in conventional masonry.

2.1 Production of Interlocking Blocks

Interlocking blocks were produced in special moulds, in which compaction is done mechanically, depending on the type of block, material used, required quality and available resources. The blocks can be made directly at the building site, or on a larger scale in a production yard.

Concrete blocks are the common manufactured interlocking load bearing blocks that require high compression strength. After opening the lid and ejecting the block, it was removed and flipped before stacking in a shaded place for curing and hardening. Moist conditions is maintained to ensure uniform curing and hardening. After demoulding, the blocks are carried away on pallets for curing.

A typical compressive strength of concrete blocks is 7N/mm^2.

Figure 2. *Hydraulic powered block press*

3. PALM OIL FUEL ASH (POFA) AS AGGREGATE REPLACEMENT

Malaysia is the world's leading producer and exporter of palm oil products with a reputation for quality and consistency. To date, there are more than two hundred palm oil mill plants operating in the country that are self sufficient industry as far as energy utilisation is concerned. Palm Oil Fuel Ash (POFA) is a waste material obtained by burning of palm oil husk and shell as fuel in palm oil mill boilers. For now, thousands of tons of ash are produced annually and are simply disposed without any profitable and commercial return. Like other fly ashes, palm oil fuel ash is grayish black in colour, becoming darker with increasing proportions of unburnt carbon. The palm oil ash that had been considered worthless can actually be used in construction industry specifically in concrete technology.

Both physical properties and chemical analysis indicated that POFA is a pozzolanic material. This pozzolanic material is grouped in between Class C and Class F as specified in ASTMC618-92a (Awal & Hussin, 1996). Meanwhile, POFA is much influenced by the operating system in palm oil factory. It also has been found that POFA could produce a more durable concrete.

Since our country is continuously increasing the production of palm oil, therefore more ashes will be produced and failure to find any solution will result in potential disposal problem. Thus, by utilizing this type of waste material we can cut down the number of landfill areas required for disposing the ash hence averting potential future environmental problem.

3.1 Experimental Methodology

Compressed concrete cubes from eight different mix ratios were produced by using a special steel mould to produce cubes of sized 100mm x 100mm x 100mm. The mixture are filled up into the mould then compacted to the size required by a hydraulic machine. The cube samples produced then went through the curing processes which are either air dried curing method or covered with wet gunny sacks for 7 days and 28 days. The strength of the compressed concrete cubes on 7 days and 28 days curing age were determined by the normal cube compression till destructive test. The density of the cubes was also recorded.

Figure 3. *Compaction by hydraulic power*

Figure 4*. Curing of POFA concrete cubes*

3.2 POFA Concrete Cube Strength

3.2.1 Compressive strength on mix ratios

Materials prepared using mix ratios of cement:sand:POFA range from 1:0:8 to 1:6:2. From the results of compressive strength of the cubes obtained, the graphs were plotted as shown below. The percentage of POFA used (1:6:2) is 22.2 % which gives the highest value of compressive strength which was 5.43 N/mm2 for 7days and 6.96 N/mm2 for 28 days. Then followed by 33.3 % of POFA (1:5:3) used in mixture. These area the only cubes that attained the 5.4 N/mm^2 minimum value for load bearing blocks.

Table 1. *Test Results Compressive Strength from each mix ratio*

Cement/sand/POFA ratio	**POFA (%)**	**Average Compressive Strength (N/mm^2)**	
		7-day	7-day
1:0:8	88.9	1.23	1.77
1:2:6	66.7	1.47	1.60
1:3:5	55.6	2.32	3.18
1:4:4	50.0	2.73	3.66
1:5:3	33.3	4.99	6.42
1:6:2	22.2	5.43	6.96

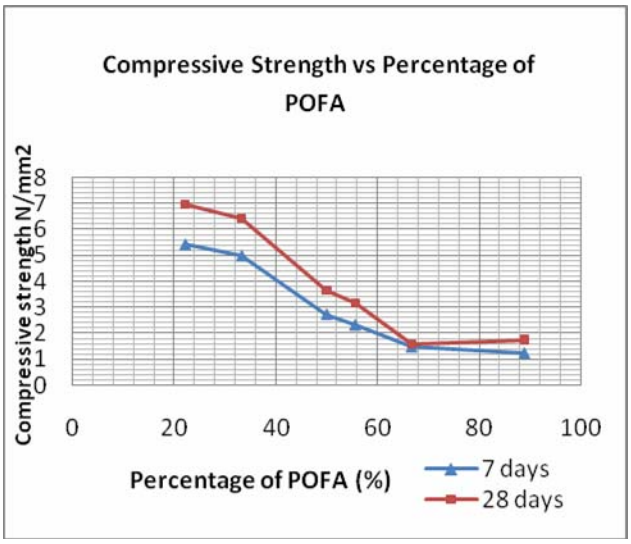

Figure 5. *Compressive strength by percentage of POFA for 1 part cement to 8 parts aggregate*

3.2.2 Compressive strength by different curing methods

A series of tests were carried out to examine the strength of the POFA concrete cubes against the curing method or exposure after production. These cubes of different mix ratios were cured by either air dried curing or moist gunny sack curing. The results of compressive strength from both curing methods were then compared as shown in Figure 6.

Table 2. *Compressive strength due to differenct curing methods.*

Cement/Sand/POFA Ratio	POFA (%)	Average Compressive Strength (N/mm^2)	
		Air dried	Covered curing
1:00:08	88.9	1.27	1.57
1:02:06	66.7	1.53	1.67
1:03:05	55.6	3.1	3.26
1:04:04	50	3.38	3.93
1:05:03	33.3	5.6496	6.42
1:06:02	22.2	5.916	6.96

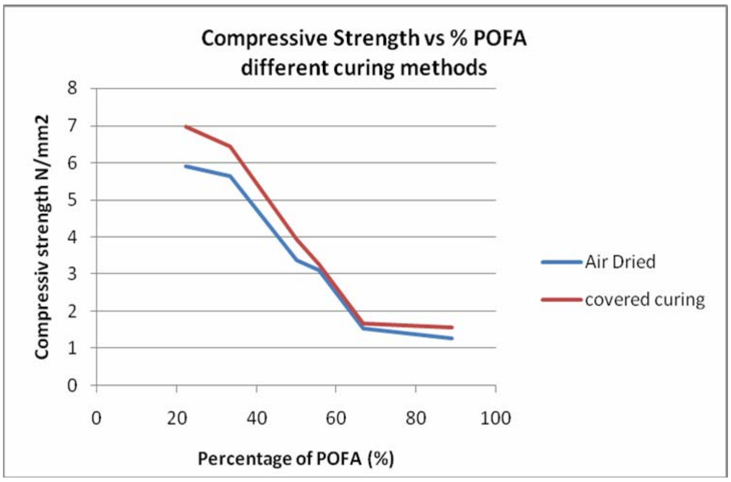

Figure 6. *Compressive Strength on 28 days cube age by different curing methods applied on each mix ratio.*

From Figure 6, we can see that the compressive strength of the POFA concrete cubes obtained by covered curing method have higher values than those that underwent air dried curing.

3.2.3 Compressive strength between POFA and Quarry Dust concrete cubes

The normal compressed concrete blocks produced in the industry is using cement:sand or cement:sand:quary dust. Two mix ratios that are being compared in this section are ratio of 1:5:3 and ratio of 1:6:2, since these ratio mixes gave cube compressive strength exceeded the minimum required of 5.4N/mm2.

Table 3. *Compressive strength of POFA and Quarry Dust concrete cubes*

Ratio (cement:sand:POFA/QD)	Compressive Strength (N/mm^2)			
Cube Age (days)	7		28	
Materials	POFA	QD	POFA	QD
1:5:3	4.99	8.00	6.42	8.91
1:6:2	5.43	8.29	6.96	10.23

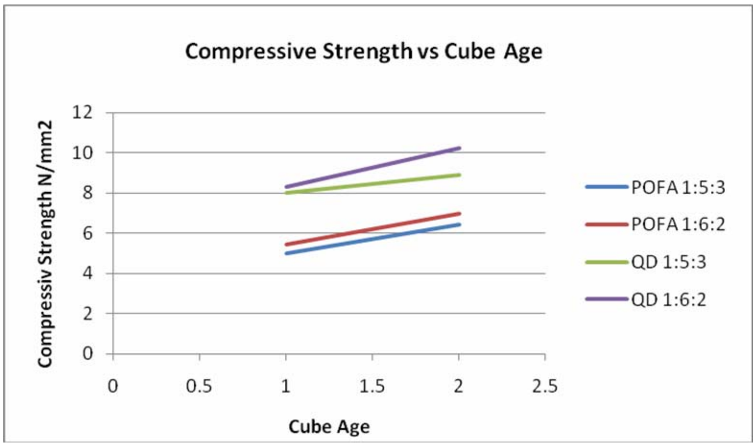

Figure 7. *Compressive Strength of Concrete Cubes using POFA or Quarry Dust as aggregate*

From the data obtained, as shown in Figure 7, the compressed concrete using Quarry Dust has higher compressive strength than the concrete cubes using POFA. The compressive strength of QD concrete is greater than 7 N/mm^2, the normal value used by most concrete block producers, it can be used to build as load-bearing and non load-bearing structures. As for the compressed concrete using POFA, it is recommended for use in external works as non load-bearing structures since the compressive strengths exceed the required 5.4 N/mm^2 standard.

4. DESIGN AND CONSTRUCTION OF BUILDING

4.1 Construction Concepts Using Load Bearing Blocks

The cavity holes of the interlocking blocks permit the introduction of vertical reinforcement embedded in concrete without the need for any formwork thus eliminating the use of wood in form work. Reinforcement can be introduced to make the building withstand earthquakes and heavy wind loads. Because of the size and resistance of the blocks, load bearing walls can be constructed.

Before placing the first course in a mortar bed, the blocks must be laid dry on the foundation around the entire building, in order to ensure that they fit exactly next to each other (leaving no gaps), and that an exact number of full blocks are used, otherwise the system will not function. When laying the first course in the mortar bed, care must be taken that the blocks are perfectly horizontal, and in a straight line, and at right angles at corners.

Once the base course is properly hardened, the blocks are stacked dry, with the help of a wooden or rubber hammer to tap the blocks gently into place. Up to 10 layers can be placed at a time, before the grout holes are filled with a liquid mortar - 1 part cement to 3 parts sand to 1 part water.

Interlocking blocks are ideally suited for load-bearing wall constructions, even for two or more storey buildings, provided that the height of the wall does not exceed 20 times its thickness, and wall sections without buttresses or cross walls do not exceed 4.5 m length (to prevent buckling).

4.2 Building design using load bearing blocks

Almost any type of building can be constructed with interlocking blocks, the main design constraints being that the plan should be rectangular and all wall dimensions and openings must be multiples of the width of the block type used. All other principles of design and construction, such as dimensioning of foundations, protection against rain and ground moisture, construction of ceilings and roofs, and the like, are the same as for other standard building types.

It is advisable to place channel blocks around the building, at window sill height, to install a ring beam. They should also be placed directly above doors and windows to install lintels, and directly below the roof to finish the walls with a ring beam. To increase structural stability, especially in earthquake regions, steel rods or treated kenaf should be inserted in the vertical grout holes, especially at corners, wall junctions and on either sides of openings.

4.3 MTDC Cafeteria building using load bearing interlocking block system

Initially the foundation, ground beam and ground slab is laid with the appropriate vertical steel implanted. It takes normally a week for the foundation to set, then the first layer of interlocking load bearing blocks is arranged to ensure that the wall is straight and right angled at the corners as shown in Figure 8a.

After that the crew of four workers can independently stack up the blocks to the required height. Horizontal bars are placed for the construction of ring beam around the house for added stiffness as shown in Figure 8b and 9a.

(a) First layer of blocks *(b) Below window sill layer*

Figure 8. *Typical construction site of an interlocking load bearing concrete block building*

(a) Below window level with water piping *(b) roof ring beams*

Figure 9. *Typical construction of ring beams*

Figure 10*. Final touches and the completed building*

5. CONCLUSIONS

The conclusions that can be drawn in this study are as follows:

1. The mix ratio of 1:6:2 and 1:5:3 (cement: sand: POFA) achieved the required compressive strength standard of 5.4kN/mm^2 for external walls construction.
2. The cubes need to be cured using moist covering to obtain higher compressive strength as compared to cubes that had free air exposure even under shades.
3. From the observation, the cubes that have high % POFA e.g. 1:2:6 (cement: sand: POFA), the cubes are more brittle especially on the surface of the cubes.
4. The interlocking load bearing block using POFA as a partial replacement to quarry dust satisfy the required specification for the construction of a single storey building.

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CIVIL ENGINEERING AND ARCHITECTURAL BUILDING FEATURES DISPARITY AND PRESERVATION OF STRUCTURAL AND FABRICS INTEGRITY IN HERITAGE BUILDING: A REVIEW

Siti Nor Fatimah Zuraidi[1], Zainal Abidin Akasah[2] and Mohammad Ashraf Abdul Rahman[3]

[1,2,3] Universiti Tun Hussein Onn Malaysia, Johor, Malaysia

[1]timi-gee@yahoo.com , [2]zainal59@uthm.edu.my , [3]ashrafr@uthm.edu.my

ABSTRACT: This paper will be presents a review of the literature pertinent to the life-cycle preservation, operation, and maintenance aspects of buildings features disparity are rarely considered at the initial design and construction stages. This is due to the facts that mainly in the disconnections within built facilities life-cycle, from their initial creation processes, through use to obsolescence. There is also a dearth of knowledge of the specific impact of upstream design and construction decisions and choices on the downstream preservation, operation, and maintenance of building structures and fabrics. Hence, in order to bridge this knowledge gap and forge a connection between design and life-cycle use and maintenance of buildings, this study aims to identify design defect which affect building maintenance and to identify the systematic framework for selecting a suitable maintenance strategy for each individual item in a building. The investigation for this study will be undertaken in two phases. The first phase involved a literature search on the identification of some of the relative defect factors in design stage that affect maintenance. The second phase involved gathering data through site visits, interviews and discussions with owners, architectural/engineering firms and contractors. The scope of this research will be limited to civil and architectural defects in building design that affect maintenance only in the Museum Adat Istiadat Melayu Kelantan (Muzium Islam). It will be observed that the different of structural and fabrics integrity of these buildings to their different defect styles and features. This finding will underscores the importance of designing architectural features of buildings with their operation, maintainability, and life-cycle preservation in mind, if structural and fabrics integrity must be maintained, and a greater sustainability achieved, while restoration ringgit are saved.

Keywords: construction, architectural, building, maintenance, life-cycle, preservation, defects

1. INTRODUCTION

Malaysia is a fast developing country and new building are being erected every to accommodate the needs of public and private sectors. Many building construction projects have been built due to the demand of both the public and private sectors. In order to meet the high demand of both the public and private sectors in new building constructions in a short space of time, it is anticipated that many defects will arise especially during design and construction phase. This will inevitably resulting in high maintenance costs. Maintenance costs of a building during their functional lifetime could easily surpass the initial outlay of a new building. Therefore it's important to consider maintenance aspects at the very outset of the life cycle of a building because decisions made at planning, design and construction stages have a large

effect on the maintenance costs and works incur later in the life cycle of a building. Aged building doesn't necessary have to be associated with high maintenance cost as a new building with numerous defects from faulty design and construction could easily equal or surpass the maintenance cost of an aged building (Ahmad Ramly et al, 2007).

Under normal condition all building begins to deteriorate the moment after they are constructed, and of course maintenance is needed. One research conducted in United Kingdom revealed about 20% of the average annual expenditure on repairs in building arises from defects (Sadi Assaf et al, 1996). As consequences if the number of defects could be reduce it would reduce the maintenance expenditures.

It is obvious that the need of maintenance is very important and needed for all sectors including properties sectors. Regard to the highly cost of maintenance it is important to study maintenance problem so that an effective maintenance can be carried out.

Maintenance is defined as the required processes and services undertaken to preserve, protect, enhance and care for the university buildings' fabrics and services after completion, in accordance with the prevailing standards to enable the building and services to serve their intended functions throughout their entire life span without drastically upsetting their basic features and uses (Lateef, 2010).

Building maintenance is defined as "work undertaken in order to keep, restore or improve every part of a building, its services and surrounds, to a currently accepted standard, and to sustain the utility and value of the building". The objectives of building maintenance are therefore (Alner, 1990):

a. To ensure that the buildings and their associated services are in a safe condition;
b. To ensure that the buildings are fit for use;
c. To ensure that the condition of the building meets all statutory requirements;
d. To carry out the maintenance work necessary to maintain the value of the physical assets of the building stock; and
e. To carry out the work necessary to maintain the quality of the building.

1.1 Problem Statement

The Malaysia is a rich developing country with huge capital resources. It is developing very fast in every area including building construction. Public and private sectors have initiated the need for large and complex construction projects. Meeting the high demand of both parties in a short time, it is expected that many errors and defects have occurred during the design and construction stages which will result later in high maintenance costs. Building maintenance is a major activity in most countries.

The researcher believes that the neglect of building maintenance in Malaysia is serious that intervention at national level must be done. Since maintenance problems in building facilities are heavily attributed to many factors, whereby the most important ones are design limitations and construction knowledge, owners must be made aware that insufficient funding of design and construction will impact future maintenance capabilities. Owners will continually be challenged to meet greater demand for improved maintenance standard with less staff, less capital and in less time than ever before if nothing is to be done (Chong, 2004).

Problems in buildings can be broadly categorized as defects or deterioration. Defects arise due to errors or omission or negligence by the designers or contractors. Deterioration on the other hand, is the natural process which is unavoidable, although maybe minimized by exercising care in design, material selection and proper construction method. A research by Gibson (1979) found that while most building defects are associated with the structure, others are associated with unsightly patterns of soiling of the elevations and lack of accessibility to services. Whereas, for a building not more than 25 years old, Ranson (1981) showed that design faults and specifications comes the second after the wear and tear factor.

Accordingly, any reduction in resources applied to building maintenance will have a visible effect on the national economy. The easiest way to cut maintenance costs is to stop doing maintenance. This approach is simple, but the long-term results are usually very costly. Thus, the goal of the new approach is to carry out as little maintenance as possible as infrequently as possible while at the same time preserving the availability of the services facilities, the building elements and the whole building. In other words, maintenance should be carried out only when necessary to ensure the continued, safe and profitable use of the building at acceptable levels of satisfaction or when there is the possibility of extending the useful life of the elements of the building.

1.2 Objective of Study

The relative objectives to achieve the aim of study are as follows:

a. To identify design defect which affect building maintenance.
b. To identify the systematic framework for selecting a suitable maintenance strategy for each individual item in a building.

1.3 Scope of Study

The scope of this study will be limited to civil engineering and architectural building features associated with certain faults and defects, and by implication, impact preservation of buildings' structural and fabrics integrity design that affect maintenance. The Museum Adat Istiadat Melayu Kelantan (Muzium Islam) will be investigated to illustrate the problem related in this study.

2. LITERATURE REVIEW

A research by Arditi and Nawakorawit (1999) has shown that design plays a major role in determining the condition of buildings after completion, especially in the aspects of defects and maintenance. Indirectly, design influences the performance and physical characteristics of the building and its durability to stand against environmental elements, noise and social interferences such as graffiti and vandalism. Therefore, the link between design and maintenance should not only be seen from the point of increasing number of repair works or cost involve, but it needs to consider also the impact of a design on structure and materials installed as well as the life cycle for each of the components or elements of the building.

Arditi and Nawakorawit (1999) have listed 22 sources of the major maintenance related complaints on building defects that designers and property owners each, reported receiving from clients. The complaints are categorized in 5 groups:

a. Safety
b. Design quality
c. Maintenance
d. User comfort
e. Building services

Results from the survey show that complaints on user comfort become the clients" number one concern followed by maintenance. Design quality category was almost not in their list as most clients (and designers themselves) believe building design and building operation/maintenance are entirely separated and both building owners and designers did not consider serious enough.

Assaf et al 1996 identified 11 major groups of faults; they are, the defects in civil design, architectural defects in design, design defects in maintenance practicality and adequacy, defects due to consultant firm administration and staff, defects due to construction drawings, defects due to construction inspections, defects due to civil construction, defects due to contractor administration, defects due to construction equipment, defects due to construction materials and defects due to specifications.

Through literatures, the author had identified six dominant variables that are associated with the decision making of building cost. The variables are:

a. Existing building condition

The existing conditions of buildings were assessed through several ways. One of the easiest methods is by using visual survey. The exterior of each building structure was viewed from the ground level and all important information would be documented and some areas of deterioration are noted through annotated sketches and plans.

b. Building age

Age of building provides important indication on level of maintenance service required. The important elements that need to be considered in allocation of maintenance resources is the building's age. In order to know the future image of building, building manager needs to offer right service so that the building has competitive advantage (Lateef, 2010). Services given must meet expectation in response to time, delivery schedules and within the agreed performance indicator.

c. Complaint received regarding building performance

Inefficient maintenance works could invite complaints by the building users. Users are normally looking for a comfortable space in a building. This includes well functions of building equipments, clean environment, and safe. If buildings do not fulfil the user's needs, complaints with regards to maintenance performance would be made by the users.

d. Client's request

Client refers to the owner of a building. The client is the one who provides maintenance finance and a project brief in the early stage of maintenance works. Quality of client's brief would influence the building performance. The key successful of design is rests much with the clients besides others such as a good budget.

e. Availability of funding

A most trying constraint from the design point of view is the budget allocation. That most important factor contributing to successful maintenance work was a sufficient budget allocated for a project. The project fund must be sufficient to ensure the maintenance works could run smoothly (Gibson, 1979).

f. Safety and health requirements

Safety and health are command factors influenced in the decision making of maintenance works (Lee, 2009). This is because building maintenance works can lead a risk to maintenance personnel and building users. It is the responsibility of building stakeholders particularly the manager to ensure that health and safety assessments and safety work procedure are documented.

2.1 Building Maintenance

Building maintenance is a major activity in most countries. Any reduction in resources applied to building maintenance will have a visible effect on the economy. Few years ago, a rapid growth of housing construction clearly appears as a part of the country development. The number of modern houses increases and more houses are being constructed. As a result, more maintenance work is required in order to cope with this type of construction.

In maintaining a building, there are usually several strategic options available to management, and many alternative decisions to be considered. There is, for example, the possibility of reducing the demand for maintenance by addressing the actual cause of failure and identifying its consequences. For instance, it may be necessary to decide whether to repair or replace an item, and whether to carry out periodic maintenance at fixed intervals or simply to respond to the requests of the users. Thus, building maintenance can be divided into three strategies:

a. Corrective

Corrective maintenance is the simplest type of maintenance strategy, where an element in a building is used until it breaks down. It covers all activities, including replacement or repair of an element that has failed to a point at which it cannot perform its required function. Corrective maintenance is sometimes referred to as failure-based or unplanned maintenance.

b. Preventive

Preventive maintenance was introduced to overcome the disadvantages of corrective maintenance, by reducing the probability of occurrence of failure and avoiding sudden failure. This strategy is referred to as time-based maintenance, planned maintenance or cyclic maintenance. Preventive maintenance tasks are performed in accordance with a predetermined plan at regular, fixed intervals, which may be based for example on operating time. Such a strategy is frequently applied to external or internal paint work.

c. Condition-based

Condition-based maintenance is defined as: "Maintenance carried out in response to a significant deterioration in a unit as indicated by a change in monitored parameter of the unit condition or performance" (Kelly and Harris, 1978). The condition-based maintenance concept recognizes that a change in condition and/or performance of an item is the principal reason for carrying out maintenance. Thus, the optimal time to perform maintenance is determined from a condition survey used to determine the actual state of each constituent item in a building.

2.2 Building Defect

Defects within new buildings are areas of non-compliance with the Building Code of Practice and published acceptable tolerances and standards. Older buildings, or buildings out of warranty period, may not comply with these standards but must be judged against the standard at the time of construction or refurbishment. Building maintenance can be broadly as the application of scientific principles to the care and preservation of building assets. A building, product or application can become defective through age and lack of maintenance and professional advice should be obtained to ensure that replacement or works undertaken are using materials and current systems of application that are appropriate to the building. Examples of building defects are where structural settlement cracks are occurring to brickwork, excessive structural sagging to a roof, ill-fitting windows and doors, leaking showers and sagging ceilings. Building maintenance is therefore concerned with the technical requirements of investigating and diagnosing faults to assess condition; with the organizational needs to specify, select, implement and supervise remedial works and with the management operations to plan, preventive and corrective programmers (Rozita Aris, 2006).

2.2.1 Failure of buildings

Buildings may fail for a number of reasons. Figure 1 shows a breakdown of the most common reasons for this.

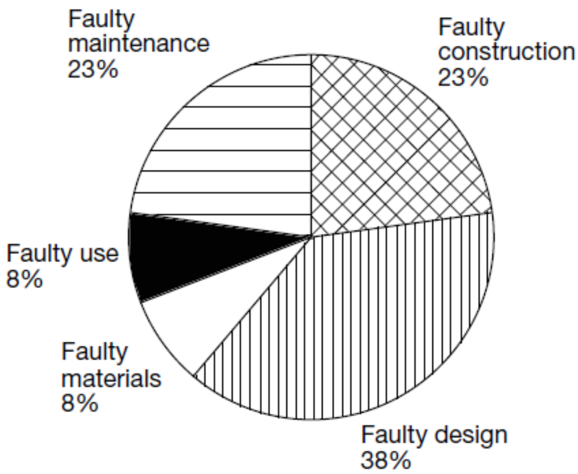

Figure 1*. Breakdown of the most common reasons (Ahmad, 2006).*

2.2.1.1 Faulty Design

Faults in building design place a heavy burden on the building for rest of its life and there is no compensation for it. In such situations, the responsibility falls on the shoulders of the designer in that they must think carefully with full concentration and consideration towards completion of their design project. Explaining the link between maintenance and building design, suggests that four sectors of building design should be considered and regarded as important if one is to avoid the need for unplanned maintenance at the post-occupation stage (Ramly, 2006). These sectors are:

a. The main fabric which includes walls, floors, roofs, doors and windows;

b. Internal finishes which includes ceiling and wall finishes as well as floors;

c. Special design features such as decorative elements for the doors, windows, glass, air vents and special brick and stone work;

d. The fourth and the last sector is cleaning and house keeping of all building components.

This study identified deterioration in these sectors that resulted from design faults, which subsequently imposed a heavy financial burden on the occupier or owner. Based on the outline of causes derived from the work of the implication of design fault on maintenance in building has resulted from the following (Gibson, 1979):

a. The consequence of thermal movement;

b. The consequence of inefficient detailing;

c. The consequence of improper material selection; and

d. The consequence of poor design for access for maintenance measures.

Faulty design decisions are the most common faults which may be grouped as follows: failure to follow well-established design criteria in the choice of structural system and selection of materials; ignorance of the basic physical properties of the materials; use of new materials or innovative forms of construction which have not been properly tested for use; misjudgement of climatic conditions under which the material has to perform; and poor communication between different members of the design and construction teams (Lee, 1987).

In most of the literature reviewed by Boyle (2003), Ramly (2006), Reason (1990), Melchers (1989) and Porteous (1992) the words "fault," "failure" and "defect" were used interchangeably to describe imperfections in constructed buildings. A building defect or fault may be considered to be a failing or shortcoming in the function, performance, statutory or user requirements of a building, and manifests itself within the structure, fabric, services or other facilities of the elected building (Watt, 1999). The term "defect" or "fault" is preferred in this paper, but not in the sense of the building or components failing to full intended functions (Allen, 1971).

Building faults, failures or defects are emotive terms and mean various things to deferent people, but always suggest that the client involved has had an unsatisfactory solution (Campbell, 2001). The vast majority of building faults and failures are not dramatic collapses, but far less newsworthy structural or mechanical problems and issues of serviceability such as minor cracking and water ingress (Holtham, 2001). Other signicant factors included inadequate design information and poor site practices. The detailed results of the research, summarized in Building Research Establishment (Building Research Establishment, 1988) provide a useful attribution of fault to particular building elements (see Table 1). Another research, conducted by the BRE Advisory Service and based on its database of building defects (Watt, 1999), identified the ten main types of defect shown in Table 2.

Table 1. *Building elements and fault attribution*

S/No	Building element	Fault attribution (%)
1	External walls	20
2	Roofs	19
3	Windows and doors	13
4	Floors	11
5	Services	9
6	Sub-structure	7
7	Internal partitions	4
8	Separating walls	4
9	Stairs	4
10	Planning and layout	4
11	General and external works	4

Note: Adapted from Watt (1999).

***Table 2.** Main defect types*

S/No	Defect types	Magnitude (%)
1	Rain penetration	22
2	Condensation	17
3	Cracking	16
4	Detachment	13
5	Entrapped water	6
6	Sound transmission	5
7	Rising damp	5
8	Floor (not involving moisture)	3
9	Indoor air quality/ventilation	3
10	Appearance/discoloration/surface defects	2
11	Other	8

Note: Adapted from Watt (1999).

2.2.1.2 Faulty construction

Faulty construction is one of the most common causes of early deterioration. Common construction faults include inadequate compaction and failure to position the reinforcement so that it has adequate concrete cover. Under almost any exposure conditions these faults will eventually reduce the service life of the structure as a result of reinforcement rusting after the concrete has become carbonated, (Seeley, 1987).

Faulty construction accounts for many building failures and this can be linked to the above. If a new system has not been specified or used previously, then the builder will have no experience of this system and may build it incorrectly. Sometimes lack of suitably qualified supervision can lead to this problem, as can the lack of testing that is carried out during construction work, such as concrete tests. An example of using systems that have not been used before creating this problem is that of the use of precast concrete structural frames. During the 1950s, in order to increase the amount of housing the development of high-rise blocks of flats became endemic.

2.2.1.3 Faulty maintenance

Faulty maintenance accounts for a similar number of building failures, and this can be broken down into two parts: maintenance that has been carried out incorrectly, or more commonly where no maintenance has been carried out during the life of the building. A large section of this book is dedicated to building maintenance, and how it can lead to the improved performance of buildings over their lifespan. If the procedures specified in this book are adopted then this figure should be reduced.

However, maintaining buildings costs money, and therefore although building maintenance can be planned and specified correctly, if the funding available is not adequate this will ultimately lead to building failure (Ahmad, 2006).

2.2.1.4 Faulty materials

Faulty materials account for fewer, but still substantial, amounts of building failures, and the reasons for this are to a certain extent the same as for faulty design. We cannot test all materials for 60 years before they are used for construction, and we cannot test all materials in conjunction with all of the materials that they may potentially come into contact with. However, as a general rule, materials that are manufactured in factories will be of better quality than materials manufactured on site (in situ). More prefabrication should reduce this problem, but it can be argued that prefabrication reduces design flair and flexibility (Ahmad, 2006).

2.2.1.5 Faulty use

Faulty use accounts for some building failures, and this generally occurs where the building is not being used for the purpose for which it was designed. For example, occupiers may wish to create more space and therefore knock down walls without the advice of the designer, which can have major implications and possibly cause a collapse (Ahmad, 2006).

3. METHODOLOGY

To achieve the objectives of this research, the following steps were carried out:

a. A literature search was made on the factors that affect maintenance.

b. Site visit, interviews and discussions with owners, designers and contractors. They helped to get the data and to know the design and construction factors that affect maintenance.

c. From the preliminary interviews and the literature review, a questionnaire was formulated.

d. A pilot study was performed before distribute the final questionnaire draft to the respondents. This pilot study was served to:

    i. Test the adequacy of the questions and review the adequacy of provided space for each question.
    ii. Estimate the time needed to fill out the questionnaire.
    iii. Review the more possible answers and increase the lists of choices.

e. Then questionnaire was distributed by post, hand or email to the concerned respondents (evaluators).

f. The questionnaire was collected but the uncompleted questionnaire will exclude.

g. Examining the questionnaire collected from evaluators and obtaining the weighted overall factors affecting building maintenance. The flow chart shows the methodology for the research.

The investigation for this study will be undertaken in two phases. The first phase will involved a literature search on the identification of some of the relative certain faults and defects, and by implication, impact preservation of buildings' structural and fabrics integrity in design stage that affects maintenance. The second phase will involved gathering data through site visits, interviews and discussions with owners, architectural/engineering firms (A/Es) and contractors.

3.1 Research population / sample

In this study, a questionnaire will be distributed to owners of the building, A/Es and contractors to facilitate input in a more standardized manner and will be design later.

3.2 Proposed data collection method

The interviews will be conducted with the questionnaire form highly supported by visual photographs and illustrations. Site survey will be conducted after each interview for further detailed investigations.

The questionnaire will be divided into three main sections under the categories of certain faults and defects, and by implication, impact preservation of buildings' structural and fabrics integrity. In the all sections of the questionnaires, the respondents will be asked to select the relevant answer from the choices provided. Figure 2 show the methodology flow chart.

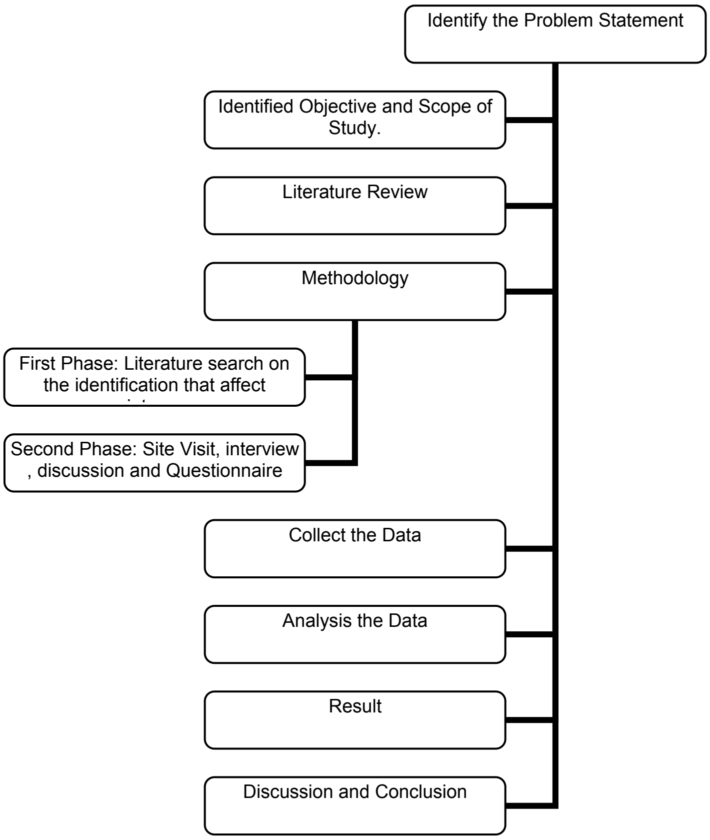

***Figure 2**. Methodology Flow Chart*

3.3 Data analysis

The collected data will be analyzed quantitatively using the severity index. The aim of formula is to determine the ranking of parameters or factors given for each answer of the questions. For every question there are five parameters that should be used by the respondents as an option to answer the questionnaire. The five options given are extremely affect, strongly affect, moderately affect, slightly affect and does not affect responses.

The severity index is computed by the following equation (Al-Hazmi, 1987):

$$I = [\sum_{i=1}^{4} (ai * Xi) / 3] \times 100\% \qquad (1)$$

Where:

I = Severity index

ai = Constant, expressing the weight given to ήh response, i = 1,2,3,4

Xi = Frequency of the ήh response given as a percentage of the total responses for each defect

i = Response category index where i=1,2,3,4

The severity indexes for all design defects and the groups of defects were calculated using the above formula (1). These indexes were ranked for owners, contractors and A/Es.

4. EXPECTED OUTCOMES

It is expected that the result will show an importance of selection and design features of buildings with their operation, and maintainability. A suggestion of guideline model will be developed and used by conservation contractors related to structural and fabrics integrity in order to maintain and sustain while restoration costs are saved.

5. CONCLUSIONS

For the conclusion, the objective of this study can be achieved if all questionnaire are done accordingly to its proper procedures and hopefully at the end of this study few designs will be indentified and a feed back to upstream creation stages of the building, in order to achieve downstream less defective buildings and better preservation through their life-cycle.

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PROBLEMS OF CONSTRUCTION QUALITY IN SARAWAK, MALAYSIA

Ibrahim, S. H.[1], Baharun, A.[2] and Ayagi, K. M.[3]

[1, 2,3]Universiti Malaysia Sarawak, Sarawak, Malaysia

[3]kabiruayagi@yahoo.com

ABSTRACT: Housing is a major concern for all people in every corner of the world as the wellbeing of a country is reflected in its people enjoying a certain standard of living. Residential and neighbourhood satisfaction is an important indicator of housing quality and condition, which effects individuals' quality of life. The housing industry in Malaysia is regarded as one of the major industries contributing to the economic and social development of the country. The industry regulated by several set of rules and the imposition of regulations, guidelines and standards to ensure that all categories of houses will be constructed according to the acceptable standard. Despite the existence of these regulatory measures, yet, there are many problems faced by house-buyers in Sarawak with regards to less quality construction of houses. There is a standard form of sale and purchase agreement (SPA) stipulated under the Housing Development (Control and Licensing) Regulations 1989 (HD Regulation 1989) but as far as the problem of sub-standard construction of houses is concerned, Sarawak house-buyers are not well protected. In general, there is no benchmark in Sarawak to measure the standard of quality of houses constructed by developers. The aims of this paper is to look into some recent amendments to the legal provisions (and its loopholes) concerning the statutory duties imposed on the three most important institutions related to building of quality houses, namely: the local authorities, the Ministry of Housing and Local Government (MHLG) and the Construction Industry Development Board (CIDB).

Keywords: House-buyers, local authority, developer, house and construction.

1. INTRODUCTION

Malaysia is known as a country having a very wide set of rules and procedures relating to housing development. In spite of this legal environment and controls, the housing industry is facing with consumers or purchasers demand for quality housing from their developers. In this paper the meaning of 'quality housing' is referred to as housing that free from defects i.e. house with good quality of workmanship and materials. Literally the term 'defective' may not accurately be used in the context of a building, but it is common in the construction industry that the term 'defective' be used interchangeably with the term 'less quality'.

Housing property and its outlook:

Housing property in Malaysia hopefully would continue to register growth especially fuelled by the growing popularity of the low mortgage rate, the continuing of the government's Malaysia my second home program in promoting home ownership by the foreigners and the government's success in managing the performance of the overall economy have contributed to the industry's encouraging performance so far. These factors have contributed to propel the property purchase activity, resulting in good performance of new sales in choice locations. Moreover, Malaysia high saving rate coupled with young demographic profile, declining average household size and urban migration would provide a consistent and sustainable stream of house buyers.

Most of the housing developers were generally still positive of the industry's performance in the year 2006 although concern over escalating of oil prices, the performance of the stock market and potential interest rate hikes are viewed with some degree of apprehension. They also agreed that the present situation was a buyers' market where supply exceeds demand which resulted to abandoned houses and buyers had the luxury choice and time to decide on a property they intend to purchase.

With competition building up, industry players are now more proactive and resort to more coordinated planning, including research and feasibility studies, before embarking on any new projects, follow by adopting means of attracting buyers to promote sales.

Population and demand of Housing:

The population in Sarawak has increased tremendously every year due to the facts that people use to migrate to the area as it provides a lot of job opportunities. Table 1 showed the population of Sarawak for the year 1980- 2000 according to the department of statistics Malaysia.

Table Density of population

Table 1. *Population of Sarawak for the year 1980-2000*

State	Size of population		
	1980	1991	2000
Sarawak	1,235,553	1,642,771	2,009,893

The Capital city and its surrounding areas registered the highest growth of population with Kuching having an average annual growth rate of 3.24% from 1991-2000 (494,109 people in 2000) as against 3.11% from 1980-1991.

The 2000 census showed that the second most populace administrative district 10years ago was Miri (221,055) followed by Sibu (210,118) and Bintulu (139,012).

Kuching had 369,065 people in 1991 as per the census taken that year while the census taken in 1980 showed its population then at 262,085.

The 2000 census revealed that out of the 2million people in Sarawak, 52% are living in the rural areas. Out of the 29 administrative districts in Sarawak, only four have more people living in urban areas – Kuching (85.5%), Sibu (79.7%), Bintulu (74.6%) and Miri (76.5%).

Based on the growth rate 3.24% per annum, the demand for houses would quite substantial. Rapid urbanization and the country's demographic profile with its sizeable population of young people offer tremendous opportunities for developers of medium priced houses. 60% of the Malaysian population is below 30 years old. In addition 17% of this category are potential first time house buying candidates.

Also reported in the Kajian Rancangan Struktur Negari Selangor (2002), during the year 1995-1999, average monthly income in Malaysia was RM2, 020 in 1995 and had increased to RM2, 472 in 1999 with the average growth rate of 5.2% per annum.

Sarawak basic economic indicators:

- Sarawak population - 2.4 million (2007e)*
- Number of household – 480,000
- Urbanization – 2007- approx. 50% (1970 – 20%)
- New housing requirement – 15,000 units per year **
- Sarawak GDP Growth rate – 9.3% (2007a) (5.0% in 2008e) (4.0% in 2009f)
- Construction contribution to state GDP Growth – 3% (2008)*
- Percentage of employment in construction – 88% (114,000) of the total 1.3 million work force (2007)*

  Note: * means State planning unit. ** Estimated by C.H. Williams Talhar Wong &Yeo
  (a) Actual (e) estimated (f) forecasted (Keli, B. A.,2008).

Statistic of houses built in Sarawak (2006 – 2008):

- ✓ Total number of houses built in Sarawak 2006 – 2008 are 13,056 units
- ✓ Average number of houses per year is 4,352units

- ✓ Kuching has the highest number of houses built at 6,862 units, while Samarahan second highest at 2,071 units and Miri the third highest at 1,894 units followed by Sibu at 1,607 units. Sriaman has the least number of houses built at 56 units.
- ✓ Double and single storey terrace houses are the most built: 7,142 units or 54.7%.
- ✓ Double storey detached are the least built: 19units or 0.15%
- ✓ Apartment and condominium: at 881 units or 6.8%.
- ✓ Low cost houses: 871 units or 6.7%.
- ✓ Low cost plus houses: 912 units or 7%.
- ✓ Double storey semi – detached: 1386 units or 10.6%
- ✓ Single storey semi – detached: 450 units or 3.44%.

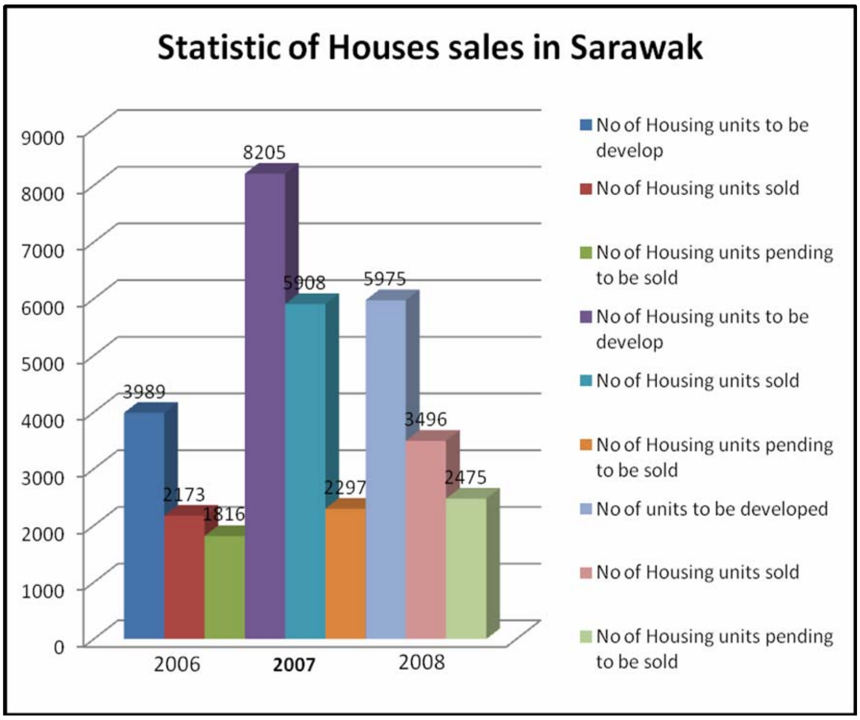

Figure 1. *Statistic of Houses Sales in Sarawak*

2. THE PROBLEMS OF UNQUALITATIVE CONSTRUCTION OF HOUSES IN SARAWAK

There are three categories of developers involved in housing development in Malaysia: Private developers, statutory bodies and co-operative societies. All these developers are governed by the housing development (Control and Licensing) Act 1966 (HDA) when they undertake housing development involving the construction of more than four units of housing accommodation. In Malaysia the practice is ‘sell then build’ i.e. houses are offered for sale prior to its construction or completion or even prior to the cleaning of land. The payment of the purchase price of houses is made progressively according to the stages of completion of construction as regulated by the housing development (Control and Licensing) Regulation 1989 (HD Regulation 1989). The system does not allow house-buyers to view the houses they purchase while having to accept the delivery of houses irrespective of their quality. Despite the common practice of ‘sell then build’, the Malaysian government is actually encouraging developers who are willing to implement the ‘build then sell’ in their housing development. Consequently effective from 1st December 2007 the HD Regulation 1989 has been amended in which the new Schedules of I and J have been inserted in order to accommodate such practice. However it is still debatable whether the practice of ‘build then sell’ will solve the problem of quality building. The only clear advantage as regard to quality of building is that to give opportunities to interested purchaser to view the house and detect any defects which are visible at the time of viewing the house. Thus how about the latest defects?

The right purchaser for quality houses in terms of good workmanship and quality materials is derived from the standard Sale and Purchase Agreement (SPA) (Schedule G and H of the HD Regulation

1989). Clause 14 uses the phrase ‘the building shall be constructed in a good and workmanlike manner’. This clause requires a developer to construct houses in good and workmanlike manner where the workmanship and materials used must conform to description. In other words there are two express terms in this clause; materials and workmanship must conform to description; and construction must be done in a good and workmanlike manner. Base on this clause any purchaser having a problem of sub-standard construction of house may claim remedies against the respective developers.

Generally, there are two kinds of remedies; first the remedies under the law of contract and second, the statutory remedies available during the statutory warranty period. The statutory remedies are derived from the standard SPA under the HD Regulation, 1989. Under the recent amendment of the HD Regulations (which came into force on the 1st December 2007), the defect liability period is valid for twenty four months from the date of handing over vacant possession of the completed unit. Since the defect liability period is quite long, this may give quite comfortable protection to purchasers. However there would be a problem of latent defect which may occur only after some time and may not show any sign during this liability period. Hence the effected purchaser still will have to take the hassle of remedying the defect at their own expenses.

2.1 Typical complains from the buyers

The star (2001) had reported that most common complaints received from the buyers are: late delivery of the houses; houses delivered without certificate of fitness for occupying; poor workmanship; misleading advertisement and houses hand over without water or electricity power supply. The complaint would surely affect the reputation of the developer and its property sales. Therefore it is another important aspect requires attention by the developer. Less or no complaint from the customers reflects good quality of houses have been delivered to the buyer. This would have direct or indirect effect in boosting the future launch or sales of properties by developers.

2.2 Late delivery of houses

This is where the houses are not being able to deliver to the purchasers within the timeframe stipulated in the Sales and Purchase Agreement. In accordance to the clause as stated in the Sales and Purchase Agreement, the purchases are entitle to claim for late delivery of the houses at the rate of 10% per annum or the purchased price from the developer. But very often, most of the purchasers are not able to recover the full sum or in some cases, not only at all as the developer claimed that they are in financial difficulty. Hence, even when the purchasers take legal action against the developer and has a favourable judgement, they may still facing difficulty to realize their full amount of claim as the developer has no money. Sometime the purchasers are faced with more legal problems and cost incurred during the legal process. However, with the understanding of the government, the Ministry of Housing has set up Tribunal in 2002.

2.3 Poor quality of the Houses been delivered

There are many complaints from the house buyers that their houses delivered are of poor quality or with a lot of defects. Some of the more common defects have been highlighted such as roof leaking, wall crack; bathroom leak and the material used are of the poor quality type. Even though there is a clause in the Sales and Purchase Agreement spelt out that the developer would have to rectify the defects at his own cost and expense, but again, most of the developers would delay it and only rectify the defect at a very late stage or after received many complaints. Worse still, when the defect rectified has reoccurred at the same place. Other problems are such as about the recovery of costs from the developer if the buyers themselves rectify the defects or the defect showing up after the expiry of the defect liability period.

Fighting the case in court would definitely involve a very long drawn legal battle which is also financial taxing. However, for those cases that pass all the conditions set by the Ministry of Housing, the buyer could bring the cases to the Tribunal set by the ministry for assisting the claim against the developer. The only set back was that most of the buyers are not very familiar or aware of the term and conditions of the claim or worst still the available of the assistance.

2.4 Houses been delivered without certificate of fitness for occupation

This is where architect or engineers work hand in glove with the developer in issuing the certificate of completion even before the actual completion of the houses. The developer has the intention to hand over vacant possession of the houses to the purchasers without applying for certificate of fitness for occupying. More often than not, this goes unnoticed because most of the financial institutions only depend on the architect's certificate when releasing money from the purchaser's loan. In this case, the purchasers would surely suffer, sometimes to the extent of the purchaser been summoned by the financial institutions because they are not able to service their loan regularly due to other commitments.

2.5 Houses been delivered without Water and Electricity Supply

The houses have been constructed and hand over to the buyers but without the water or electricity supply been connected. No doubt, the responsibility of the developer end with the term that all internal works has been constructed and ready for the external supply's connection, it could still be the developer's fault due to developer failed to contribute the amount of money required by the authority concerned.

2.6 Misleading Advertisement

This is where the developer has mislead the customers by not providing the same quality or specifications as promised or advertised in the brochure, newspaper or actual specifications, design or building material used were differed from the show unit.

How to solve the problems of sub-standard construction in Sarawak:

There are three (3) main authorities in Malaysia having power to control building quality i.e. Local authority, the Ministry of House and Local government (MHLG) and the Construction Industry Development Board Malaysia (CIDB).

3. LOCAL AUTHORITY

3.1 Approval of building plan:

The approval of building plan is vested with the local authority where the local authority shall ensure that the applicant-developer has complied with all necessary rules and regulations, in particular the UBBL.

If the housing development involves construction of low-cost units, the local authority will have to check if the building plans submitted for approval are in compliance with the CIS 1 or CIS 2. It is within the power of the local authority to ascertain that the building plan submitted by a qualified person. By-law 5 of the UBBL provides that the responsibility to supervise the construction works until its completion lies on the qualified person who has submitted and certified the plans and specifications for building approval, and issued the certificate of completion and compliance (CCC). Generally Malaysian context, the qualified person would be an architect. Thus in cases of defective houses, a local authority has power to act against an architect.

3.2 Issuance of certificate of completion and certification:

A new method of processing the issuance of Certificate of Fitness (CF) has been introduced and now it is known as the CCC. Prior to the amendment of several statutes governing the housing industry the process of issuance the CF was under the jurisdiction and duty of the respective local authority. Under the CCC, the certificate of fitness is issued by any architect or building draughtsman or engineer upon completion of the construction of a house according to its plan and specifications as approved by the local authority. Under the CCC, the responsibility of the local authority is transferred to the professionals. The main issue here would be on the extent of independency of the professional involved. This CCC seems to be a move towards self-certification, which may remedy the problem of delay in issuance of the CF, but it may not be an answer to the problem of defective houses (Azlinor Sufian, 2001).

3.3 Supervision or inspection of construction work:

The SDBA and UBBL do not impose any specific duty on the local authority to inspect the construction work done by a contractor. As mentioned earlier this duty rest solely on the shoulder of a qualified person who submitted the plan. The local authority will only conduct an inspection if there is a report on failure of building (section 70B, SDBA). A failure of building refers to defective building which may affect health and safety of the occupants, such as failure of the structure. Therefore mere defect such as sub-standard quality of paint or uneven flooring may not fall within the meaning of building failure.

4. MINISTRY OF HOUSING AND LOCAL GOVERNMENT:

4.1 Licensing of developers:

Housing development in Malaysia falls within the purview of the MHLG. No housing development may be under taking without licensing granted by the MHLG. The main law governing housing development in Peninsular Malaysia is the HDA. In Sarawak the applicable law is the Housing developers (Control and Licensing) Ordinance 1993 (No. 5) and in Sabah, the Housing Developers (Control and Licensing) Ordinance 1978 (No. 24).

Under the HDA no housing development may be carried on, undertaking or caused to be undertaking except by a housing developer in possession of a license issued under the Act (section 5(1). It is an offence to carry on a business of housing development without a license, as decided in the case of Kheng Soon Finance Bhd. V MK Retnam Holding Sdn. Bhd. & Anor. [1989] 1 MLJ 457; '............to carry on a business of housing development without license is a serious offence........' The controller of housing has a power to revoke or suspend the license if he is satisfied that the license housing developer is carrying on his business in a manner detrimental to the interest of the purchasers or to any member of the public, or has insufficient assets to cover his liabilities, or is contravening any of the provision of the HDA or has ceased to carry on housing development in West Malaysia. Generally through the licensing system, the MHLG can control the activities of the developers.

The imposition of HDA on the license developers will enable the authority to utilize its provisions to promote serious commitment from the licensed housing developers to delivered quality houses to their purchasers, particularly through the enforcement of section 6A, 6B, 7A and 11 of the HDA. Section 6A of the HDA may be used by the MHLG to make developers comply with the obligations imposed on them through the standard sale and purchase agreement. Section 6A stated that the controller has the power, subject to any direction of the Ministry, to keep the deposit made under paragraph 6(1) (a) and (b) until the expiry of the defect liability period of the housing development. If the MHLG finds that a particular developer has not taken proper action in relation to complains of purchasers on defects of workmanship or materials, the MHLG may use the power under this section to withhold the return of deposit until the rectification is completed.

Similarly section 6B (a) of the HDA may be invoked by the authority in which the section empowers the controller to forfeit the whole or part of the deposit if any licensed housing developer is carrying on his business, in the opinion of the controller, in a manner detrimental to the interest of purchasers [emphasis added] or to any member of the public. This section may be considered as one of the methods to make developers comply with the terms and conditions of the sale and purchase agreement. Nevertheless under this section, the controller must be satisfied that non-compliance on the part of the developer with the terms of sale and purchase agreement has really been detrimental to the interest of purchasers. This may be possible in cases of serious defects such as defect of workmanship for substructure works or in cases where the whole units of houses in the project are suffering from poor quality of workmanship and /or materials.

The concept of withholding and forfeiting the deposit above may be applied also towards the monies in the Housing Development Account. Section 7A of the HDA requires a developer to open and maintain Housing Development Account for his housing project. The management of the Housing Development Account is specifically laid down in the Housing Development (Account) Regulation, 1991. It is provided in the regulations that all monies in the Housing Development Account may be withdrawn when the housing development has been completed; and the solicitor for the developer has satisfied that the obligations of the developer in respect of transfer of all titles under the sale and purchase agreement in that housing development have been fulfilled (Regulation 11). This regulation is to be read together with section 7A of the parent Act where all monies receive by the developer from the sale of the housing accommodation in the housing development shall be paid into the Housing Development Account (Regulation 7A(3)). Money in the account therefore includes the purchasers' monies (Regulation 4). Rectification of defects of houses is part of the developer's obligation towards purchasers in carrying out his duties as a licensed housing developer. Thus, if a developer does not discharge this obligation, it is possible for the controller to withhold the release of all monies in the Housing Development Account. Apart from that, the existing section 11 of the HDA that vests powers in the Ministry to give directions for the purpose of safeguarding the interest of purchasers should properly be enforced. The power of the Minister under this section is very wide where if the controller is of the opinion that the licensed housing developer becomes unable to meet his obligation to his purchasers, the Minister may give direction under Section 12 for the purpose of safeguarding the interest of the purchasers. These powers of the Minister may include giving directions to the licensed housing developer in question to take such steps as he may consider necessary to rectify any matter or circumstance (section 11(1)(a)), direct that a person be appointed or himself appointed a person to advice the licensed housing developer in the conduct of his business (section 11(1)(b)) or to take such actions as the Minister may consider necessary in the circumstances of the case for carrying into effect the provisions of the HDA. It is highly desirable that the Minister shall use his power under this section if consumers are encountering critical problem of poor workmanship and materials. Nevertheless in order to utilize the benefit of this section the consumers must present cogent evidence that a particular problem is really detrimental to their interest as purchasers. Therefore a minor problem of quality of finishing of houses may not be a good enough reason for the use of this power by the Minister.

4.2 Supervision of project

Being a ministry having power to issue the license for housing development, it is the duty of the MHLG to ensure that purchasers would be delivered the houses as prescribed by the sale and purchase agreement as well as to settle complaints on matters concerning housing. Currently there are three mechanisms used to supervise progress of housing development. It consists of supervision through documents (files), visit program and investigation (based on complaints). These methods seemed to be sufficient to monitor the project undertaking by developer. However due to lack of manpower, the site inspection will be conducted only upon receiving specific complaint from the purchasers. This normally happens when purchasers noticed that claims for progressive payments are issued by developers do not correspond with the stages of completion of work.

The new provision of the HDA on the power of supervision and enforcement are more concerned with offences committed under the HDA and the Regulations there under. As for the problems related to the rights and obligations derived under the sale and purchase agreement, the Ministry does not have direct power to take any action against the developer. Thus as far as the right for quality workmanship and materials of building are concerned, the MHLG does not have direct power to act except for giving advice to the developers.

5. CONSTRUCTION INDUSTRY DEVELOPMENT BOARD

5.1 Licensing of contractors

The licensing of contractor is done under the Construction Industry Development Board Act 1993 (CIDBA). The duties and function of the CIDB are stipulated in the CIDBA. Generally the Act provides for the licensing of contractors and accreditation and certification of building materials. Even though the CIDBA is not comprehensive enough to regulate the activities of contractors, the CIDBA has engage administrative mechanisms to promote the growth of the construction industry.

5.2 Training of skilled workers

The CIDB has no direct control over housing development in Malaysia as compared to the local authorities and the MHLG. Nevertheless as far as quality of building in the construction industry is concerned, the CIDB is the main authority which control activities of contractors. It was contended that reliance on the unskilled labour to do manual work is one of the causes for shoddy workmanship (Viswanathan Mukaya et.al 1995). In Malaysia the availability of skilled workers is very important to ensure the quality of building since Malaysia is still using the conventional building system labour-intensive [8]. Under the CIDB skilled workers are described as concreter, bar-bender, carpenter, bricklayer, mason, plasterer, paver, tiller, painter, jointer, metal worker, drain layer, glazier, welder, construction plant operator, plumber, and electrician (Section 32(2). Being a paymaster to the contractors, a developer may indirectly be responsible to ensure that their contractors engage skilled workers since it would have significant impact on the quality of the workmanship of the houses constructed. If developer could not care less on the engagement of skilled workers by its contractors but look for tenders with the lowest quotation, all these would contribute to the poor commitment of contractors to the quality of work of his workers.

6. CONCLUSIONS

Since the problem of Sub-standard quality housing in Sarawak is so rampant, it is timely for the government to consider immediate steps to remedy the situation. It seems that the statutory provisions have conferred sufficient powers to the Ministry to monitor and supervise the performance of licensed housing developers. Nevertheless how far these powers are practically enforced by the MHLG still remains in doubt. The local authorities, the MHLG and the CIDB, these institutions should properly enforce the powers vested on them and take necessary steps to prevent non committed developers and contractors from entering into the business of housing development and construction industry.

In order to achieve government aims in providing adequate and decent housing for all as stated in Ninth Malaysia Plan, something needs to be done especially at the implementation level. Special attention must be given to low cost housing since majority of this country population are in this category. The need for National Housing Policy then become more crucial since the government should provide clear direction for housing development in this country in order to achieve develop nation status by the year 2020.

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AN INVESTIGATION INTO ABANDONED HOUSING PROJECTS IN SARAWAK, MALAYSIA

Ibrahim, S. H.[1], Baharun, A.[2] and Ayagi, K. M.[3]

[1, 2, 3]Universiti Malaysia Sarawak, Sarawak, Malaysia

[3]kabiruayagi@yahoo.com

ABSTRACT: Construction industry has always been one of the major contributors to the national growth, many companies jumped into the property bandwagon to build and build. This research work aims at investigating abandoned developed housing in Sarawak Malaysia, with special reference to a number of areas Kuching, Kota Samarahan and Betong. The completed housing that has been abandoned may be imposed by structural disintegration of the building over the years. Abandoned housing also does indeed have a negative impact on the value of houses in close proximity. Abandoned developed housing is also a threat to the environment because are ideal hide outs for criminals, drug addicts and vice activities. This research will identify the kind of problems faced with abandoned developed housing and discusses the challenges facing case study areas in addressing the problem. The problems and information will be generated through questionnaire and personal interview with developers, an analysis of reports and plan prepared by public agencies and community development corporation involved in revitalization effort in case study areas.

Key words: Abandonment, housing, developers, effect and causes

1. INTRODUCTION

Housing development is one of the important agendas in Malaysia since independence. It has been placed in the list of the top Government's priorities in the Malaysian Plan. After Independence, one of the most important objectives of the Government is to restructure the society and eradicate poverty , through the means of the various Malaysian plans, the pre-emptive new Economic policy (NEP) and the National Development policy (NDP) adopted by the Government. One of the approaches contained in these programmes and means, is to provide sufficient and suitable housing accommodation to its citizens Malaysian. However, due to the high demand of housing, the government couldn't afford to build totally on their own; therefore, private developers were jointed to construct housing accommodation, subject to the rules and regulations imposed by them. Although housing industry has served as the development, economic and social catalyst and has brought many benefits to the nation, yet, one of the spill-over problems which are carries is the problem of abandoned developed housing.

2. Housing system in Malaysia

The Government of Malaysia recognizes housing as a basic human need and important component of the urban economy. This has led to the formulation of policies and programmed aimed at ensuring that all Malaysian have access to adequate shelter and related housing facilities. In Malaysia housing development programmed are carried out by the public and private sector. The public sector concentrate mainly on low cost housing programmed while the private sector (housing developers) a part from complying on the 30 percent low cost housing units, concentrate on medium and high cost housing programmed. The Malaysian Government has also formulated a housing policy which aims to strengthen the involvement of private sector in housing production and delivery especially in housing schemes development (Asiah Othman,1999).

2.1 Industrialized Building System (IBS)

Industrialization in construction industry is part of a process in change and improvement management. The degree of industrialization and its characteristics carried out the different understanding in its terminology and definitions. Industrialized building system (IBS) can be defined as a construction system that is built using pre-fabricated components the manufacturing of the components are systematically done using machine, formworks and other form of mechanical equipment. The components are manufactured offsite and onsite completed will be delivered to construction site for assembling and erection (Abdullahi, M. R. et.al 2009). IBS is also defined as an integrated manufacturing and construction process with well planned organization for efficient management, preparation and control over resources used, activities and results supported by the used of highly developed components (Abdullahi, M. R. et.al 2009). Perhaps the most comprehensive definition of IBS was clarified by Junid, (1986), started that, an IBS in the construction industry includes the industrialized process by which components of a building are conceived, planned, fabricated, transported and erected on site. The system includes a balanced combination between the software and hardware components. The software components includes system design, which is a complex process of studying the requirement of the end user, market analysis, development of standardized components, establishment of manufacturing and assembly layout and prices, allocation of resources, materials and definition of building designer's conceptual frame work. The software elements provide a pre-requisite to create conducive environment for industrialized to expand.

Meanwhile**,** the hardware elements are categorized into three major groups. This includes frame or post and beam system, panel system and box system. The frame structures are defined as those structures that carry the loads through their beams and girders to column and to the ground whilst in panel system load, three dimensional modules (or boxes) for fabrication or habitable units are capable of with standing load from various direction due to their internal stability.

2.2 Modern Building in Sarawak:

The modern building in Sarawak (low-income housing) has not followed the traditional designs, in an eagerness to appear 'modern' many designers in the low-income housing field have abandoned the tradition "tested and proved" solution to the housing problems presented by the local climate and this has resulted in unacceptable discomfort problems (Ibrahim, S.H,2004). The government of Malaysia has been focused more on low medium cost housing during the seventh Malaysian plan with a total of 350,000 units or 44% from the total 800,000 units planned (refer figure 1 & 2.). During the Seventh Malaysia plan the construction of medium and high cost housing by private sector has achieved 187% and 435% respectively of the targeted units. The situation created the over supply of housing stock for both categories during 1997- 2000. The Asian economic crisis worsened the properties scenario in Malaysia with many abandoned developed housing including medium and high cost housing.

In modern low-income housing design, ventilation is provided only through windows and doors and the glazed window areas allow heat to be transmitted in to the house. Most of the low-income houses are constructed using relatively heavyweight materials which absorb heat; the absorbed heat is lettered emitted into internal spaces of the houses making them more uncomfortable. The roof design of these types of houses are built with a short roof overhang which offers little solar shading or protection to the walls. These factors all contributed to an uncomfortable internal environment in the modern house but this can be remedies by air insulation mechanical cooling system such as air conditioners. Unfortunately these can not be afforded by low-income groups. The modern low-income houses also are built with a minimal area and the occupants are forced to adapt to the layout specifications, any extension or renovation must seek the approval from the authority before its build. This process is not only time consuming but incurs extra expenditure for the design process and approval which is not easily affordable by many low income families.

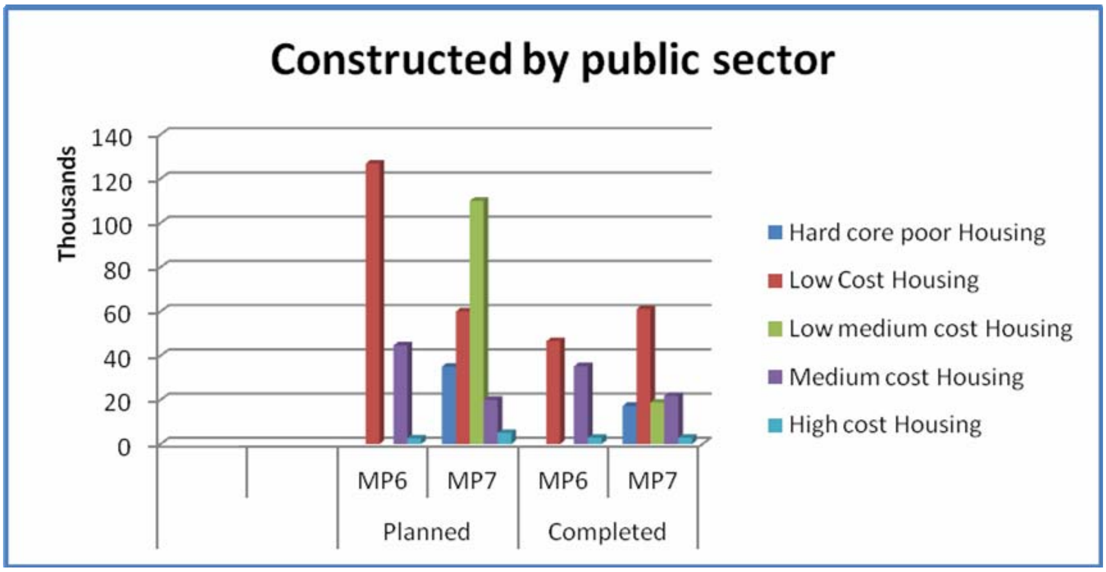

Figure 1. *planned and completed housing units by private sector during sixth and seventh Malaysian plan*

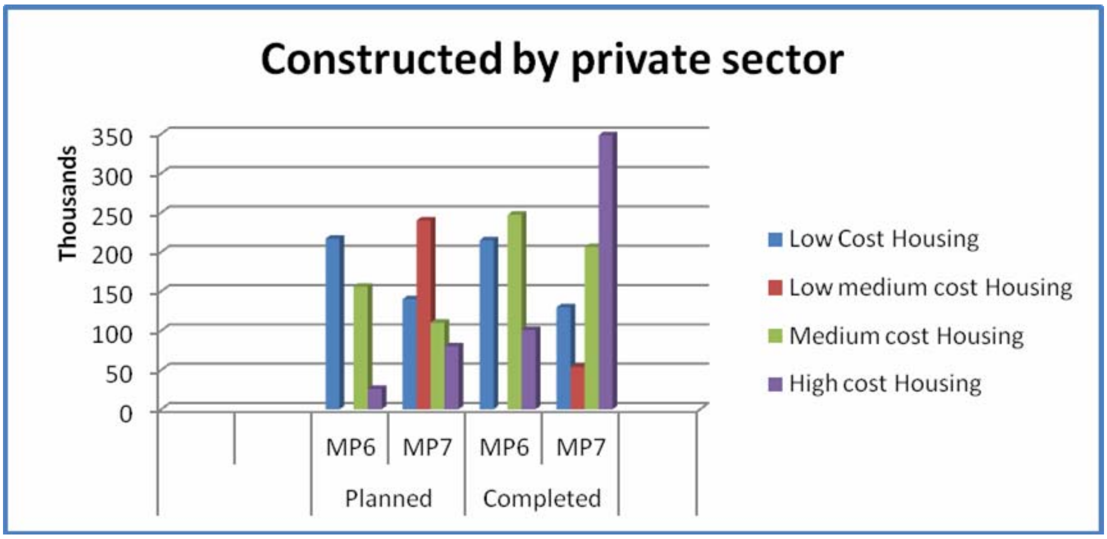

Figure 2. *planned and completed housing units by private sector during sixth and seventh Malaysian plan*
Source: Eighth Malaysia plan 2001

2.3 Traditional Building in Sarawak

Traditional Malay house is well ventilated by large doors and large body height windows which have perforated panel or louvers to remote air movement to reduce high temperatures. Also the house is built on stilts and has floor boards which are spaced with 5mm gaps to allow air to circulate from beneath to the building. Cool air from the ground passes through the floor gaps to replace the warmer air in the house. The traditional Malay house has a pitched roof with large overhangs. The over hanging roofing system plays beneficial role in preventing the rain water from entering the house and it also protect the walls from direct solar radiation. The roof is often built from Nypa or Palm leaves and these materials are good thermal insulator so that little heat is radiated to the interior of the house. Most the houses start with a core house which basically consists of areas for sleeping, cooking and dining. Malay people tend to live in extended families and as such, as the size of the family grows, the size of the house also has to be increased. Due to this reason, most of the traditional houses are easily extended. Distance between the houses are maintained so as to promote good ventilation and since the houses are generally self built, the members of the family have sense of belonging towards their property.

The statistic of the Ministry of housing Sarawak 2006, 2007 and 2008, indicated the number of houses to be developed, houses sold and houses pending to be sold (refer figure 3.)

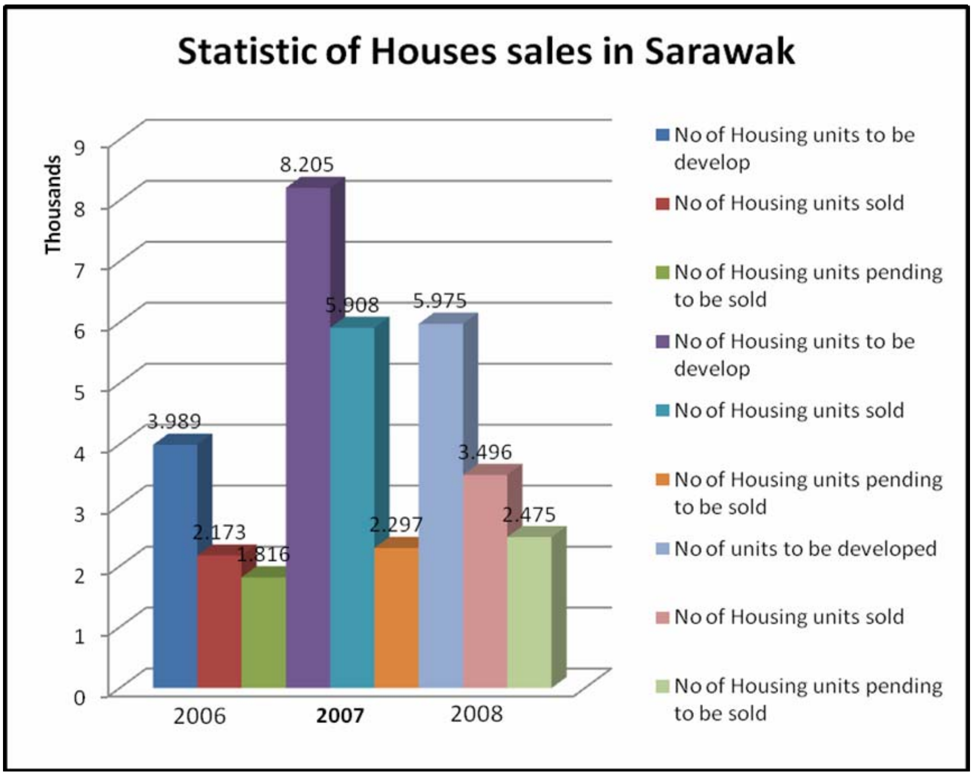

***Figure 3**. statistic of houses sales in Sarawak*

Development of 8 units and below is not included.

- ✓ Average numbers of units to be built by private developers are 6,065 compare to 15,000 requirements per year.
- ✓ Average no of units unsold – 36% per year (Nash, W.W. & M.L. Colens,1959).

3. CONCEPT OF ABANDONED DEVELOPED HOUSING:

Data for studies regarding abandoned structures suffers from inconsistencies with the definition of “abandoned” (Pagano, M.A. & Bowman, A.,2000). Survey designers must contend with the many different interpretation of “abandoned” that each city uses in it own audits Cities criterion includes length of vacancy, structural condition and other subjective factors such as “eminent danger” to the community or threatens a city “health and safety” (Pagano, M.A. & Bowman, A.,2000), some cities use all the above in making determinations on structures.

Abandonment has often been conceived of as an “on-or-off’’ state, but it is actually a multidimensional process, with functional, physical and financial aspects. Sarawak or Malaysia in general has relatively few physically abandoned (vacant) buildings. A large problem is functionally abandonment, some of the low-cost, medium low-cost, medium cost, high medium cost, high cost etc have vacancy rates higher than 50 percent. Financial abandonment is not a problem in Sarawak or Malaysia in general per se, but may be a useful area indicator of other abandoned issues.

Housing abandonment is difficult to describe precisely as mentioned above, but there are nearly as many definitions of abandonment as there are municipal governments tracking the issue and scholars writing about it. The broad contours of the phenomenon and its most extreme manifestation are uncontroversial, but obtaining a definition is important for academics and policy makers.

According to Mallach, 2006, the definition of abandoned housing in the state of New Jersey, that property be legally unoccupied for at least six months and fulfil one of the number of the criteria, such as being in need of rehabilitation, Sarawak or Malaysia in general likely has hundreds of abandoned structures. According to the definition used by Hillier, 2003: that property is neglected functionally, financially, or physically and further more is deemed to be imminently dangerous (Hillier, A.,2003).

3.1 Three different types of abandonment

3.1. Physical abandonment: occurs when owner neglect the interior or exterior upkeep of a property. This could include minor violations of the housing code that relate to the safety and comfort of the occupants or more serious structural problems. When it's often unsafe for people to live in these structures, vacancy is not a necessary precondition of physical abandonment, particularly when squatters or peoples with limited housing option may view these as viable sources of shelter.

3.2. Financially abandonment: occurs when an owner stop meeting their minimum financial responsibilities and property begin accumulating dept in excess of equity. Property tax arrearages, defaulted mortgages, and liens are all indicators of financial abandonment.

3.3. Functional abandonment: occurs when a building ceases to be used: a row house in good repair, on which all property taxes are being paid, but which is boarded off and vacant, is functionally abandoned. A building that is still occupied but no longer has mail service or utilities is like wise functionally abandoned to certain extent.

These three aspects of abandonment are interconnected and often occur simultaneously, but they are sufficient distinct analytically to justify treating them distinctly (see figure 4. below)

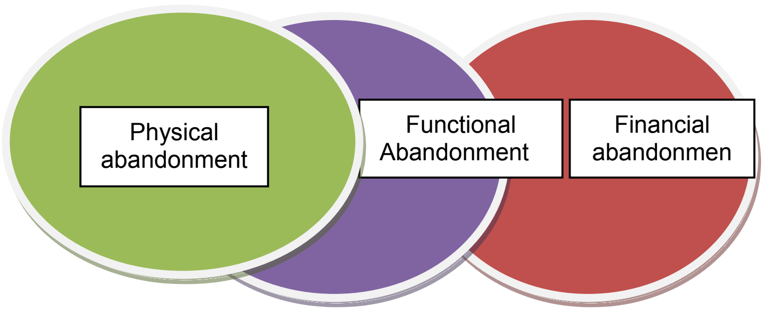

Figure 4. *Abandonment has three distinct but related aspects*

4. Causes of abandonment

The basic causes of abandonment is an owner decide to abandoned his or her property (Mallach, A.,2006) this most often occur when an owner concludes rightly or wrongly, that the potential loses from continuing to occupy or maintain the property exceed the potential benefits. Different types of property are abandoned for different reasons:

- The normal functioning of housing market: According to Accordino and Johnson, 2000 When the rent do not keep up with up keep cost in the low-end rental market, either because housing demand falls (and with it, rents) or because maintenance cost increase (due to increase in labour cost or property taxes).

- Nature of materials used for building: The modern houses are constructed using relatively heavy weight materials which absorb heat; the absorbed heat lettered omitted into internal spaces of the house making them more uncomfortable. The roof design of this type of house are built with a short roof over hang which offers little solar shading or protection to the walls.

- Types of building: The traditional Malay houses start with core house which basically consist of area for sleeping, cooking and dining. Malay people tend to live in extended family and as such, as the size of the family grows, the size of the house also has to be increase (Ibrahim, S.H.,2004). While the modern low-cost houses are built with minimal area and the occupants are forced to adapt to the layout specification, any extension or renovation must seek the approval from the authority before it's built.

- Cost of the houses: The modern housing in Malaysia are expensive therefore, deny the majority of the population, who are poor, access to housing by concentrating the housing resources in the hands of a few, Malaysians (Choon, T. M. 2003), One of the declared objectives of the national housing policy is to provide all Malaysians, particularly those in low-income categories, accessibility to adequate and affordable housing (Choon, T. M. 2003).

4.1 Effect of abandoned developed Housing:

As intimated in the introduction, it takes little by way of imagination to begin to understand the problems created by abandoned buildings, or how these problems are exacerbated as abandoned buildings within Sarawak and Malaysia in general. More systematic studies of abandonment confirm the intuition that abandoned buildings are associated with a variety of social, economic and environmental ills. The chief amongst these problems are:

- Wasted resources and lost tax revenues: Abandoned houses are a waste of resources. This waste is particularly cruel when the scarcity of affordable housing is an important issue for some sections of the community, as it is in the case of groups such as the homeless. At the same time, abandonment involves lost tax revenues for the community as a whole

- Declining property values**:** Economic losses, both private and public, are not confined to abandoned houses themselves. Abandonment affects other properties by lowering property values (Greenberg,et.al 1993). The importance of this issue is attested by the fact that, in the past, whole studies have been devoted to the economics of rehabilitating building so as to increase the value of rents and property selling prices (Nash, W.W. & M.L. Colens,1959).

- Effects on community within the area: Abandoned building have a negative impact upon social as well as purely economic aspects of well being. First, abandoned buildings are often unattractive. At worst, abandoned buildings are eyesores. Their grounds become unkempt and overgrown, while the building themselves become dilapidated through lack of maintenance.

4.2. Solution of the problems:

Abandoned housing and lots represent decline, neglect and devaluation of properties. Malaysia, like many countries face a significant challenge in dealing with vacant and abandoned properties. This challenges is exacerbated both by weakness in the local and regional housing markets including an over supply of housing related to demand. Therefore, to address housing abandonment, the following should be considered:

- Housing design: Most of the houses built for low income groups neglect the importance of ventilation to cool the occupant of the house and other favourable design features which are desirable to promote a thermally comfortable environment. The environmental quality of the low income housing indicated that in terms of thermal comfort due to the lack of understanding of climatic factors in design, many occupant experience high temperature in their houses, especially just after mid day and concluded that one of the main drawback in most of the housing is the poor internal layout. This often fails to relate to the basic social and cultural structure of Malaysian people. To achieve and maintain comfortable living condition in the houses in hot and humid climate, the design should promote continues internal air movement and avoid excessive solar gain through the building envelop. To control the solar gain, consideration should be given to specific building elements such as the roof, roof overhangs, walls, doors and windows and how the interact with the external environment. Traditional design principle has shown that good ventilation promote good living condition, natural ventilation should be utilized to the best advantages.

- Housing location: The shortage and arising cost of urban land has discourage developers from building low income housing in or near the cities, although the generally aim is to built houses to accommodate low income families, the main purpose is been defeated by their location been too far from the location where employment is available. This scenario has happened in Bandar Bukit Beruntung in Selangor (near Kuala Lumpur) where many houses remain unsold due to their in accessibility location (Ibrahim, S.H.,2004). One of the main reason, the housing has failed to attract buyers is, the difficulty faced by them when having a commute to their place of work due to the lack of public transportation. Poorly located low income housing project place a huge burden on the house hold economy when the people have pay for transportation. Therefore, the house should be strategically located where the occupants will have proximity to their work places and the degree of traffic is controlled apart from all the necessary facilities.

- Dedicated housing code: The consolidation of all property related cases into dedicated housing court has been effected in several cities. Specialized housing courts have been particularly effective where judges a specially elected or appointed to that court. A specialized court allows matters such as code enforcement to be a priority, rather than falling to the bottom of the judicial docket (Mallach, A.,2006) Judges from this court can be active champions of healthy and safe environment. The court wills handles property related actions, including a special urban rehabilitation docket, demolition and repair of extremely dilapidated house and so fourth.

5. CONCLUSIONS

The proceeding discussion clearly shows that the unsold, overhang and oversupply of housing are the factors contributed to abandonment of housing in Malaysia. To overcome these issues, it requires a profound understanding of numerous internal and external factors and their inter-relationship. Besides looking at the imperfections and failure of the operation of housing market which is commonly argued contributes to that issue, identification of weakness in the land use planning system as clarified by the NEAC, the Ministry of housing and local government and by the Department of Town and Country planning should also need to be explored in detail.

Abandoned buildings can be associated with a wide variety of socio-economic problems. They are magnets for criminal activity, including the consumption and trade of drugs, prostitution, and crime against property. Abandoned buildings cause property values in surrounding areas to decline, and represent a waste of resources that is cruelly ironic in areas plagued by homelessness and a lack of affordable housing. To reduce the rate of housing abandonment in Malaysia element such as an effective demand (population's affordability and household willingness to pay for housing), the housing choice (choice by housing tenure, dwelling type, form of housing and method of new houses to be developed) and the market criteria (condition of local housing market and buyers preference in terms of price, location, type of housing etc) should be incorporated in the overall housing planning process, particularly in the preparation of development plans and at the stage of development control by the local planning authority.

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RECYCLED COARSE AGGREGATES (RCA) AS NATURAL COARSE AGGREGATES REPLACEMENT IN CONCRETE DESIGN; THE BETTER ALTERNATIVE

Kam Kenn Jhun[1], Mohd Zailan Sulieman[2] and Roslan Talib[3]

[1,2,3]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[2]mzailan@usm.my

ABSTRACT: This paper introduces the potential use of recycled coarse aggregates (RCA) as natural coarse aggregates replacement in concrete design. RCA is obtained from the demolition and waste of old building. RCA is suggested to be used as substitution for natural coarse aggregates in new concrete mixture in order to reduce the consumption of natural resources. In the pass research towards RCA suggested that by replacing natural coarse aggregates by RCA can be more environmental friendly and reducing the consumption of natural resources. This paper will be pointed on the further study on the various relative range of RCA replacement in concrete design. In this study, the percentage of replacement is undertaken by specimens 0% (control specimens), 15%, 30%, 60% and 80% by weight. Compressive strength test, flexural strength test, density test, ultrasonic pulse velocity test will be carried according to British Standards.

Keywords: recycled coarse aggregates, natural coarse aggregates replacement, environmental friendly, consumption of natural resources.

1. ENVIRONMENTAL ASPECT

Nowadays, every country in our earth is looking into one of the most popular topic which is “the sustainability development for a better future”. The sustainable construction concept was introduced due to the growing concern about the future of our planet. This is due to construction industry is a massive consumer of natural resources and on the other hand a huge waste producer (Cachim, 2009). On the other hand, the cost of raw materials is increased while the natural resources are keeps on reduction. Thus, waste materials become a potential alternative in the construction industry. This waste material, when properly processed, it will be effective as natural construction materials and readily meet the design specifications standard. (Mannan, Ganapathy, 2004). There are some kinds of study about concrete design to obtain the solutions in this construction industry. By implementing industrial waste such as recycled coarse aggregate (RCA) in concrete design may be contribute some recycling method on these industry wastes. In addition, we can decrease the usage of primary resources on casting concrete (Mehta, 2001). In Germany alone, there are 77millon tons of demolition waste per annum that produced in that country. Approximately 13 million tons estimated values of concretes are demolished in France every year and as well as Japan for a total quantity of concrete debris is in value of 10-15 million tons each year. On the other hand, Hong Kong generates about 20 million tons demolition debris per year and facing serious problem for its disposal solution. Then, United State of America alone is utilizing approximately 2.7 billion tons of aggregate annually out of which 30 to 40% are used in road works and balance in structural concrete work (Singh, 2005). The massive quantities of demolished concrete waste are available at most of the construction sites due to the purpose of redevelopment of the cities. This situation leads to a serious problem of waste concrete disposal in urban areas. But in a conservative way, this can easily be recycled as aggregate and used in new fresh concrete. Research & Development activities have been carried on all over the world to prove its environment sustainability, economic viability, cost effectiveness and feasibility on this construction industry (Oikonomou, 2005).

2. RECYCLED COARSE AGGREGATES

RCA is those aggregates resulting from the processing of inorganic material previously used in concrete building. RCA is obtained from crushing of demolished concrete from the crushing plant.

Most of the waste product from building and demolition wastes can be used to produce recycled coarse aggregates. Nowadays building and demolition wastes can easily obtain due to there are country that rebuild and develop their cities for the purpose of revolution in a new mankind era (Marta & Pilar, 2009) RCA often contain a large amount of attached cements paste and mortar. The old mortar may contain up to 20-30% from the volume. This is mainly depends on the properties of the original concrete and the concrete crush production process. The main difference between RCA and natural coarse aggregates is mainly accounts on the attached mortar and cement paste on the recycled coarse aggregates (Li, 2004).

3. PREVIOUS STUDY ON RECYCLED COARSE AGGREGATES

3.1 TENSILE STRENGTH OF RECYCLED COARSE AGGREGATES

According to figure below, the tensile behavior of concrete is shown with different RCA contents according to direct tensile test which evaluated and tested by Liu et al. (2005), Xiao and Lan (2006). This figure shows that, as the RCA content in concrete design increases, the tensile strength will decreases.

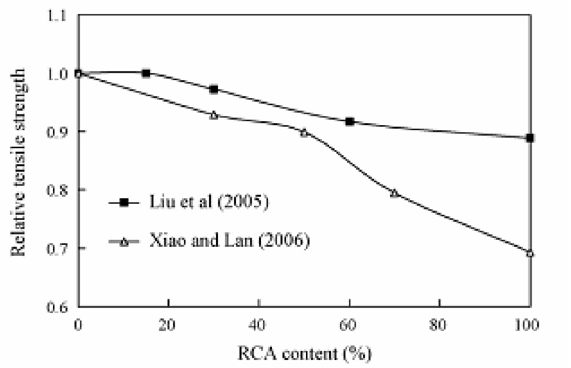

Figure 1. *Influence of RCA content on the concrete uniaxial tensile strength (Li, 2008)*
(Sourcr: Liu et al, 2005)

The tensile strengths can only be reached at 69% and 88% of the control concrete for the concrete that contain 100% of RCA content. According to the research, they also found that if the percentage of RCA contents when is not exceed 20%, the influence of RCA on a concrete design can be neglected as Figure 1. (Li, 2008).

3.2 COMPRESSIVE STRENGTH OF RECYCLED COARSE AGGREGATES

On the other hand, the compressive strength of the concrete will decrease as the RCA increase in the mixture. Figure 2 below show the decrease of compressive strength when RCA increased in a concrete design (Li, 2008).

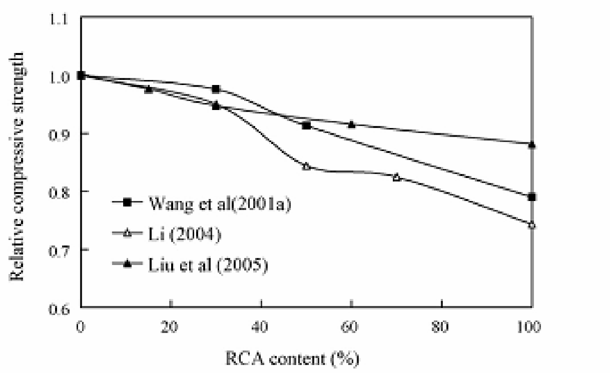

Figure 2. *Influence of RCA content on the concrete compressive strength (Li, 2008)*
(Source: Li, 2008)

3.3 DENSITY OF RECYCLED COARSE AGGREGATES

The density of dry surface recycled aggregate concrete (RAC) is in the range of 2340 kg/m³ with coarse aggregates size (4-8mm) and 2490 kg/m³ with coarse aggregate size (16-32mm) (Hansen & Narud, 1983).
According to another researcher, he pointed out the density of waste concrete is 2510 kg/m³ with coarse aggregates size (15-30mm) when the outer surface is dried (Turanh, 1993). While for ordinary concretes, these values are between 2500 kg/m3 with coarse aggregates size (4-8mm) and 2610 kg/m3 with coarse aggregates size between (16-32mm) (Hansen & Narud, 1983).

4. RESEARCH DEVELOPMENTS

4.1 COMPRESSIVE STRENGTH TEST

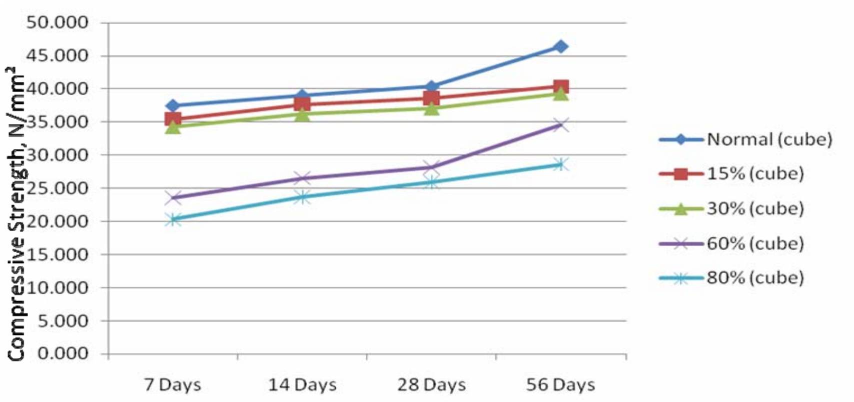

Figure 3. *Influence of RCA content on the concrete compressive strength (Source: Li, 2008)*

According to Figure 3 above, for the most basic understanding, the compressive strength of the concrete is directly proportional to the curing time. The longer the age of the concrete, the higher compressive strength of the concrete will be achieved. Among all the specimens, the normal concrete achieved the highest strength. Then, followed by 15%, 30%, 60% and 80% modified concretes. This is due to RCA contain attached mortar that attached to the RCA and the strength of attached mortars is very weak. Thus, this will affect the entire compressive strength of the RCA replacement concrete. The relationship between compressive strength and the percentage of RCA content in concrete is in inversely proportional in direction. The higher the RCA contents in a concrete, the lower compressive strength that can be achieved by the concrete.

4.2 FLEXURAL STRENGTH TEST

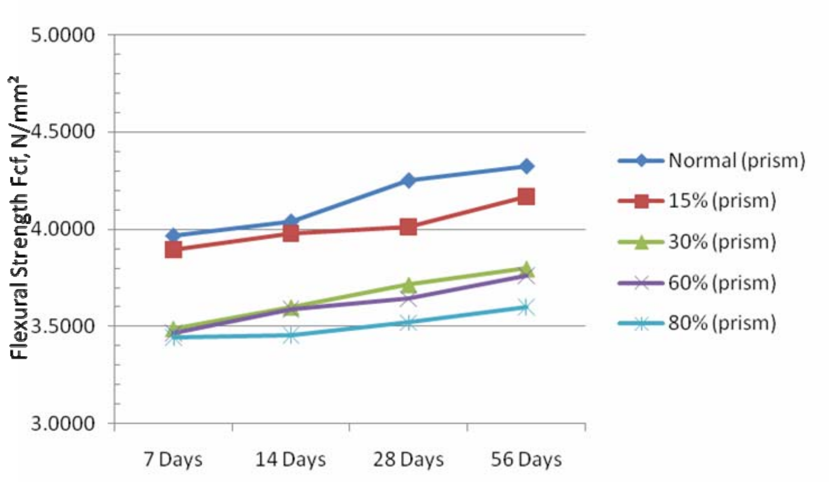

Figure 4. *Graph of flexural strength test*

The results in figure 4 show that the flexural strength at all specimens increases with the increasing age at curing. The normal specimen shows an increase from 3.967 N/mm2 at about 7 days to 4.322 N/mm2 at the age at 56 days. The modified 15% concrete shows the highest flexural strength compare to others modified specimens at 3.896 N/mm2 at the 7 days age to 4.167 N/mm2 at the age of 56 days.
The flexural strength of the concrete is decreasing while the percentage of RCA replacement in the concrete increase. Normal concrete achieved highest flexural strength reading which is and the flexural strength of the concrete decrease in percentage of RCA replacement. On the other hand, 80% RCA replacement concrete achieved the lowest flexural strength. The relationship between flexural strength and RCA content is inversely proportional. The higher quantity of RCA replacement in concrete design, the lower flexural strength will be achieved.

4.3 DENSITY TEST

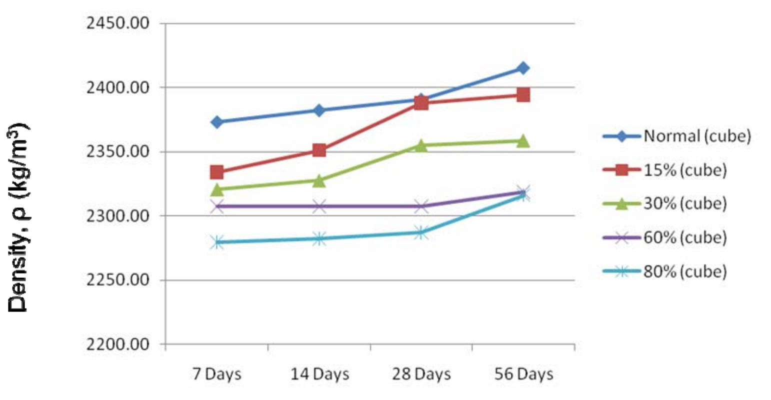

***Figure 5**. Density of cube specimens*

The relationship between density and the age of concrete gives a directly proportional result. The results in figure 5 show that the density at all the specimens increases with the increasing age of curing. The increased in density of concrete when the age of concrete increased is due to the internal harden of concrete increased in time. The particle inside the concrete is hydrated in time while the void of the concrete will decrease. The concrete will become more and more compacted and the strength of the concrete will be increase in time as well. On the other hand, the density of the concrete achieved decreased when the percentage of RCA replacement in concrete increased. The highest density along the entire 7 days to 56 days curing age achieved which is normal concrete and followed by 15% modified concrete, 30% modified concrete, 60% modified concrete and the lowest 80% modified concrete. The relationship between density and percentages of RCA replacement in concrete design gives an inversely proportional result. In conclusion, we can conclude that the density of the concrete will affect the strength of concrete. While the higher the concrete density achieved, the higher the strength of the concrete harden.

4.4 ULTRASONIC PULSE VELOCITY TEST

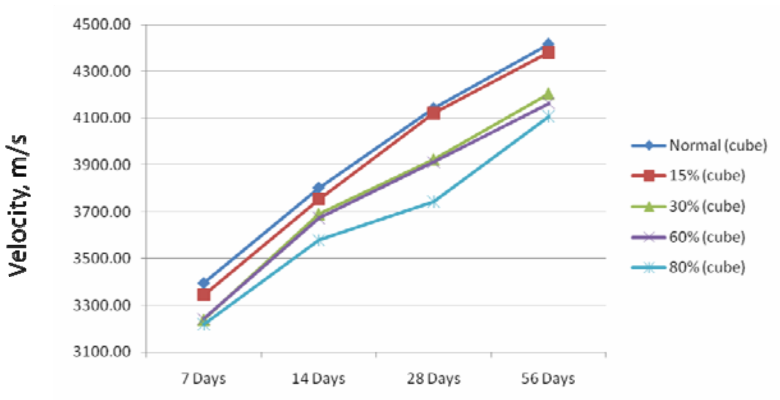

***Figure 6.** Graph of ultrasonic pulse velocity test*

According to figure 6 above, we can notice that the velocity of the ultrasonic pulse velocity test is in the range of 3100 m/s to 4500 m/s. The readings between the specimens are inversely proportional to the velocity achieved. The higher the RCA replacement in the concrete design, the lower velocity would be achieved. This is due to the RCA attached mortars that contain voids that affected the velocity of the test.
When looking into the relationship between the velocity of the ultrasonic pulse velocity test towards the age of the concrete, we can clearly see that the velocity in directly proportional to the age of the concrete. The higher the age of the concrete, the velocity achieved will be higher. This is due to the internal particles of the concrete takes time to bind within each others. The hydration of the concrete takes time to achieve fully hydrated. The longer the time of hydrating process, the concrete will achieve stronger strength. When the binder between the particles of the concrete is fully harden and there is lesser void in the concrete, the velocity from the ultrasonic pulse velocity test will be achieve higher reading. The higher the velocity of the concrete reading, the stronger strength will be achieved by the concrete.

Table 1. *Suggested pulse velocity ratings for concrete*
Source: Malhotra 1976

Pulse Velocity		
ft per sec	**m/s**	**General condition**
Above 15000	4575	Excellent
12000 to 15000	3660-4575	Good
10000 to 12000	3050-3660	Questionable
7000 to 10000	2135-3050	Poor
Below 7000	2135	Very poor

According to table 1, we can know the rating for our concrete specimens. During the age of 56 days, all the concrete specimens achieved velocity on the range of 4100 m/s to 4400 m/s. The lowest reading 4108.33 m/s achieved which is modified 80% concrete and the highest reading 4418.33 m/s achieved by normal concrete. As a result, we can conclude that the general conditions on the specimens are good in rating.

As a results, ultrasonic pulse velocity test can be use to understand the characteristic of the internal particles of concrete for predicting the quality of the concrete, by understanding the relationship between velocity achieved toward the characteristic of internal particles of concrete.

5. CONCLUSIONS

The characteristic of RCA in concrete mixing design is determined and understand in this paper. The percentage of RCA replacement in concrete design will affect the physical property and concrete characteristic of the concrete. The main concern is the strength of the concrete, the percentage of RCA replacement in concrete is inversely proportional to the strength of the concrete. The higher the percentage of RCA replacement in concrete design, the lower the strength achieved.

As a conclusion, the knowledge on characteristic of the RAC is very important, for our future construction development. If RAC successfully implemented in a concrete design accordingly; it can fulfill our future environmental green and sustainable development concept which is towards the conservation of our natural resources.

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MANAGING RISK CONSTRAINTS OF MULTIPLE DESIGN & BUILD PROJECTS

S.P. Narayanan[1], Arazi B. Idrus[2] and CT. Ramanathan[3]

[1,2]Universiti Technologi PETRONAS, Perak, Malaysia.

[3]Kumpulan Liziz Sdn Bhd, 33-11-1, 1 Lorong Ruang Grace Square, Grace Square, 88000 Kota Kinabalu, Sabah, Malaysia.

[1]narayanan_sambu@petronas.com.my , [2]arazi_idrus@petronas.com.my , [3]ramoo_ctr@yahoo.com

ABSTRACT: The Design and Build (D&B) procurement method has gained popularity in recent times in the construction industry due to its attractive financial aspects. The development projects are implemented by D&B for its special nature, crucial time periods and size. The contractors have lack of knowledge on the other major constraints in this procurement method. Many of these D&B projects had ended in the hands of contractors who were unable to complete the projects within agreed time and price. Even some of those who completed the projects had committed to begin implementation without carrying any analysis of the risks involved. They were basically committed by judgement and inadequate experience. The risk is more while committing to multiple simultaneous projects. Even though there were studies conducted for risk management of projects and studies on contractors' view of D&B projects, so far no studies were conducted on the time and cost risks of D&B projects. This is also extended further to study on multiple projects. The aim of this study is to identify elements, procedures to analyse and overcome the risks of the important components of time and cost in management of multiple D&B bridge projects. The final outcome of this research will be of practical use for the industry.

Keywords: **Design and Build, Multiple projects, Project Management, Time and cost risk**

1. INTRODUCTION

Due to increased competition in construction industry, companies procure many projects which have to be managed simultaneously. The execution of multiple projects requires high quality project plans and controls. This also holds true for multiple D&B projects where the design also had to be done by the company. Not only is on-time delivery important, it translates directly into whether the contractor will meet the client requirement, quality and provide a return on investment. Delivering a project on time does not occur by hoping that the required completion date will be met. Majority of D&B projects encounter events and/or changes that affect the original plan of executing a project. Further, resources such as labour, material, and equipment may be scarce, in high demand and as a result may hamper project execution. Attempting to solve these unforeseen issues during a project without a plan in place to determine the immediate impact is a major risk which can often lead to delayed projects and disputes between the parties (Galloway, 2006). The situation is more complicated for projects having differing starting and completion date. Multi-project management is not simply the management of a list of individual projects, rather a complex operation with a given capacity and workload. Sharing common human resources and construction machinery effectively with no idle time need proper planning. The prerequisites of planning and control for one project are impaired by another project.

2. Background of Design and Build (D&B)

Types of contract procurement used in Malaysia.

Generally construction projects are procured in Malaysia through Traditional contract, Design and build contract, Management contract, Construction management contract, hybrid and miscellaneous contract (Jatarona, 2007).

1. Traditional contract – The client has separate contractual undertaking with both Design Consultant and Contractor/builder.
2. Design and build contract - Known as 'package deal' or 'turnkey contract' undertake both designing and constructing works. The milestones of D&B contractor's performance are shown in Figure1.
3. Management contract – Role to manage, coordinate and supervise the work.
4. Construction management – Provide managerial and supervisory service for the project.
5. Hybrid
    a. Develop and construct – concept design by independent professional.
    b. Design and Manage – contractor responsible to manage design process.
    c. Design and Construction Management
6. Miscellaneous Contracts
    a. Build, Operate and Transfer Contract – Privately financed, operated and transferred to the employer after concession period.
    b. Serial Contract – more than one project in accordance with the tender submitted for the initial project and in series.
    c. Continuation contract – Scope of the original contract extended beyond the contract domain.
    d. Periodic contract – execution of works is required in intervals.
    e. Partnering contract – Receives all contracts from employer payment in reference to initial agreed formula.
    f. Independent contract – undertaking stipulated tasks for an agreed consideration.

Design and build is an arrangement where contracting organization takes sole responsibility, normally on a lump sum fixed price basis, for the bespoke design and construction of a client's project (Masterman, 1992). Design and build is a procurement method where one entity or consortium is contractually responsible for both the design and construction of a project (Murdoch & Hughes, 2000). In contract to the lump sum method, design and build has two concurrent phases. The clients enter into one contract with a design and build company that is responsible for project development and construction. The contractual arrangement is the simplest of the other contractual arrangement like management contracting (MC) and construction management (CM). There is one contract and one line of communication for the client. The designer-builder may be a single company that employs both the design and construction staff (known as D&B) or the design-builder may be a partnership of two or more design and construction companies (known as joint venture D&B). The objectives of this study are two-folds: to identify Time and Cost risks associated with design and build method and to make recommendation in order to mitigate the risks (Adnan, H et.al. 2008).

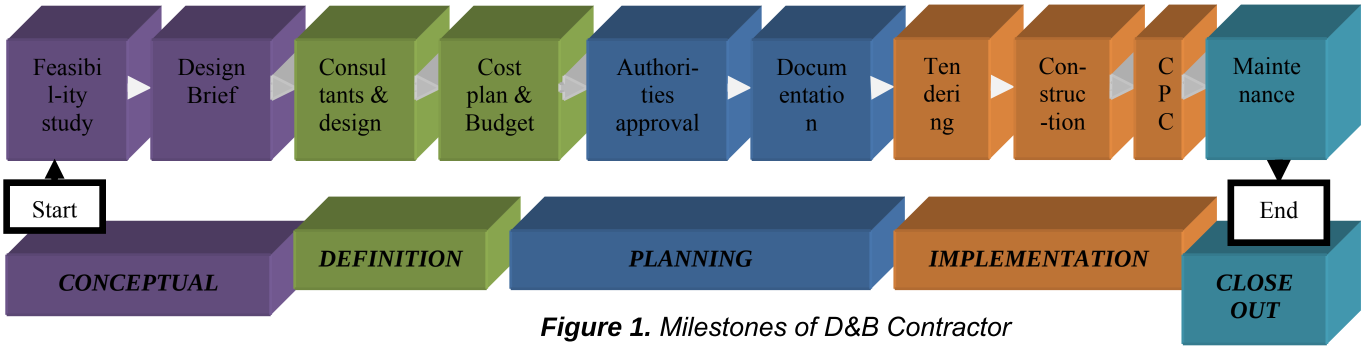

Figure 1. *Milestones of D&B Contractor*

3. PROBLEM STATEMENT AND MOTIVATION FOR STUDY

Problem Statement

A Project is successful when it is completed on time, within budget, specified quality and to stakeholders satisfaction. Unfortunately, due to many reasons, high project performance and project success are not commonplace in the construction industry, especially in developing countries. Therefore, professionals and scholars have been motivated to take extensive efforts to meet this challenge.

- There is no additional time or cost given for D&B projects which has fixed contract sum and contract period. Many D&B projects are carried out without proper checking of risks involved and are ending up in variation orders which are not considered for contracts procured in this method. This is because so far there are no proper guidelines or standards set because there were no studies conducted on these aspects. Because of which there are many disputes between consultants and contractors or clients and contractors. This is the problem that this research intends to solve by conducting research in the Malaysian perspective.
- Time delays, cost overruns and change orders are generally due to factors such as design errors, unexpected site conditions, increase in project scope, weather conditions and other project changes. A study to analyse the uncertainties of cost overrun and time delay problem is required to address the problem with appropriate recommendations.
- There are evidences that contractors who are able to perform and deliver their earlier projects successfully are not producing consistence results. The recorded failures of the same contractor to deliver similar upcoming projects by facing time and cost overrun, is an alarm to the industry for both client as well as contractor. This is due to the gap in lack of studies related to time and cost risk in project management.
- It is concluded that a separate study is required for identifying and analyzing the factors of delay causers for projects operated in Sabah, East Malaysia.

Scope of Research:

To create risk modelling assessment to evaluate a project's risk factors before proposed project commencement by performing statistical modelling of uncertainty related to time and cost through questionnaire surveys. Record and monitor the real time project performance in Sabah. Comparing the qualitative and quantitative results from the responses (of questionnaire response with the practical case study project's observation) to analyse and prepare a useful Risk Response Plan. Analyse also extended to derive results of influential factors causing time and cost uncertainties in multiple projects.

The Objective of the study

- Study on the management of known uncertainties of time and cost in Multiple D&B bridge projects.
- To identify the risk of time and cost factors for multiple D&B projects from the view point of employer, consultants and contractors through questionnaire.
- To study and identify the recurring time and cost factors influencing the project case study from the data collected in the ongoing multiple bridge projects.
- Risk modelling with comparative analysis of case study data with the questionnaire survey of respondents on the risk of time on cost in multiple bridge projects.

4. LITERATURE REVIEW

Michael Latham – 1994 the report to this government sponsored examination that the construction industry in the United Kingdom was inherently ineffiencient and excessively costly. The report identified many of the key failings of the traditional approach to procuring the managing contracts such as: the emphasis on lowest price rather than value for money; an adversarial culture throughout the supply chain; and the lack of a client focus. It made detailed recommendation including: the need to bring together client, design consultants, contractors and subcontractors as an integrated team; a partnership approach to working with an emphasis on teamwork, openness and continuous improvement; and a move away from awarding tenders solely on the basis of lowest price. It emphasised the role of the client in bringing about the necessary changes within the industry and recommended that Government commit to becoming a best practice client. The report estimated that if the recommended improvements were implemented there was potential to achieve efficiency saving of 30% of construction costs over five years.

Sir Peter Levene – 1995 subsequent to the Latham report, the Government commissioned this Efficiency Unit Scrutiny to examine its performance as a construction client. The report concluded that the performance of departments was poor and that this was contributing in a significant way to the poor performance of the industry. It criticised: poor communication with contractors; lack of understanding of risk and how to manage it; unrealistically optimistic budgets and timetables; and over simplistic view of competition with emphasis on long tender lists and initial price rather than quantity

and long term costs. The report made detailed recommendations to improve performance by: encouraging team working and partnering; improved forms of contract; training for staff; and improvement of best practice guidance.
Sir John Egan – 1998 despite implementation of many of the recommendations of Latham and Levene, progress in improving performance on construction projects was perceived to be slow. A Construction Task Force was set up to advise the Deputy Prime Minster from the client's perspective on the opportunities to improve the efficiency and quality of delivery of construction, to reinforce the impetus for change and to make industry more responsive to customer needs. The Task Force produced the report Rethinking Construction which indentified five "drivers" which needed to be in place to secure improvement; four key processes which had to be significantly enhanced; and seven quantified targets for the level of improvement to be achieved.

Despite involvement of UK government from Higher level to study to overcome the cost and time risk posed to the project implementation, it has difficulties in improving or to follow the recommendations made in the reports since 1994. Hence it is highly required to have a practical study in Malaysia to manage the risk of time and cost in projects.

5. RESEARCH METHODOLOGY

The Research involves multiple infrastructure projects involving construction of Bridges undertaken in Design and Build concept.

Real time project case study:

Forty Five bridges have been constructed in this concept of D&B and delivered to JKR. The data of risks involved in time and costs are collected and analysis of the causes will be carried out. Further, structured questionnaire related to these parameters of time and cost will be given to clients, consultants and contracts handling projects of similar nature. Their personal feedbacks on the previous projects data are compared and solutions for the risks are derived. The results of the studies will be applied and tested in the upcoming project of 12 bridges (duration of 20 months) for the confirmation of the study results. Any correction after practical checking will be reported with final conclusions on management of risk on time and cost of multiple D&B bridge projects in Malaysia.
Questionnaire survey:
This Research Study is focused on the paramount important constraints of Time and Cost effects in project management in multi-project setup. The different activities for the questionnaire survey are listed below. The questionnaires are distributed to PWD, Project owners, Design Consultants, General Contractors. The responses are compiled. The questionnaires covers 6 aspects of Time Management, 3 aspects of Cost management and 6 aspects of risk modelling, listed under sections C, D and E. The integrated study will be conducted on Time risks and Cost risks with the following aspects as established in PMBOK of PMI.

Time Management

- Define Activities - Planning Process
- Sequence Activities - Planning Process
- Estimate Activity resources - Planning Process
- Estimate Activity duration - Planning Process
- Develop schedule - Planning Process
- Control Schedule - Monitoring and Controlling Process

Cost Management

- Estimating cost - Planning Process
- Determine Budget - Planning Process
- Control Costs - Monitoring and Controlling Process

Risk Modeling

- Plan Risk Management - Planning Process
- Identify Risks - Planning Process
- Perform Qualitative Risk Analysis - Planning Process
- Perform Quantitative Risk Analysis - Planning Process

- Plan Risk Response - Planning Process
- Monitor and Control Risk. - Monitoring and Controlling Process

The questionnaire was divided into three parts. The first part requested background information about the respondents. The second part of the questionnaire focused on time and cost management. The respondents were asked to highlight their recommendations to improve the performance of Malaysian construction industry through an open-ended question at the end of second part of the questionnaire. Third part of the questionnaire focused on the risk factors influencing time and cost in Malaysian construction industry. A five-point Likert scale ranging from 1 (not important) to 5 (extremely important) was adopted to capture the risks of time and cost in multiple design and build bridge projects.

- The case study has been used for practical examples to identify Known unknowns and Unknown unknowns.
- Plan for risk responses and methods of planning and handling risks are being studied on Negative Risks as Threats and Positive Risks as Opportunities.

6. REAL LIFE PROJECT - A Case Study

Completed Multiple Bridge Projects

As D&B is the fastest way to begin and implement various development projects across the country (Malaysia), the case study Multiple D&B Bridge projects was also proposed and implemented successfully in Sabah. The research on this case study has highlighted the complexity while implementation and presented Project Management studies used to solve problems from beginning of the life cycle of this projects. Overview of this multiple project setup in this case study is shown in Table 1.

Table 1. *Overview of Multiple Projects completed in Sabah*

Project Number	No. of Bridges	No. of district Locations	Project Duration	
			Months	Period
1	12	3	18	Jul 03 – Jan 05
2	5	3	18	Jan 05 – Jul 06
3	8	3	18	Jul 05 – Jan 07
4	13	3	20	Oct 05 – Jun 07
5	7	3	20	Oct 05 – Jun 07
Total	**45**	**12**	**48**	**Jul 03 – Jun 07**

Presently ongoing Multiple Bridge Projects
The details of Multiple Bridge projects currently in progress are shown in table 2:

Table 2. *Overview of Multiple Projects currently ongoing in Sabah (Project VI & VIII)*

Project Number	No. of Bridges	No. of district Locations	Project Duration	
			Months	Period
1	8	4	20	March 09 – Nov 10
2	4	2	20	July 09 – Feb 11
Total	**12**	**5**	**23**	**March 09 – Feb 11**

7. SUMMARY

- The research has currently reached to the stage of completing Literature review and finalised Research Methodology to achieve the research objectives.
- The risk modelling in the aspect of Time delays and Cost overruns factors are being identified.
- Data collection patterns for the ongoing projects are in progress.

- Simultaneously the data compilation from the completed real time multiple bridge projects (as stated in the case study) are in progress.
- Questionnaire are prepared and scheduled to distribute to clients, consultants, contractors and various experts in the field to have comprehensive data for analysis.
- Before starting the real survey a pilot study is planned to obtain field experts' opinion on the questionnaire items.

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FULLY UTILISATION OF FINE BOTTOM ASH AGGREGATE (FBAA) AS REPLACEMENT AGGREGATE IN CONCRETE BLOCK

Mohd Syahrul Hisyam bin Mohd Sani[1], Fadhluhartini bt Muftah[2], Zulkifli Muda[3]

[1,2,3]Faculty of Civil Engineering, Universiti Teknologi MARA Pahang, Malaysia

[1]msyahrul210@pahang.uitm.edu.my, [2]fadhlu@pahang.uitm.edu.my

ABSTRACT: This paper presents the use of bottom ash as fine aggregate in producing concrete block. The fine bottom ash aggregate (FBAA) is a waste material that is taken from electric power plant. FBAA is by product of electric generator by coal material. The objective of the study is to investigate the feasibility and potential use of FBAA in concrete and concrete application especially block. It is essential to use waste material due to a scarce of natural material, shortage area for dump, high cost and environmental problem. FBAA used to replace the fine aggregate is passed through 5 mm sieve. The chemical, physical and mechanical properties of FBAA were discussed. Chemical composition of FBAA was determined by using X-Ray Fluorescence (XRF) and physical properties of FBAA was analysed. Different concrete mixes that depend on different proportion of FBAA with constant water cement ratio were prepared. Mechanical results indicate that all proportion percentage replacement by FBAA is illustrated lower compare to control mix. The compressive strength of concrete with 80% FBAA replacement by weight of natural sand is found to be an optimum usage in concrete. Therefore, 80% FBAA replacement can be used as an alternative of natural fine aggregate in concrete block production.

Keywords: Bottom ash, fine Aggregate, concrete, properties

1. INTRODUCTION

A main source of Fine Bottom Ash Aggregate (FBAA) is a waste product of coal electric power plant. Waste product known as bottom ash is estimated to increase in the develop country such as China, Korea, Malaysia and other countries where landfill for dumped is limited. In the Peninsula Malaysia, there are 4 famous coal electric power plants that use a coal (coal-fired) as a power material for generates electricity. There are at Perak, Johor, Selangor and Negeri Sembilan. TNB's electric power plant in Perak by using coal to the burning began operations in September 2002 with a capacity of 2100 MW (3 X 700 MW). This coal was taken directly from Sarawak or import high quality coal from Indonesia, Australia, the United States, Canada and also China. TNB will use 1.5 million tons of coal each year. There are two types of waste product (fly ash and bottom ash) were produced. To control waste product hovering in the atmosphere, recipients electrostatics was built and used for trapping 99% of ash and dust. While another waste product that disposed after process of electicity and collected from the bottom of furnaces that burn coal or the bottom of combustion chamber. This coal bottom ash is physically coarse, porous, glassy, granular, greyish and incombustible materials. The type of bottom ash produced depends on the type of furnace and also the sources of coal. From the burning process of coal, 80% of product will become fly ash and remain 20% of product is bottom ash.

Bottom ash is collected at the bottom of the combustion chamber in a water-filled hopper, is removed by means of high-pressure water jets and conveyed by sluiceways to a decanting basin for dewatering followed by stockpiling and possibly crushing (Steam, 1978). Figure 1 show the typical steam generating system that illustrated the bottom ash dispose at the bottom furnace and fly ash is dispose to atmosphere by very tall chimney. Table 1 presents the physical properties of bottom ash.

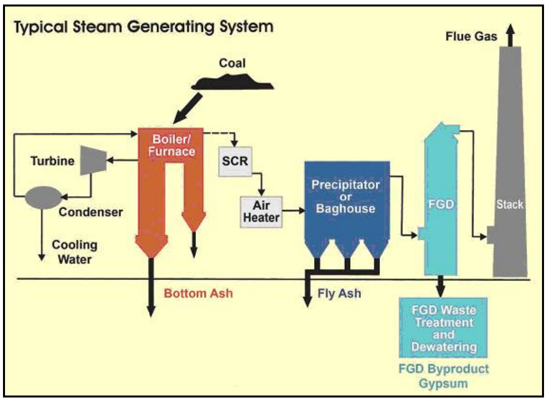


Figure 1. *The production of coal combustion byproducts in steam generating system (NETL, 2006).*

Table 1. *The typical physical properties of bottom ash (Majizadeh et al., 1979)*

Property	Bottom Ash
Specific Gravity	2.1 -2.7
Dry Unit Weight	7.07 - 15.72 kN/m^3 (45 - 100 lb/ft^3)
Plasticity	None
Absorption	0.8 - 2.0%

The problem to find additional dumped sites is also becoming more difficult due to acute shortage of land as the pace of development. Apart from that, conventional construction material depends on natural resources, which are now becoming scarce and give a lot of environmental effect (landslide and water pollution). The use of bottom ash as filler and aggregate of suitable size in concretes would save a million tons per year of natural materials and would reduce landfill-related problems (Ferraris et al, 2009). In order to overcome both of these problems, there is a need to provide viable solution by reducing the use of natural resources and by reusing waste material. From the observation of physical properties and physical appearance, bottom ash can be replaced or substituted in concrete mix. The physical shape and geometry of bottom ash had similar to fine aggregate or sand. Besides that with a lot of problem of waste product to environment and due to shortage of fine aggregate material, bottom ash could be a great material to replace of sand.

According to 2006 statistics, 45% bottom ash is used in transportation applications such as Asphalt Concrete Aggregate, road base material, Embankment or Backfill Material and structural fill (American Coal Ash Association (ACAA, 2006). Hjelmar et al (2006) said that bottom ash or municipal solid waste incinerator (MSWI) bottom ash in Denmark is utilisation for back-filling (landfill) and as sub-based in road construction purposes. Besides that, bottom ash is also used as fine aggregate in producing lightweight concrete masonry and as cement replacement in structural and masonry purpose (Saikia et al, 2008), (Jaturapitakkul and Cheerarot, 2003). Many investigation found that the bottom ash has some cementaneous properties in which may increasing the strength and long-term than concrete with natural sand. Siddique (2003) carried out strength properties test and indicate significant improvement in the strength of concrete by the addition of class F fly ash as partial replacement and can be effectively used in structural concrete. The physical properties of fine aggregate that carried out as shown in Table 2. Figure 2 illustrates the common applications of coal bottom ash. The strength and drying shrinkage of concretes with furnace bottom ash (FBA) as sand replacement at 0%, 30%, 50%,70% and 100% by mass, were studied at fixed water–cement ratios (W/C) and fixed slump ranges (Bai et al, 2005). The results showed that, at fixed water–cement ratios, the compressive strength and the drying shrinkage decreased with the increase of the FBA sand content.

Table 2. *Physical properties of aggregates (Siddique, 2003)*

Property	Fine aggregate	Coarse aggregate
Specific gravity (SG)	2.63	2.61
Fineness modulus	2.25	6.61
SSD absorption (%)	0.86	1.12
Void (%)	36.2	39.6
Unit weight (kg/m3)	1690	1615

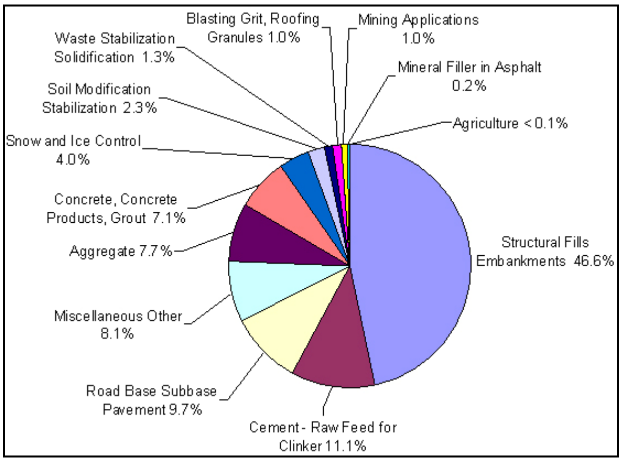


Figure 2. *Bottom ash applications as a percentage of totals reused. (ACAA, 2006)*

Besides that, many researchers have explored for utilizing bottom ash in special concrete application such as roller compacted concrete and self-compacting concrete. Ghafoori et al. (1997) carried out investigations on a series of laboratory-made roller compacted concretes (RCC) containing high-calcium dry bottom ash as a fine aggregate. Kasemchaisiri et al. (2008) presented the test results of mechanical properties of self-compacting concrete (SCC) incorporating bottom ash as partial sand replacement of 10%, 20% and 30% by weight. 10% replacement by weight of total fine aggregate showed a better durability, chloride penetration, carbonation depth and drying shrinkage compare to control SCC mix.

Concrete block is normally utilised as a building material in the construction as a wall or retaining units. A concrete block is form in solid or hollow shape and their surface cast in smooth condition. The concrete raw materials commonly used to produce concrete blocks is a mixture of cement, water, fine aggregate (sand) and coarse aggregate (gravel). Evaluation of compressive strength of processed bottom ash for use as lightweight aggregate in the production of concrete masonry units (8” x 8” x 16”) has done by Phillips et al (2005). Concrete with incinerator ash were show no gain in the compressive strength and the leaching of elements from ash block sample did not show significant variation in the pH of the contact water (D’Souza & Shrihari, 2007). This study explores the possibility and prospect of fully replacing of fine bottom ash aggregate (FBAA) as a way of incorporating significant amount of FBAA. Next, this paper also reports on the properties of hardened concrete with bottom ash as substitute for natural sand.

2. MATERIAL AND EXPERIMENTAL WORK

Bottom ash used was collected from the bottom of furnaces of TNB Electric power plant (Coal Power Station) in Perak. The bottom ash is then dried at normal temperature for 24 hours and sieved around 5 mm and below 5mm as to be a FBAA. This FBAA is of Q – DEJ + 10% Lati Coal type. Q – DEJ and Lati are noted as the origin of the coal located in Indonesia. Ash content in Q – DEJ and Lati was 10 – 13 % and 4% respectively. The Q – DEJ contain high fly ash and bottom ash while Lati contain low fly ash and high bottom ash. This BA has 9.76 pH concentrations. The discharging method of bottom ash was by circulating water and it is free from salt and chloride.

After 24 hours, the FBAA were taken and sieve analysis is then carried out to get the particle size distribution accordance to BS 812-103.1: 1985. The FBAA was also applied as partially sand replacement by weight in concrete mix to produce cube concrete for crushing test accordance to BS 1881: Part 116: 1983 . The FBAA was substitute into concrete with its natural water to cement content of 0.55. The Portland cement used was a single source of Portland cement. Concreting activity also utilized a 20 mm coarse aggregate of granite origin and clean tap water free from material deleterious to concrete.

3. CHEMICAL AND PHYSICAL PROPERTIES OF WASHED BOTTOM ASH

3.1 Chemical Properties

Chemical composition of the FBAA was determined by using by XRF method on 500g coal. It was results as tabulated in Table 3. In general the FBAA is composed of 39.4% and 34.3% of silica and alumina respectively. The lost of ignition of 0.63. The chemical properties are almost same as Ordinary Portland Cement (OPC) hence it is suitable used in concrete.

Table 3. *Chemical properties of bottom ash and OPC*

Chemical composition	Bottom ash Weight percent%	Cement* (OPC)
Silica, SiO_2	39.4	20.6
Alumina, Al_2O_3	34.3	6.3
Iron Oxide, Fe_2O_3	15	3.6
Titania, TiO_2	3.08	-
Magnesia, MgO	1.57	-
Calcium Oxide, CaO	6.13	63
LOI at 1000ºC	0.63	-

* (Bai, Ibrahim and Basheer, 2010)

3.2 Sieve Analysis

Aggregate grading is very important in relation to the plastic properties and might effects the properties in harden state. Physical properties analysis conducted for FBAA sample was sieve analysed and fineness modulus was determined. FBAA used to replace the fine aggregate is passed through 5 mm sieve. Sieve analyses of the natural sand and FBAA sample were carried out as per British Standard (BS 410: 2000). The sample was initially sieved through 5mm in order to identify the fine particles in the bottom ash. The sieve grading is done by dry sieving using in-house method and the results are tabulated in Table 4.

Table 4. *Sieve analysis of FBAA*

BS sieve size	Mass (kg)	Cumulative Mass retained (kg)	Cumulative percentage retained (%)	Cumulative percentage passing (%)
10 mm	0	0	0	100
5.00mm	0.5096	0.5096	12.56	87.44
3.35mm	0.4942	1.0038	24.74	75.26
2.00mm	0.4645	1.4683	36.18	63.82
1.18mm	0.4278	1.8961	46.72	53.28
600µm	0.4095	2.3056	56.81	43.19
425µm	0.3696	2.6752	65.92	34.08
300µm	0.3669	3.0421	74.96	25.04
212µm	0.3438	3.3859	83.43	16.57
150µm	0.3391	3.7250	91.79	8.21
63µm	0.3272	4.0522	99.85	0.15
0	0.0060	4.0582	-	-
Total	4.0582	-	493.12	-

Table 5. *Grading limit for Fine Aggregate (Derived from BS 882)*

BS sieve size	Percentage by weight passing BS sieve (%)							
	Overall limit	Additional limits			Zone 1	Zone 2	Zone 3	Zone 4
		C	M	F				
10 mm	100	-	-	-	100	100	100	100
5 mm	89-100	-	-	-	90-100	90 – 100	90-100	95-100
2.36 mm	60-100	60-100	65-100	80-100	60-95	75-100	85-100	95-100
1.18 mm	30-100	30-90	45-100	70-100	30-70	55-90	75-100	90-100
600 µm	15-100	15-54	25-80	55-100	15-34	35-59	60-79	80-100
300 µm	5-70	5-40	5-48	5-70	5-20	8-30	12-40	15-50
150 µm	0	-	-	-	0-10	0-10	0-10	0-15

Aggregate grading zone 2 and 3 is often described as concreting sand which is derived from BS 882: 1992. Table 5 show the grading limit for fine aggregates for overall limit, additional limits and grading zone 1 to zone 4. The bottom ash sand is not fully compliance to grading any limit specified four grading zones.

Nevertheless the FBAA samples are almost complied with grading zone 1 and zone 2 of the fine aggregate as shown in Figure 3 and Figure 4. The FBAA is not compliance only for sieve size of 212 µm up to 600 µm for specified grading Zone 1. This non-compliance was mainly due to the large percentage of fine particle in FBAA and may be considered as too fine. In contrast, the bottom ash sample is not compliance to the large particle of specified grading Zone 2 for sieve size of 1.18 mm up to 5 mm. It was happened due to small percentage of large particles in FBAA samples and indicates that the sample is too coarse. The higher the zone number indicates a finer material. Therefore, the FBAA is considered as coarse to medium sand which is almost applicable in any concrete application that specified to Zone 1 and Zone 2.

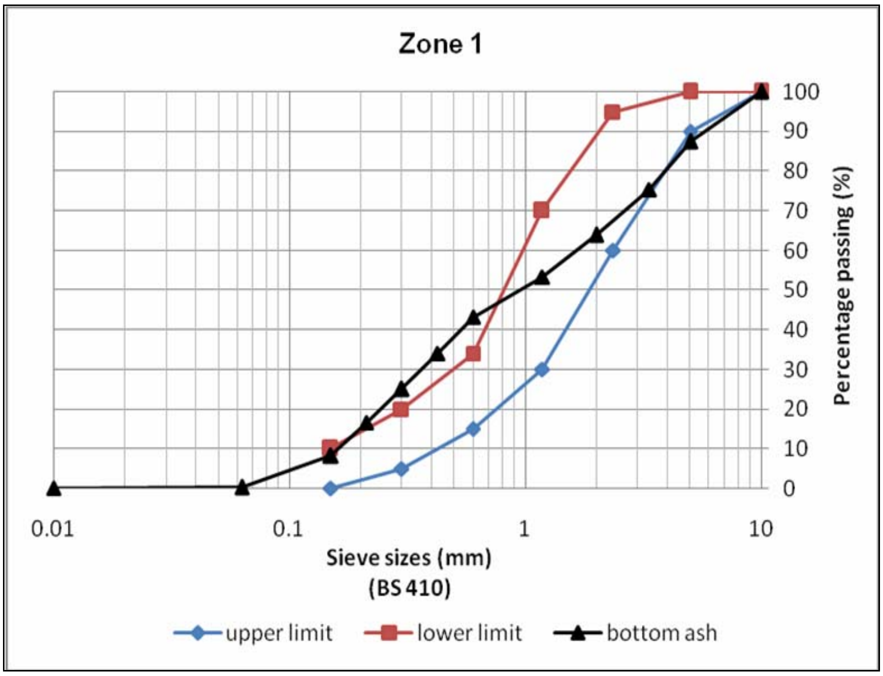


Figure 3. *Grading curve of FBAA for Zone 1*

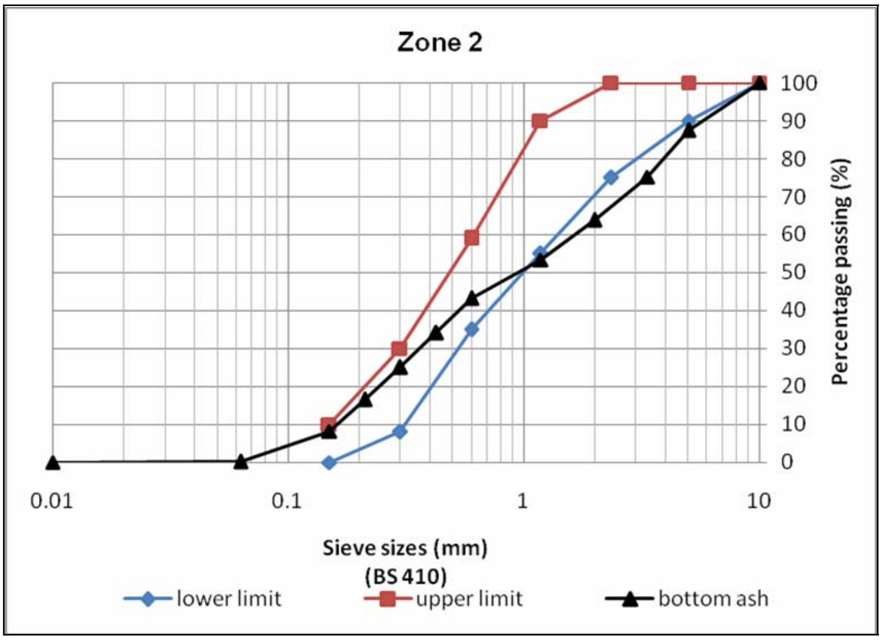


Figure 4. *Grading curve of FBAA for Zone 2*

In Figure 5, the gradation curve of FBAA sample clearly shows that the percentage passing are distributed more towards the lower limit and the percentage passing for 5 mm size is almost approach to the lower limits hence considered as coarser sample. However the FBAA sample was fully comply with overall limits as given in BS 882 (1992). Therefore the FBAA is suitable for general concrete application. Considering the additional limits for grading sand, the FBAA is also fully comply with the grading C and M except F as shown in Figure 6. Therefore the FBAA is applicable for heavy duty concrete floor finishes as stated in BS 882 (1992). It was clearly shows that the process of limiting the size of bottom ash to maximum of 5 mm is more applicable to mortar rather that concreting sand.

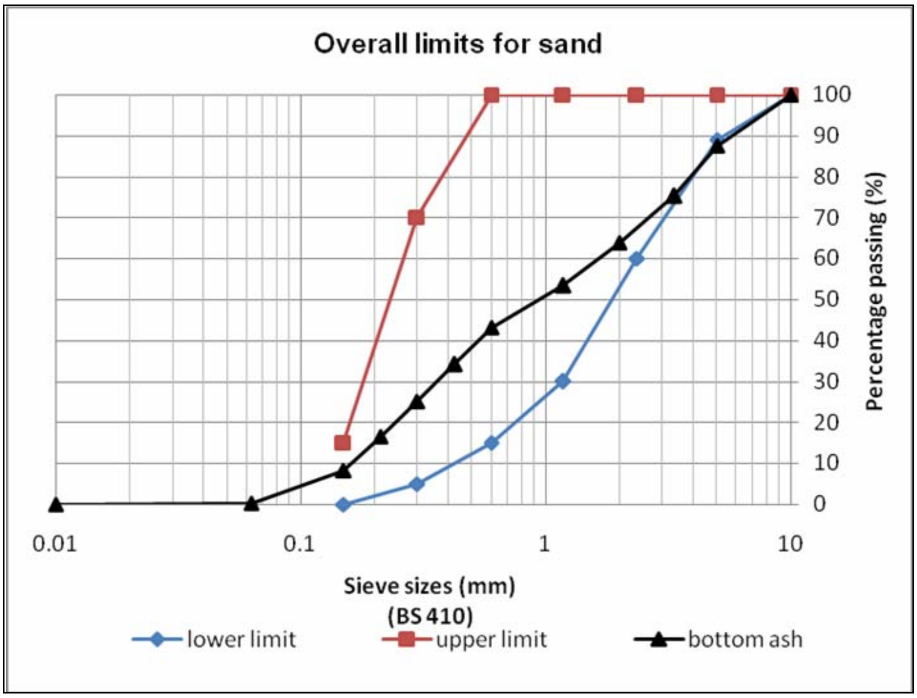
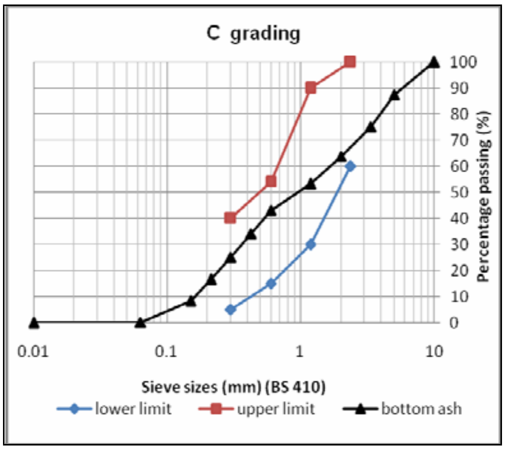


Figure 5. *Grading curve of FBAA for Overall zone*



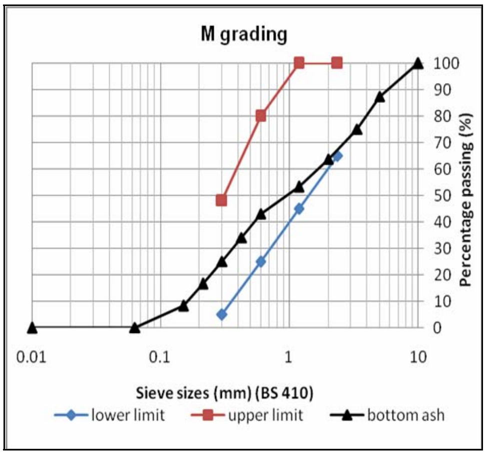


Figure 6. *Grading curve of FBAA for C and M grading*

The results of sieve analysis of FBAA samples tabulated in Table 4 directly obtained the fineness modules of the FBAA. The fineness modulus is the summation of the cumulative percentage retained on the sieve standard series of 150, 300 and 600 μm, 1.18, 2.36, 5.0 mm up to the larges sieve size used. The calculated of fineness modulus of FBAA is 4.93 which are more than 3.5 and it is considered very coarse. The categorization given in BS 822:1992 based on percentage passing the 600μm sieve which is 43.19%. Therefore, the bottom ash sand is considered as course to medium sand (Alexander & Mindess, 2005).

4. SPECIFIC GRAVITY AND WATER ABSORPTION

Specific gravity of FBAA was determined by in-house method and results an average specific gravity is 1.84 while natural sand has higher specific gravity of 2.66. It was shows that the FBAA is 30% lighter than natural sand and may producing lighter concrete. However the FBAA has higher water absorption of 26.6% while natural sand is only 1.3%. Higher water absorption indicates that the bottom ash has high porosity. The pores at the surface particle of bottom ash affect the bond between the aggregate and cement paste and may reducing the strength of concrete. Besides that, FBAA has water absorption higher than 2-3%, therefore it may need to be treated as suspect and may influence to concrete performance such as higher dry shrinkage (Alexander & Mindess, 2005).

5. MECHANICAL PROPERTIES OF CONCRETE

5.1 Mixtures and dosage

Five types of concrete were produced with the substitution of FBAA by weight into natural sand in a various percentage of 60%, 70%, 80%, 90% and 100% and noted as M60, M70, M80, M90, and M100 respectively. Control mix concrete using 100% natural sand was produced for comparing its relative strength. Material for control mix as follows:

Free water = 210 kg/m^3
Cement = 403.85 kg/m^3
Sand = 686.34 kg/m^3
Gravel = 1119.81 kg/m^3

The compositions of concrete and FBAA content as sand replacement are shown in Table 6.

***Table 6**. Mix detail of FBAA concrete*

Concrete Mix	Cement (kg)	FBAA (kg)	Natural Sand (kg)	Gravel (kg)	Water (kg)
Control mix	28.62	0	48.64	79.37	14.88
M60	28.62	29.18	19.46	79.37	14.88
M70	28.62	34.05	14.59	79.37	14.88
M80	28.62	38.91	9.73	79.37	14.88
M90	28.62	43.78	4.86	79.37	14.88
M100	28.62	48.64	0	79.37	14.88

5.2 Sample Preparation and Testing

The study of FBAA as sand replacement involves mechanical properties analysis i.e. compressive strength evaluation. The compressive strengths were obtained for all concrete at 3, 7, 28, and 60 days of curing with three samples per age of testing to obtain the average value. Standard cube specimen sizes of 150 mm x 150 mm x 150 mm were used in confirmative with BS 1881-108:1983. The sample were exposed to constant temperature (24 ± 2 ºC) and constant relative humidity (95 ± 5 ºC) environment inside curing tank until the tested ages. At 24 hours after the concrete casting, the moulds were removing and the samples were kept into the curing tank until tested day. The weights of samples were recorded to measure amount of water absorption by capillary and determining the mass difference during the time interval of 1 day to 28 days.

6. FRESH CONCRETE TESTING

6.1 Slump test

***Table 7.** The value of slump and description of workability*

Concrete Mix	Slump (cm)	Description of workability
Control Mix	6 cm	Medium
M60	2 cm	Low
M70	3 cm	Low
M80	5 cm	Medium
M90	8 cm	High
M100	10 cm	High

6.2 Compacting Factor test

***Table 8.** The value of compacting factor and description of workability*

Concrete Mix	Compacting factor (%)	Description of Workability
Control Mix	98.50%	High
M60	90.74%	Medium
M70	91.84%	Medium
M80	87.26%	Low
M90	88.45%	Low
M100	96.54%	High

6.3 Slump flow test

Table 9. *The value of slump diameter*

Concrete Mix	Slump diameter (cm)
Control Mix	35cm x 33cm
M60	35cm x 36cm
M70	37cm x 38 cm
M80	40cm x 43cm
M90	42cm x 46cm
M100	58cm x 57cm

Table 7, 8 and 9 were explained the relation between five concrete mixes with the workability of the FBAA mixes. From the result in Table 7, show that the M100 and M90 gave high condition of workability when compare with control mix. This is because of the physical of FBAA show its porous and low density. While in compacting factor test, FBAA with 100% replacement also show the high workability. Besides that, compacting factor of M100 illustrate low surround of air void in mix and achieve to meet control mix percentage. M80 show the high containing the air void in concrete around 87%. Table 9 prove the M100 are high workability and more flowable compare with other percentage of replacement. High value of workability was not recorded high strength value.

7. COMPRESSIVE STRENGTH OF FBAA CONCRETE

The compressive strength of concrete mixes made with various percentage of FBAA as sand replacement inclusive of control mix (fully natural sand) was determined at 3, 7, 28, and 60 days of curing. It can be seen that the compressive strength of concrete mixes of sand replacement is lower than control sample at all tested day. The summary test results for all concrete are provided in Table 10 and presented in Figure 7. Generally, all concrete with FBAA replacement has decrease in strength until long term duration i.e. at 60 days. M100 results in increasing in compressive strength at 7 day; however the strength is not achieving the target of control mix. The used of FBAA has revealed lower strength concrete compared to control sample strength. From the Figure 7, show that all FBAA mixes far from the control mix average compressive strength. It was found that the compressive strength at day 3 for M60 and M80 is the highest compared to other FBAA concretes. Starting at 28 days, M80 recorded the highest strength until age 60 days of 20.33N/mm^2. For 3 days and 7 days, M60 determined the highest strength compare with other FBAA mixes.

Table 6. *Compressive strength of concrete with FBAA*

Curing	3 Days	7 Days	28 Days	60 Days
MIX	Average Compressive strength (N/mm^2)			
0% (control mix)	28.07	34.21	39.52	43.52
M60	12.98	15.20	16.53	18.56
M70	11.14	13.22	16.23	18.82
M80	12.23	13.93	18.06	20.33
M90	10.63	12.82	15.76	18.77
M100	7.04	9.45	9.96	13.11

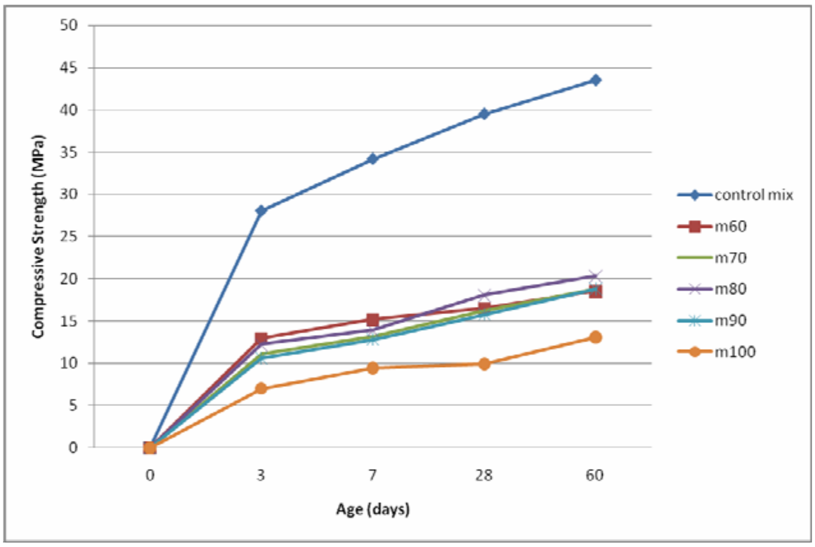


Figure 7. *Behaviour of concrete strength with FBAA*

Therefore, it can be concluded that 80% of FBAA as sand replacement in concrete is the optimum amount in order to get favourable strength to producing concrete block. Other FBAA mixes recorded lower strength and lastly not suitable to use in concrete application.

8. CONCLUSIONS

The following conclusion may be drawn from this study:

1. Sieve analysis conducted on FBAA from Electric Power Station, TNB Perak is more suitable used for mortar and brick masonry rather than concreting sand and concrete block.
2. FBAA of 100% and 90% sand replacement by weight is not suitable for concrete because it has produced a lower strength concrete at the early ages which can results in ruptures during construction.
3. 80% FBAA replacement is found to be the optimum amount in order to get a favourable strength and good strength development pattern over the increment ages.

The following recommendation may be use in the future study:

1. Change the mix design by using volume in calculation of FBAA amount in concrete.
2. Study of the self-compacting concrete (SCC) by using FBAA. This is because the FBAA in porous condition and ready to meet requirement of workability and flowablity.

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STRENGTH AND PERMEABILITY OF FIBRE REINFORCED POOR FINES HIGH STRENGTH CONCRETE

Kwan Wai[1] Hoe and Mahyuddin Ramli[2]

[1, 2]University Sains Malaysia, Pulau Pinang, Malaysia

[1]kwanwaihoe@hotmail.com , [2]mahyudin@usm.my

ABSTRACT: Fibre has remarkable effects in overcoming the brittleness of the high strength concrete (HSC) especially when silica fume is incorporated as partial cement replacement for high strength achievement. However, inconsistent and confusing of the past results diminish the confidence level of the user in the construction industry. The properties of the fibre and its sensitivity in the variations of contents towards its performance must be identified. This study explored the influences of newly engineered Barchip fibre (BF) in poor fines HSC, for three important parameters, which are permeability, compressive and flexural strength. In addition, the performance was also compared with HSC reinforced by coconut fibre (CF) and glass fibre (GF). Statistical analysis (Boxplot and line plot of mean) was employed to study the achievements of the specimens and indentified the sensitivity in variations of fibre content. Fibre has significantly increased the flexural strength of HSC than compressive strength. However, the permeability of HSC was affected by incorporation by short discrete fibres. The BF specimen has achieved the highest compressive and flexure strength, while intrinsic permeability was as good as GF specimens, slightly higher than normal. CF series were not sensitive in variations of fibre content in compressive strength parameter. The same situation happened to BF and GF in permeability parameter. In short, BF was appeared to be the best choice to incorporate into HSC for better performance.

Key words: high strength concrete (HSC), poor fines, barchip fibres, coconut fibres, glass fibres, permeability, boxplot, line plot of mean

1. INTRODUCTION

Fibre Reinforced Concrete (FRC) is the concrete mixed in the conventional way which containing short discrete and randomly distributed in the mixture. Typically, these fibres have a well-known effect in improving structural performance and integrity, increasing ductility, shear strength, energy absorption capacity and damage tolerance in flexural and shear-critical members under reversed cyclic loading. (Parra-Montesinos, 2005) (Ko¨ksal, Altun, Yig˘it, & Sahin, 2008). A study by Ramli and Dawood also came out with the same conclusions as other researcher whereby both fibre and hybrid fibre system has brought encouraging effects to the performance of concrete. (Ramli & Dawood, 2010; Dawood & Ramli, 2010) Apart of the numerable advantages has brought by fibres, there is also some risk to introduce fibres into concrete. The universal finding was that the workablilty of fresh concrete greatly affected by incorporation of fibres. (ACI Committee 309, 2008). Especially when high volume of fibre has been mixed into the concrete, the significant level of reducing effects in workability has substantially increased the risk of air entrapped in the concrete and hence affecting its rigidity and followed by durability. (Tatnall, 2006). However, each types of fibre and its composition gives different effects to the performance of FRC. Besides that, even though there are tons of researches have been done in this field, but the data recorded were inconsistent and therefore diminishing the confidence of potential user in the construction industry. For instance, the influence of fibre on permeability remains as the most arguable parameter. A study by Toutanji, McNeil, and Bayasi (1998) concluded that polypropelyne fibre resulted increases in permeability of conventional concrete. However, Bhargava (2006) found that fibres showed positive effects in controlling permeability of FRC. Under both stressed and unstressed conditions, the FRC showed the similar behaviour in reducing the permeability of concrete. The increase dosage of fibre in FRC can also improve the durability of concrete as well. In this study, the idea is to explore on the influences newly engineered fibre (Barchip fibre) on the strength and permeability properties of poor fine high strength concrete (HSC) and also compare its performance with the well studied fibre which is coconut fibre and glass fibre.

2. MATERIALS

Type I Portland cement and condensed silica fume used as partial replacement to the binder content during the experiment. The chemical compositions of both cementations materials were showed in Table 1. River sand with specific gravity 2.51 and fineness modulus 3.98 used as fine aggregate. The undersize particles (below 600 micron) were eliminated. Crushed granite with specific gravity 2.7 and nominal size 19 mm used as coarse aggregate. Conplast SP 1000, a chloride free super plasticizing admixture based on sulphonated naphthalene polymers which complying BS 5075 used to established a workable fibre reinforced high strength concrete. Coconut fiber, Barchip fiber and Glass fiber with specifications are shown in Table 2 to Table 4 respectively. Mixture proportions of all thirteen mixes was showed Table 5.

***Table 1.** Chemical composition of ordinary Portland cement and Silica fume*

Percentage by weight		
Constituent	Ordinary portland cement	Silica fume
Lime (CaO)	64.64	1.0% (max)
Silica (SiO_2)	21.28	90% (max)
Alumina (Al_2O_3)	5.60	1.2 % (max)
Iron Oxide (Fe_2O_3)	3.36	2.0% (max)
Magnesia (MgO)	2.06	0.6% (max)
Sulphur trioxode (SO_3)	2.14	0.5% (max)
N_2O	0.05	0.8% (max)
Loss of ignition	0.64	6% (max)
Lime saturation factor	0.92	-
C_3S	52.82	-
C_2S	21.45	-
C_3A	9.16	-
C_4AF	10.20	-

***Table 2.** Specifications of coconut fibre*

Item	Unit	Test value
Diameter	mm	0.32
Length	mm	20-30
Tensile strength	mpa	176
Elastic modulus	GPa	22.4
Specific gravity	-	1.13

***Table 3.** Specifications of Barchip fibre provided by Elastro Plastic Concrete Inc*

Characteristic	Unit	Material property
Base resin	-	Modified Olefin
Length	mm	24±2
Tensile strength	MPa	640
Surface texture	-	Continuously embossed
No. fibers per kg	Nos	37, 000
Specific gravity	-	0.95
Young's modulus	GPa	10
Melting point	°C	159-179
Ignition point	°C	>450

Table 4. *Specifications of alkaline resistance glass fibre provided by Berjaya Bintang Timur (M) Sdn. Bhd*

Item	Unit	Standard	Test value
Diameter	μm	15.0±2.3	15.70
Length	mm	24±1.5	24.40
Moisture content	%	≤ 0.2	0.18
Combustible matter content	%	1.9÷0.3	1.83
Elastic modulus	GPa	71.7-80.0	72.00
Specific gravity	-	2.6-2.8	2.70
ZrO^2	%	16.8±0.5/0.3	16.57
TiO^2	%	5.5±0.5	5.45

Table 5. *Mixture proportions of control mix*

Index	Cement (kg/m3)	Silica fume (kg/m3)	Water (kg/m3)	SP (kg/m3)	Sand (kg/m3)	Coarse aggregate (kg/m3)	Fibre by binder volume (%)
Normal	435.6	59.4	183	9.9	689	1033	0.0
0.6BF	435.6	59.4	183	9.9	689	1033	0.6
1.2BF	435.6	59.4	183	9.9	689	1033	1.2
1.8BF	435.6	59.4	183	9.9	689	1033	1.8
2.4BF	435.6	59.4	183	9.9	689	1033	2.4
0.6CF	435.6	59.4	183	9.9	689	1033	0.6
1.2CF	435.6	59.4	183	9.9	689	1033	1.2
1.8CF	435.6	59.4	183	9.9	689	1033	1.8
2.4CF	435.6	59.4	183	9.9	689	1033	2.4
0.6GF	435.6	59.4	183	9.9	689	1033	0.6
1.2GF	435.6	59.4	183	9.9	689	1033	1.2
1.8GF	435.6	59.4	183	9.9	689	1033	1.8
2.4GF	435.6	59.4	183	9.9	689	1033	2.4

3. EXPERIMENTAL PROGRAM

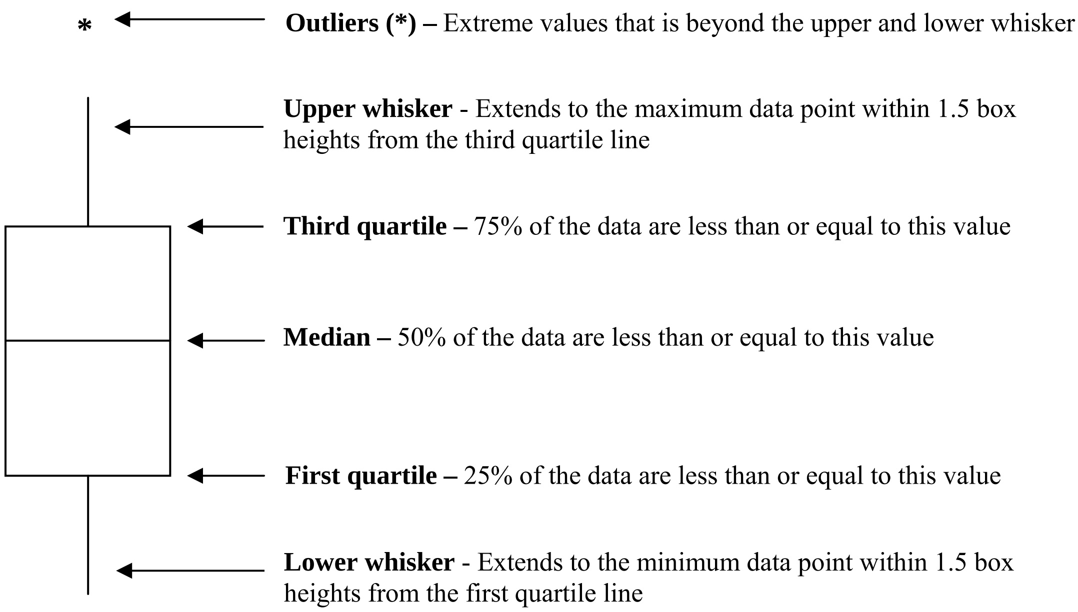
Thirteen mixes were prepared with the mixture compositions as shown in Table 5. The fresh specimens were kept in the room conditions ± 20 °C and relative humidity at ±65%. Then, the specimens were demoulded after 24 hours and subjected to water curing at temperature 23±2°C until the age of testing. For each kind of specimens, three 100 x 100 x 100 mm cubes and 100 x 100 x 500 mm prisms were prepared for compressive and flexural strength test. Both tests were implemented in accordance to BS EN 12390-3:2009 and BS EN 12390-5:2009 respectively. The permeability test was achieved by the samples cored from prisms right after flexural strength test, using Leeds cell permeameter. The procedures are as follows (Cabrera & Lynsdale, 1988):-

- Six cylinders with dimension of 45 mm Ø x 40 mm were cored from three different prisms of each mixes and put into ventilated oven at temperature 105 ± 5 °C for 72 ± 2 hours. All cylinders were then kept in desiccators for another 24 ± 2 hours.
- The cylinder was took out from desiccators and housed in the silicone rubber located inside a stainless steel ring cylinder. Then, it capped tightly with a stainless steel cap. Any leakage was strictly prohibited in order to force the nitrogen gas travel through the sample in uniaxial direction only.
- The system was left for 15 minutes to achieve steady state prior to data recording.
- Travel period of the bubble in the flow meter was recorded at five consecutive points, each at 10 cm distance. These data was used to calculate the flow rate.
- The intrinsic permeability was determined by the expression below:-

$$K = \frac{(2P2 \quad VL \; x \; 1.76 \; x \; 10^{-16})}{A\,(P1^2 - P2^2)} \qquad (1)$$

Where,
K is intrinsic permeability, m^2
P_1 is absolute applied pressure bars [pressure used + atmosphere pressure] usually 2 bars
P_2 is pressure at which the flow rate is measured [atmosphere pressure] usually 1 bar
A is cross section areas of specimen, m^2
L is thickness of specimen, m
V is the flow rate, cm^3 / sec

Followed by the testing, the experimental results were interpreted in Boxplot summarize the distributions of the samples and variability and line of mean plot to illustrate the mean score of the samples. Below is the structure of boxplot (Minitab Inc., 2007):-



4. RESULTS AND DISCUSSIONS

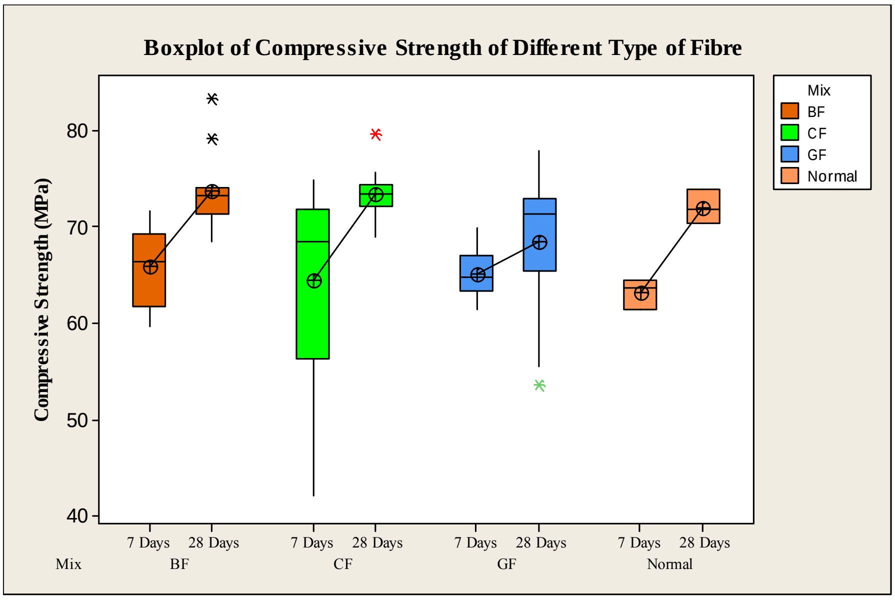


Figure 1. *Boxplot of compressive strength of different type of fibre at 7 and 28 days*

In a single boxplot, it consisted of fibres at four different contents, which were 0.6%, 1.2%, 1.8% and 2.4% while the two respective boxplots at two different ages (7 and 28 days) were connected by the mean line. From this statistical interpretation, the HSC reinforced by BF and CF have obtained higher compressive strength than the plain HSC at both ages. Except for specimens with GF, they obtained lower average strength compared to normal HSC at 28 days. This was because the GF specimens were only showed enhancement in strength at high fibre content, i.e. 2.4% of fibre. It was clearly understood from line plot of mean at different fibre content which presented in the following section. In BF series specimens, there was two outliers were found in the boxplot at 28 days. The highest outlier was represented the compressive strength result of the cube at 1.8% of BF content, obtained 83.2 MPa. While for the CF series specimens, the outlier was showed by the cube at 0.6% of CF content, obtained 79.7 MPa. The dispersion of compressive strength at different fibre content could be understood by the length of the boxplot. Therefore, it could be concluded that the CF series specimens have the least sensitivity in varying the fibre content towards the achievement of compressive strength at 28 days, followed by BF series and GF series specimens.

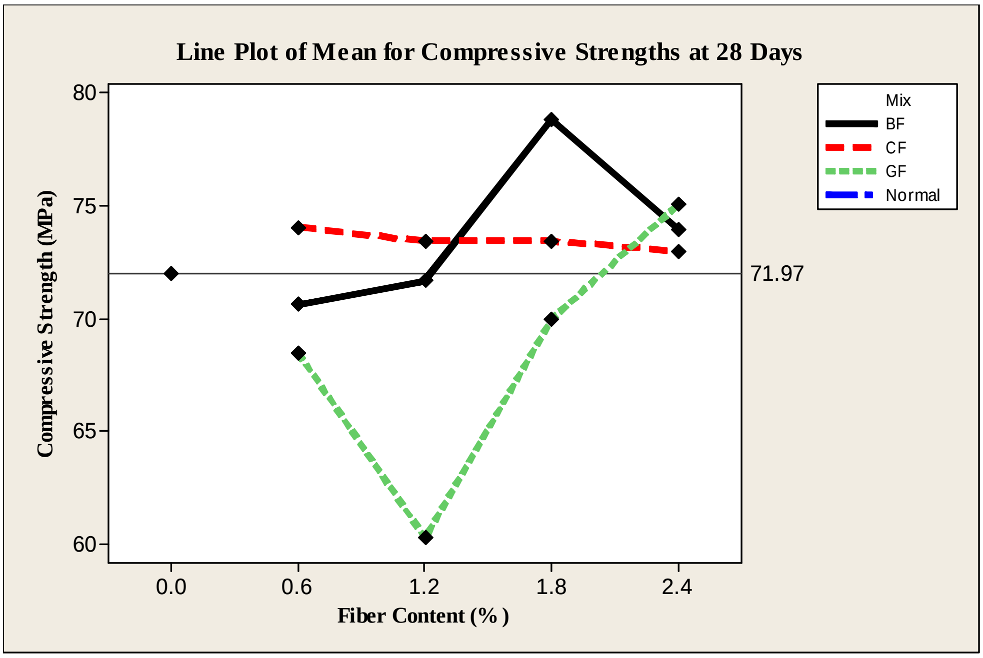


***Figure 2.** Line plot of mean for compressive strength of various series of specimens at respective fibre content at 28 days*

As subsequence from the data interpretation and discussions above, Figure 2 showed the mean compressive strengths attained by HSC specimens reinforced by three different types of fibres with different content at 28 days. Plain HSC had showed compressive strength of 71.97 MPa, plotted as reference line for comparisons. The highest average compressive strength of 79.07 MPa was registered by HSC reinforced by 1.8% of Barchip fibre. The high quality of the fibre with very high tensile strength (640 MPa) enhanced the toughness of the concrete matrix. In addition, the fibre was also designed specifically to have contour surface in order to optimize the reinforcing effects in the concrete matrices. The CF series showed improvement in compressive strength in all level of fibre content. However, it was in a decreasing trend when the fibre content increases. This was contradicted with GF series specimens. Although compressive strength decreases from 0.6GF to 1.2GF, however it showed an increasing trend as the fibre content increases. Short discrete fibres are very effective in arresting cracks. They would reduce the formation and propagations of cracks under axial loading as well as plastic shrinkage during hydration process. The properties of the fibres such as embossed treatment to have contour surface and high tensile strength as in Barchip fibre and high specific area of reinforcing posses by monofilament glass fibre, rough surface on the tough coconut fibre govern the bonding forces at the interfacial zone between fibre and concrete matrices and the bridging effects across the cracks during loading stage. All of this resulted higher efficiency of load transfer within the microstructure under loading and therefore contribute to the enhancements of strength achievement as proven by the results. (Sivakumar & Snathanam, 2007; Topcu and Canbaz, 2007; Brandt, 2008).

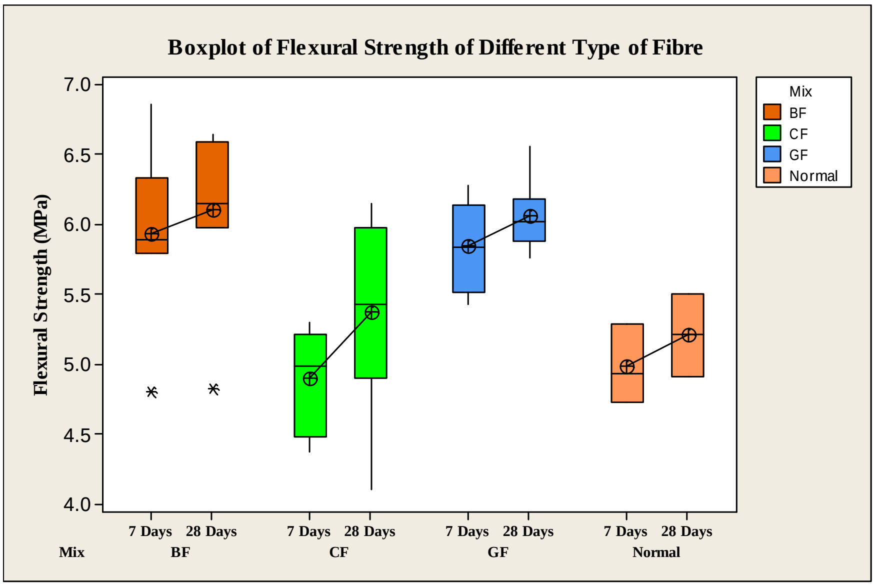


***Figure 3.** Boxplot of flexural strength of different type of fibre at 7 and 28 days*

Figure 3 presented that the HSC containing fibres exhibited higher flexural strength compared to that of plain HSC. Both BF series and GF series showed comparable results in flexural strength, but still the BF series attained higher mean flexural strength. In addition, the specimens from both series had already attained higher flexural strength in 7 days than normal HSC at the age of 28 days. The dispersion of the boxplot showed GF series has less sensitivity in the variation of fibre content after received water curing of 28 days. The statistical results of CF series indicated that coconut fibre seem did not help much in enhancing the flexural strength of HSC. However, since they were relatively sensitive in the variation of fibre content with respect to flexural strength, therefore the judgement of the degree of influencing have to refer to the following figure. This is because there was still a marginally improve in flexural strength at optimum fibre content.

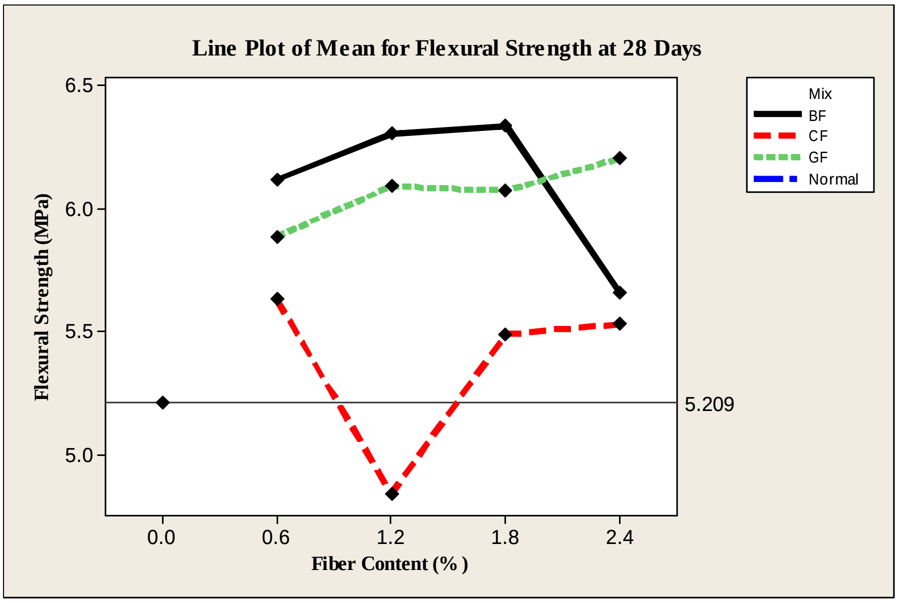


***Figure 4.** Line plot of mean for flexural strength of various series of specimens at respective fibre content at 28 days*

The normal specimen attained mean flexural strength of 5.209 MPa was plotted as reference line for comparison purpose. Except the 1.2CF specimens, all other specimens have obtained higher flexural strength than normal. Among them, BF series has the highest capability in this way, up to 1.8% of fibre, after that the performance was decrease drastically. GF series showed an increasing trend towards the high fibre content. On the contrary, the CF series showed the results in the other way round. The flexural strength was decreases as the fibre content increases. It was also found that there is a sharp drop to below reference line at 1.2% of fibre content. Fibre has very negative effects in altering rheological behaviour of fresh mix. The frictions exist between fibres and paste increase the cohesiveness of the mix and therefore result an unworkable mix. (Tatnall, 2006) In this situation, the uniformity of the mix was affected and zone of weakness might probably formed, which resulted lower flexural strength. However, the reinforcing effects arise from the fibre is far greater than this. The reinforcing mechanisms could be elaborated in this way. When the specimens subjected to flexure load, cracking would initially happened at the weak zone and the bridging effects of the fibres across the cracks starts working to slow down the crack propagation process. If the fibre is low in tensile strength (coconut fibre and glass fibre), the fibre would ruptured and lead to ultimate failure of the specimens. While in BF series specimens, fibre was being pull-out instead of ruptured due to its high tensile strength properties. During the pull-out process, mechanical effects of the fibre also play an important role. The good bonding between the Barchip fibres and concrete matrices created large frictions when they were being pulled-out and hence further increased the peak load. Therefore, BF series could attain the highest flexural strength among other series specimens. (Reis, 2006; Hamoush, Abu-Lebdeh, Cummins and Brian Zornig, 2010).

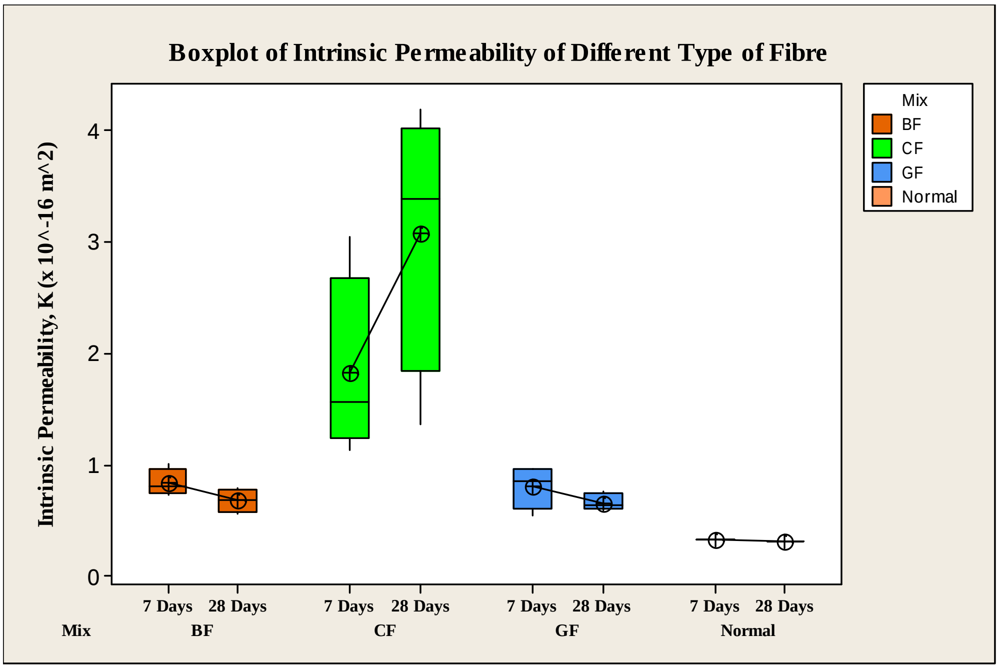


Figure 5. *Boxplot of intrinsic permeability of different type of fibre at 7 and 28 days*

The results presented in Figure 5 illustrated that all of the fibre reinforced specimens attained higher permeability at both 7 and 28 days. The result obtained by BF series and GF series was comparable, and shared the similar profile with normal specimens where the intrinsic permeability was decreases over the age. This is because the micro structure of the concrete was refined by the calcium silicate hydrate (C-S-H) formed during the hydration process and the reaction between silica fume and calcium hydroxide. (Caldarone, 2009). However, CF series specimens displayed the performance in opposite way. The intrinsic permeability increases over the age. In order for this situation to be happened, it was believed that the coconut fibre was degraded in the concrete as coconut fibres are alkaline sensitive. The fibres would complete lost its strength when immersed in alkaline solutions for 300 days. Furthermore, the migration of hydration products especially calcium hydroxide from matrix to fibres would induce microcracks around the fibres and caused fibre embrittlement. (Filhoa, Scrivenerb, Englandc, & Ghavami, 2000). All of these was happened over the age and increased the chances for gas molecules to pass through the concrete specimens. Besides, coconut fibre has a porous cellular structure. The nitrogen gas was also believed to be passed through the fibre itself during the testing. Hence, the intrinsic permeability of CF series was relatively high compared to other specimens. Permeability of coconut fibre reinforced HSC was also sensitive in variation of fibre content.

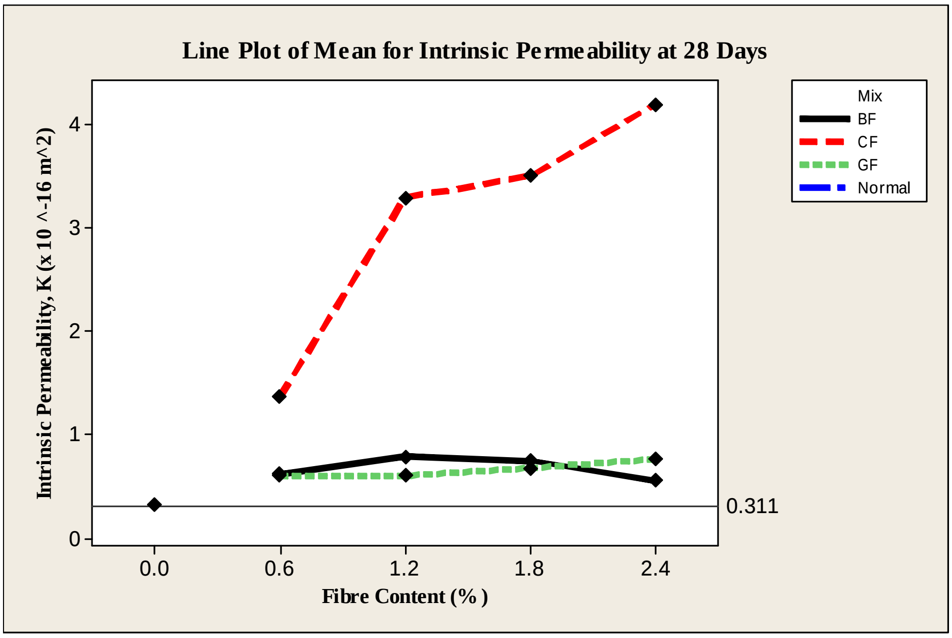


Figure 6. *Line plot of mean for intrinsic permeability of various series of specimens at respective fibre content at 28 days*

As consequence to the above elaborations, the line plot of mean for CF series showed that the permeability of CF specimens increases drastically as the fibre content increases. BF series and GF series displayed a quite consistent result. Variation of fibre content did not result significant changes in permeability properties of the concrete. The BF series was even showed a slightly decreasing trend on permeability as the fibre content increases. Nevertheless, all of the curves was still been plotted above the reference line which represented the intrinsic permeability of normal specimen. This implied that the permeability of fibre concrete is higher compared to plain concrete. This was partly due to the fibre has high potential than conventional concrete in entrapping excessive air content into the mixes as fibre may arise difficulties which would cause incomplete consolidation. (Tatnall, 2006).

5. CONCLUSIONS

From this study, several findings and key results are summarized as follows:

- Barchip fibre is the best among the fibres that have been studied in this research. It registered the highest compressive and flexural strength for the fibre reinforced HSC and showed consistency in permeability properties which is comparable to that of plain HSC.
- Glass fibre is sensitive in changing of contents when the specimens examined for the compressive strength performance.
- Coconut fibre has the worse effect in affecting durability properties of HSC.

ACKNOWLEDGEMENT

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T132

CHARACTERISATION OF HIGH CALCIUM WOOD ASH FOR USE AS MINERAL ADMIXTURE IN CONCRETE

Cheah Chee Ban[1] and Mahyuddin Ramli[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]volrath15@hotmail.com, [2]mahyudin@usm.my

ABSTRACT: Throughout the laboratory investigation, potential use of High Calcium Wood Ash (HCWA) as partial cement replacement material in production of mortar was investigated. Chemical composition and phases of HCWA were evaluated using X-Ray Fluorescence (XRF) and X-Ray Diffraction (XRD) analytical methods respectively. Physical properties of HCWA namely particle size distribution, average particle size d_{50} and specific surface area were assessed by laser particle size analyzer. In addition, mechanical strength of structural grade mortar mixtures produced by partial replacement of cement using HCWA were investigated as an assessment of the suitability of HCWA as supplementary binder in concrete. A total of 6 mortar mixtures including control mortar (containing only neat cement as binder) and HCWA mixtures with varying level of cement replacement using HCWA were produced. For the HCWA mortar, HCWA was used at partial cement replacement level of 5, 10, 15, 20 and 25% by total binder weight. Water to binder ratio was held constant at 0.35 for all mortar mixtures to maintain desired strength level. Results indicated that the use of HCWA as partial cement replacement did not have significant adverse effect on workability of fresh mortar. Mortar mixture with 15% HCWA exhibited higher compressive strength relative to control mortar at the age of 91days. Enhanced flexure strength was observed for mortar mixture with HCWA content of 5% and 10% at the same age.

Keywords: **cement replacement material, chemical composition, wood biomass ash, recycling, workability, mechanical strength, cement mortar.**

1. INTRODUCTION

The incorporation of wood ash as partial cement replacement material in concrete reduces requirement of hydraulic cement in production of concrete [9]. As the production of cement requires both raw material (Limestone) and energy intensive coupled with massive emission of green house gasses, the reduction in consumption of cement in the manufacturing of concrete is beneficial towards environmental conservation. Besides, the use of wood ash as partial cement replacement material in production of concrete material will also contribute towards reduction in construction cost hence the cost of houses to a level which is affordable to a large segment of the population especially in the developing countries (Udoeyo, Inyang et al. 2006).

Therefore, in recent years, several researchers (Elinwa and Mahmood 2002; Elinwa and Ejeh 2004; Elinwa, Ejeh et al. 2005; Abdullahi 2006; Udoeyo, Inyang et al. 2006; Elinwa, Ejeh et al. 2008; Rajamma, Ball et al. 2009) have investigated mechanical and durability properties of structural concrete and mortar produced by the incorporation of wood ash as partial cement replacement. Results of the studies indicated that inclusion of high silica wood biomass ash as partial cement replacement in structural concrete and mortar up to 20% by total binder weight produces concrete and mortar with acceptable mechanical strength and durability performance.

Some researchers (Elinwa and Mahmood 2002; Udoeyo and Dashibil 2002; Elinwa and Ejeh 2004) detected significant amount of silica in wood ash can be used as pozzolanic additive in concrete. Elinwa and Ejeh (2004) evaluated the pozzolanicity of high silica wood ash and concluded that the ash is highly pozzolanic with pozzolanic activity index (PAI) of 75.9% which exceed the minimum of 70% specified in ASTM Standard C618 (ASTM 2003) for Class N fly ash.

There have been a significant amount of studies on the use of high silica wood ash as pozzolan in concrete and mortar. However, there were few research performed to investigate the properties of concrete or mortar containing high calcium wood biomass ash as partial cement replacement. As wood biomass ash is generally rich in calcium oxide content (Vassilev, Baxter et al. 2010), this necessitate research studies to be performed to assess mechanical and durability performance of concrete and mortar containing woody biomass ash with high calcium oxide content. Such studies are important to determine the viability of high calcium wood biomass ash inclusion as partial cement replacement in the production of structural concrete and mortar for application in the construction industry. The presence of laboratory data on the use of high lime wood biomass ash (which constitutes a large portion of wood biomass ash generated) as constituent material in concrete will enable widespread application of the ash material in the production of cement based material and concrete.

The present work aims to study the characteristics of high calcium wood ash (HCWA) and properties of mortar produced by partial replacement of cement binder by high calcium wood ash. Specific objectives of research were: (1) to characterize HCWA in terms of chemical composition and mineralogical phases of chemical compound of the ash. (2) to study compressive strength of mortar containing HCWA at various level of cement replacement.

2. METHODOLOGY

2.1. MATERIALS

a. Cement

ASTM Type I Portland cement (PC) with median particle size of 3.9µm, specific surface area of 1.0432m^2/g and specific gravity of 3.02 were used in this study. Both physical and chemical properties of cement used comply with specifications in ASTM Standard C150 (ASTM 1997). The chemical composition of PC used is presented in Table 1.

b. High Calcium Wood Ash (HCWA)

HCWA is a by product acquired from a small scale fully automatic boiler unit (commercially known as Bio-Turbomax boiler) used in wood kiln drying factory, Amkawood K.D Sdn. Bhd., in Malaysia . The wood waste biomasses were derived from local timber species dominantly *Shorea Parvifolia*, *Shorea Macroptera*, *Shorea Faguetiana*, *Shorea Multiflora*, *Dyera Costulata* and *Hevea Brasiliensis*. The wood biomasses were incinerated under self sustained burning condition within an atmosphere with turbulence air flow supplied by an in-built air pump unit. Temperature of incineration was maintained within the range of 800±10ºC. Raw ash obtained from the boiler unit were sieved through laboratory sieve with opening size of 1.18mm to remove large agglomerated ash particles and unburned charred materials. Ash passing the 1.18mm sieve was then grinded in a ring mill to fineness whereby merely 0.73% of the ash was retained on 150µm sieve upon dry sieving. The chemical compositions of HCWA are presented in Table 1.

Table 1. *Chemical Compositions, Loss On Ignition (LOI) And Physical Properties Of Portland Cement And HCWA.*

Chemical Compound	Mass %	
	Cement	Wood ash (AKW)
MgO	1.500	10.000
Al_2O_3	3.600	4.100
SiO_2	16.000	28.000
P_2O_5	0.057	3.900
SO_3	3.100	1.000
Cl	n/d	0.050
K_2O	0.340	7.400
CaO	72.000	39.000
TiO_2	0.170	0.130
MnO	0.028	0.510
Fe_2O_3	2.900	2.500
NiO	n/d	0.015
ZnO	trace	0.047
SrO	0.035	0.130
ZrO_2	0.018	0.036
PbO	0.012	n/d
Rb_2O	trace	0.031
C	n/d	2.000
Na_2O	n/d	1.000
As_2O_3	n/d	trace
BaO	n/d	0.077
Loss on ignition (%)	2.53	7.22
Specific Surface Area (m^2/kg)	1043.2	1087.1
Specific gravity	3.02	2.52
Median particle diameter,d_{50} (µm)	3.90	5.16

**n/d indicates that the given compound was not detected in the ash sample*

c. Aggregates

Fine aggregate used were locally sourced quartzitic natural river sand in uncrushed form with specific gravity of 2.83 and maximum aggregate size of 5mm. Fine aggregates were dried to saturated surface dry condition prior to use as constituent material in mortar mixes. Fine aggregates were graded in accordance to BS812: Part 102 (BSI 1989) and grading of fine aggregates used was in compliance with overall grading limits of BS 882 (BSI 1992) as shown in Figure. 1. Fineness modulus of the fine aggregates was determined to be 3.26.

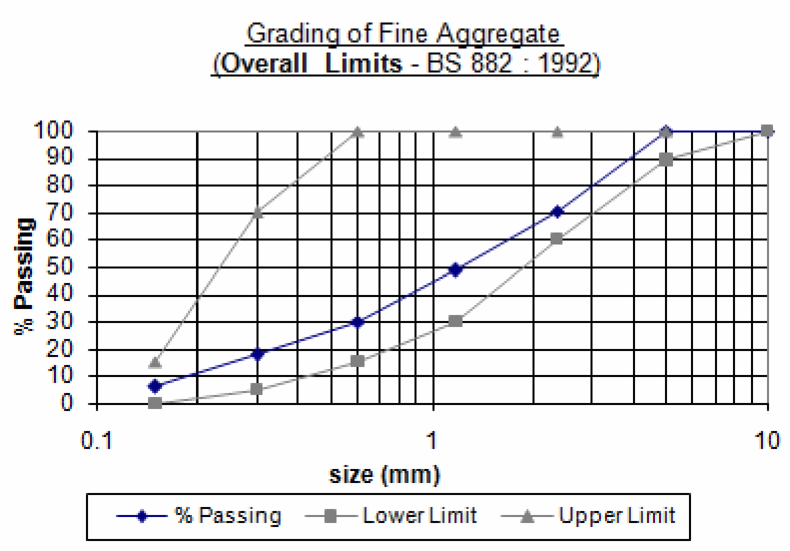

Figure 1. *Grading Of Fine Aggregates.*

d. Superplasticizer and mixing water

Type F superplasticizer of sulfonated melamine formaldehyde condensates category with commercial designation of CONPLAST SP1000 supplied by Fosroc Sdn. Bhd. was used in the study. Potable water from local water supply network was used as mixing water.

2.2. Experimental Programmes

a. Characterisation of HCWA and cement

Chemical compositions of HCWA and cement were determined by X-Ray Fluorescence analytical method using X-ray spectrometer with the commercial name of Rigaku RIX3000. Mineral phases of oxide compounds detected from X-Ray Fluorescence analysis were identified by X-Ray diffraction method using Bruker X-Ray diffractometer. Analysis of X-Ray diffraction pattern for determination of mineral phases was performed using fingerprint method (Whitfield and Mitchell 2008). Loss on ignition (LOI) of HCWA and cement was determined in accordance to procedures prescribed in ASTM Standard C311 (ASTM 2004).

Particle size distribution, median particle size diameter (d_{50}) and specific surface area of both HCWA and cement were determined by laser diffraction analysis using Malvern laser particle size analyzer. Specific gravity values of the samples were determined using Le Chatelier Flask and procedures prescribed in ASTM Standard C188 (ASTM 1995).

b. *Mixture proportioning and mixing*

The binder: sand and water/binder ratios were maintained constant at 1:2.25 and 0.35 respectively for all mortar mixtures cast. The Portland cement binder was partially replaced using HCWA at substitution level of 5, 10, 15, 20 and 25 % by total binder's weight. Superplasticizer was dosed at appropriate dosages to maintain desired mortar slump of 70±20mm, as prescribed in BS EN 206: Part 1 (BSI 2000) as S2 (medium workability) slump range. The mix design of control mortar mixture (C) was performed using absolute volume method prescribed in design code ACI 211.1-91 (ACI 2000) to achieve compressive strength of 40MPa at the age of 28days. The composition of mortar mixtures are summarised in Table 2.

c. Mortar mixing and curing

Each batch of mortar was produced using an epicyclic type mechanical mixer complying with specifications in ASTM Standard C305 (ASTM 1994). During mixing of mortar mixtures containing HWCA in partial substitution of Portland cement, HCWA and Portland cement were initially dry mixed at low mixing speed for duration of 3 minutes prior to addition of other constituent materials. Further mixing sequence and durations were performed in accordance to standard procedures prescribed in ASTM Standard C305 (ASTM 1994).

d. Rheological properties

ASTM Flow test was performed on fresh mortar mixtures using flow table complying with specifications prescribed in ASTM Standard C230 (ASTM 1997) and standard testing procedures described in ASTM Standard C109 (ASTM 1998). Besides, slump values of the mortar mixtures were determined by slump test performed in accordance to procedures prescribed in BS 1881: Part 102 (BSI 1983). Fresh mortar flow and slump values obtained are presented in Table 2.

***Table 2**. Proportion Of Constituent Materials And Rheological Properties Of Mortar Mixtures.*

Batch Designation	% HCWA	Cement (kg/m^3)	HCWA (kg/m^3)	Sand (kg/m^3)	Water (kg/m^3)	SP Dosage (%)	*w/c*	Mortar Flow (%)	Slump (mm)
Control (C)	0	684	0	1538	239	1.85	0.35	84	90
A5	5	650	34	1538	239	1.85	0.35	68	70
A10	10	616	68	1538	239	1.85	0.35	57	50
A15	15	581	103	1538	239	1.98	0.35	62	50
A20	20	547	137	1538	239	1.98	0.35	58	50
A25	25	513	171	1538	239	1.98	0.35	54	50

e. Compressive Strength, Ultrasonic Pulse Velocity (UPV) and bulk density tests

Mortar cube specimens with edge dimensions of 50 mm were moulded, cured and tested in accordance to procedures described in ASTM Standard C109 (ASTM 1998) for determination of compressive strength of hardened mortar mixtures produced. All mortar specimens fabricated were cured in lime saturated water for duration of 3, 7, 28 and 90 days prior being tested. The reported compressive strengths at given age of mortar are the average of three number of specimens tested.

Propagation velocities of transmitted ultrasonic pulse through hardened mortar mixtures were determined using an electrical pulse generator and testing methods prescribed in BS EN 12504: Part 4 (BSI 2004) on a representative mortar prism specimen with dimensions of 100x100x500mm. Transmission of ultrasonic pulse was performed by direct transmission method through constant path length of 100mm between transducers. Bulk densities of hardened mortar mixtures were determined in accordance to methods in BS 1881: Part 114 (BSI 1983).

3. RESULTS AND DISCUSSIONS

3.1. Characterisation of HCWA

a. Chemical composition of HCWA

The results of X-ray fluorescence analysis on HCWA are presented in Table 1. From the test results, it can be observed that the major oxide compounds present in HCWA are CaO, MgO, SO_3, Al_2O_3, SiO_2, P_2O_5, Na_2O, K_2O, and Fe_2O_3. Other substances namely Cl, TiO_2, MnO, ZnO, SrO, ZrO_2, Rb_2O and BaO was detected in minor composition. Apparently, CaO, SiO_2 and MgO are the dominant compounds each comprising 39%, 28% and 10% of total mass of HCWA. Large composition of CaO of the ash indicates that the ash could be hydraulically reactive (Rajamma, Ball et al. 2009).

The sum of composition of essential pozzolanic oxide namely SiO_2, Al_2O_3 and Fe_2O_3 of HCFA was found to be merely 34.6%. Hence, HCFA does not satisfy the requirements prescribed in ASTM Standard C618 (ASTM 2003) to be classified as Class N pozzolan. However, the SO_3 compound which constitutes 1% by total mass of the ash is in good compliance with the upper limit of 4% prescribed in the standard.

Loss on ignition of HCWA was found to be 7.22% which is significantly higher in comparison to Portland cement (2.53%). The significant loss on ignition of HCWA is due to the fact that although the wood biomass was incinerated at high temperature of 800±10ºC within the boiler, there is always some inefficiency in carbon conversion due to kinetic and mass transfer limitations. As a result, some amount of organic residue is present within the ash (Rajamma, Ball et al. 2009).

b. X-ray diffraction analysis

The X-Ray diffraction pattern of HCWA is shown in Figure 3. A careful analysis of X-Ray diffraction pattern indicated that the major chemical phases of the HCWA were Monticellite ($CaMgSiO_4$), Akermanite ($Ca_2MgSi_2O_7$), Pectolite ($HNaCa_2Si_3O_9$), Calcium Potassium Hydrogen Phospahate ($CaK_3H(PO_4)_2$) and Calcium Borate ($Ca_3(BO_3)_2$). As natural timber does not usually contain Borate chemical constituent, the presence of Borate compound in HCWA is most probably due to the use of Boric acid in the treatment of raw timber during the timber product manufacturing process. The presence of Monticellite and Akermanite is in close agreement with the result of X-Ray Fluorescence analysis, Table 1, which indicates presence of significant quantities of CaO (39%), MgO (10%) and SiO_4 (28%) within HCWA. Broad diffuse band between 24º and 37º on 2θ scale of the XRD pattern of HCWA is an indication that the silicate minerals were glassy in nature (Lin, Wang et al. 2003). Silicate mineral in glassy or amorphous state is highly reactive with Portlandite produced from hydration of cement to form additional calcium silicate hydrate (CSH) gel. Additional CSH gel formed contributes towards reduction of Portlandite amount and at the same time increases quantity of CSH gel hence resulting in overall enhancement in strength of hardened cement paste matrix.

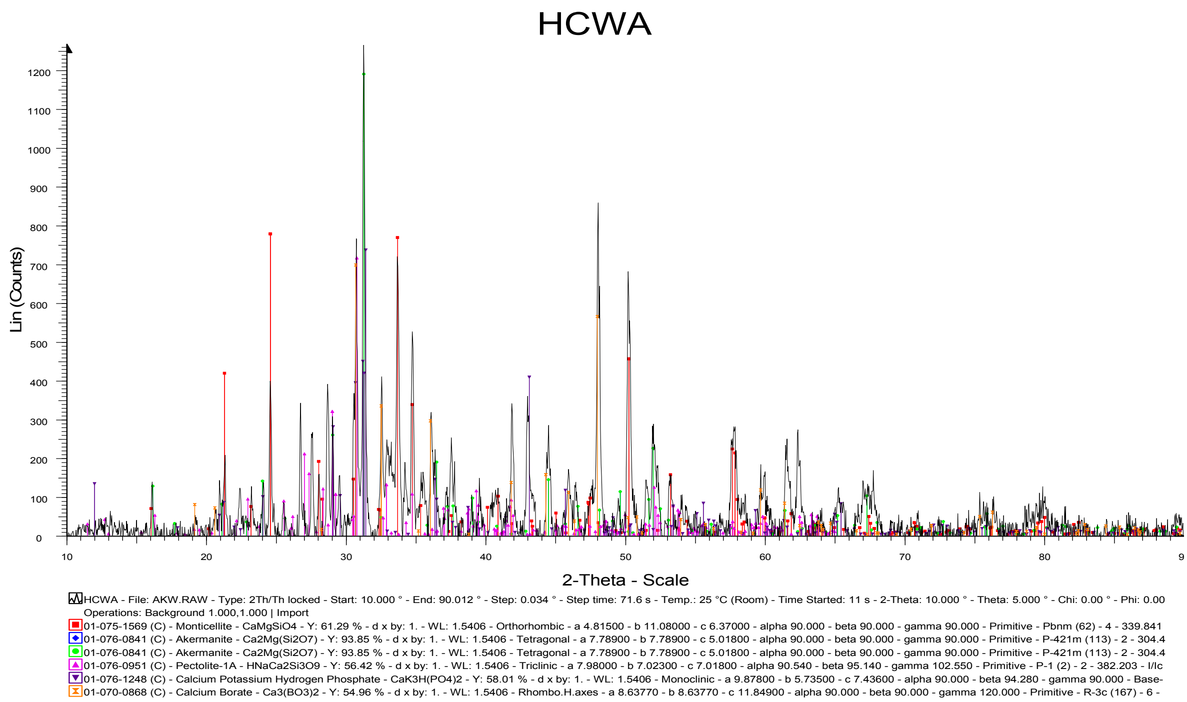

Figure 3. *XRD patterns of HCWA.*

3.2. Compressive strength

Compressive strength test results and bulk densities of hardened mortar are presented in Table 3 Additionally, the compressive strength development of the hardened mortars containing various level of cement replacement by HCWA is presented in Figure. 4. Compressive strength test results indicate that target 28 days compressive strength of 40MPa was achieved by mortars containing various level of cement replacement with HCWA up to 20% of total binder weight. Generally, at early age of hydration up to 28 days, it was observed that the increase level of cement replacement from 0% to 25% at stepped increment of 5% resulted in gradual decrease in compressive strength of mortars. A similar trend was observed by several other researchers (Elinwa and Mahmood 2002; Udoeyo and Dashibil 2002; Udoeyo, Inyang et al. 2006; Wang, Baxter et al. 2008). However, under prolonged curing duration beyond 28 days, a convergence of compressive strength between mortar with HCWA and the control mortar could be observed. The higher rate of strength gain of mortar mixtures containing HCWA on prolonged curing duration of 7 days up to 90 days which contributed to the convergence in compressive strength is most probably due to presence of pozzolanic reaction between Portlandite produced from hydration of Portland cement with amorphous silica content of HCWA. At the age of 90 days, compressive strength of mortar with 15% HCWA was found to exceed the compressive strength of the control mortar by 2.7% while other HCWA mortar with HCWA content up to 25% maintained a marginal difference in compressive strength relative to the control mortar.

In the production of structural mortar, the optimum level of cement replacement with HCWA is 15%. At this level of cement replacement long term strength of mortar mixture at the age of 90 days was found to be higher than that of Portland cement mortar. In addition, the correlation between compressive strength of mortar produced with propagation velocities of ultrasonic pulse transmitted through the hardened mortar is shown in Figure. 5. Regression analysis reveals the following relationship between ultrasonic pulse velocities with corresponding compressive strength of the mortar.

$$f_{cu} = 77.78v^3 - 1092.1v^2 + 5108.2v - 7913.4 \qquad (1)$$

where f_{cu} = compressive strength of the mortar (MPa) ; and v = ultrasonic pulse velocity (km/s)

Table 3. *Compressive Strengths And Bulk Densities Of Hardened Mortar Mixtures.*

%Wood waste Ash	Bulk Density (kg/m^3)	Compressive strength (MPa)-normalized			
		3days	7days	28days	90days
0	2266	34.89-100	38.15-100	44.90-100	48.25-100
5	2281	31.89-91.4	37.04-97.1	44.39-98.9	46.81-97
10	2274	31.61-90.6	36.54-95.8	42.98-95.7	47.26-97.9
15	2258	27.01-77.4	31.85-83.5	41.77-93	49.55-102.7
20	2268	27.03-77.4	33.61-88.1	40.06-89.2	44.64-92.5
25	2273	26.52-76	33.34-87.4	38.99-86.9	43.77-90.7

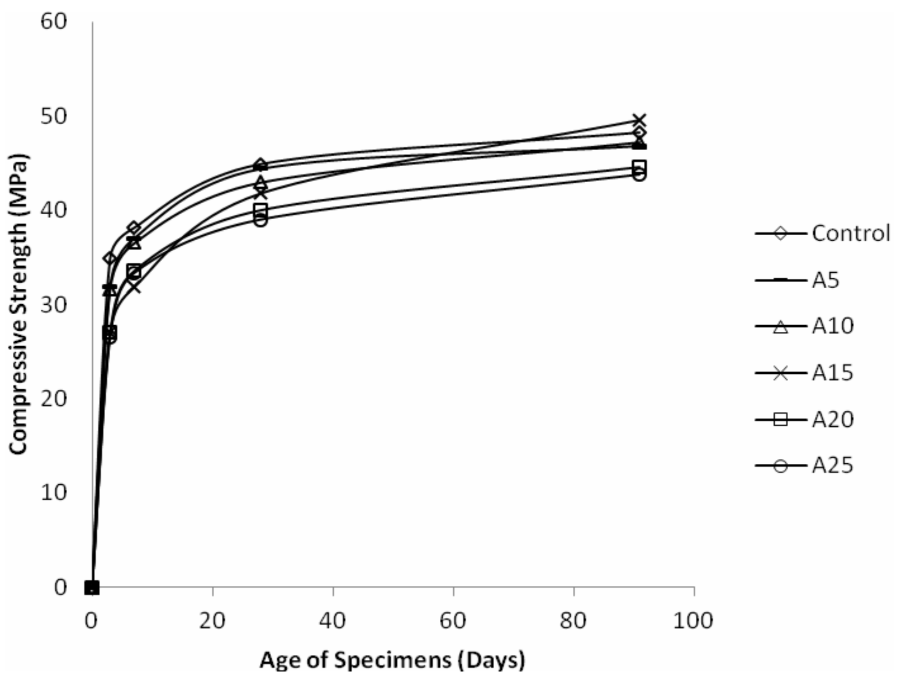

Figure 4. *Compressive Strength Development Characteristics.*

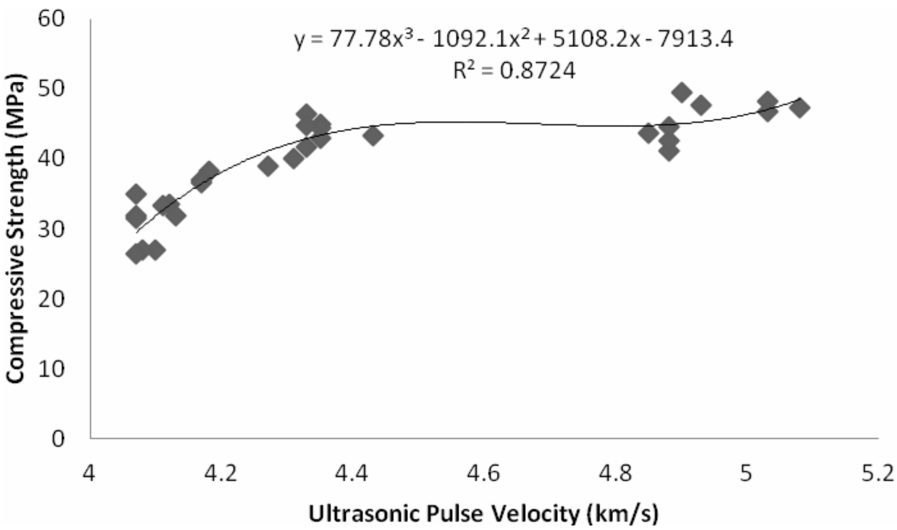

Figure 5. *Relationship Between Compressive Strength And Ultrasonic Pulse Velocity Of HCWA Mortar.*

4. CONCLUSIONS

As referred to results acquired throughout the laboratory investigation, the following conclusions can be derived:

- Median particle size (d_{50}) and specific surface area of HCWA was found to be higher than that of ordinary portland cement.
- Major oxide compound present in HCWA is silica (28%) and calcium oxide (39%).
- Silica mineral found in HCWA were glassy or amorphous in nature.
- Incorporation of HCWA in the mortar mix as partial cement replacement material up to 10% by total binder weight did not incur additional dosage of superplasticizer to maintain constant level of workability. The use of HCWA at higher level of cement replacements (15%, 20% and 25%) raised superplasticizer demand by 7% to achieve a specified level of workability.
- Mortar containing 15% of HCWA by total binder weight exhibited higher compressive strength compared to that of the control mortar mix on prolonged curing duration of 90 days. Incorporation of HCWA at cement replacement level up to 25% produced mortar mix with compressive strength exceeding 90% of the control mortar.
- The optimum level of cement replacement with HCWA with reference to compressive strength is 15%. Higher level of cement replacement resulted in lower compressive strength.

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ILLUMINATING INDOOR SPACES FOR THE WELL BEING OF OCCUPANTS USING INNOVATIVE ROOFING SYSTEM

Hazril Sherney b. Basher[1], Abdul Malek Abdul Rahman[2],
[1,2]Universiti Sains Malaysia, Penang, Malaysia,
[1]hazril_skywalker@yahoo.com , [2]malik@usm.my

ABSTRACT: Illuminating indoor spaces using natural or passive solution is the key of saving energy from burning fossil fuel and energy consumption. Its function is to illuminate the indoor spaces of all types of building design during broad daylight without excessive usage of artificial lighting. The creation of innovative roofing system consist of glazing roofing and hybrid turbine ventilator (HTV) which can reduce the building indoor temperature as well as giving independent free natural lighting to building occupants. Even though glazing roofing tend to create green house effect towards indoor spaces which lead to adverse heat and increasing indoor temperature, the HTV kicks in to extract heat from radiated roof top and building spaces. Thus it increases indoor air quality and occupants thermal comfort. Basic parameters of thermal comfort were measured: air temperature, relative humidity, air velocity and solar radiation. The parameters were measured using experimental scaled model which project accurate controlled empirical data for analysis before and after implement of the innovative roofing system. The findings from experimental and simulation measurement reveal that the innovative roofing system reduces the temperature of indoor spaces using single HTV on scaled model rooftop and giving the occupants clean and free lighting source. This reliable data will open new gateways for future recommendation to improve passive lighting and indoor air quality.

Keywords: natural illumination, stack ventilation, turbine ventilator, indoor spaces, thermal comfort

1. INTRODUCTION:

Nowadays, the world population have consumed and depleted major resources all around the globe. Many sectors are seeking to restore and to find alternatives to energy depletion. In the year 1983 sustainable development (SD) were introduce in the World Commission on Environment and Development (WCED) to address the nation concerning human resources that will ultimately deteriorate nations social and economical development. In architecture background many form or solution have been introduce and implement in building sectors and design so that people will appreciate the fundamental of sustainable architecture. In Malaysia the Green Building Index (GBI) have been carried out and merely will formed as compulsory in later years to come an energy efficiency index to calculate building green performance(MS 1525 :2001 UBBL code).

In this paper, the main issues regarding energy consumption is burning fossil fuel using air conditioning system (A/C) and also excessive usage of artificial lighting during broad daylight which will lead to many circumstances and increasing of annual electricity bills. Moreover, the continuity of using artificial lighting and air conditioning system in straight hours will lead to unhealthy living. Therefore, desperate measures have to be taken as a step to counter the problems that occur in illumination and ventilation. According to Gon Kim et al. (2009) to embody a healthy lifestyle or living, architectural design plays a very important role. The innovative roofing system is an alternative approach or green solution which combine both lighting and ventilation solution to solve the problem statement in order for occupants achieve thermal comfort and sustainable living. In previous research combination of both approach have been introduced such as the used of conventional ventilator and light pipe by Zain-Ahmed et al (2007), it is also stated that by solving simultaneously illumination and ventilation approach, problem of energy consumption will be solved without additional cost. By following this concept of both approach, a study consider unique due to the fact it investigates occupants thermal comfort which is safer, greener and will not consume major electricity bills will be conducted.

The innovative roofing system consists of a Hybrid Turbine Ventilator (HTV) and glazing roofing to allow day lighting penetration. The function of the glazing roofing is to enhance day lighting or illumination as a natural resource of lighting during broad daylight. Its function is considered to be an alternative to artificial lighting such as fluorescent light. Natural illumination is a free source of lighting which comes from sun rays. In this research the unit to calculate the intensity of light luminance is lux. To achieve certain amount of lighting in indoor spaces vertical approach such as the atrium system is remarkable compare to minimal source of lighting from windows opening which is horizontal. In building design, recommendation of light level is essential for occupants to perform activities. These activities rely mostly on certain amount of light level intensity.

Table 1.*Ttable of recommendation of light inside indoor spaces*
(Source www.engineringtoolbox.com)

Activity	Illumination *(lux, lumen/m^2)*
Warehouses, Homes, Theaters, Archives	150
Easy Office Work, Classes	250
Normal Office Work, PC Work, Study Library, Groceries, Show Rooms, Laboratories.	500

Table 1 indicates normal illumination or recommended light level for indoor spaces which rely mostly on types of activity performed (www.engineringtoolbox.com). Normal standard requirements for an effective lighting inside indoor space as shown for this paper are set from 500-1000 lux. Even though the standards shown are example readings of lux level using artificial lighting, the real natural illumination will definitely exceed the amount of lux level due to material chosen for method of data collection. The glazing roofing as mentioned act as a vertical source and projects atrium concept which allow day lighting penetrates from above. These natural lighting or illumination will bring along heat as well as sunlight inside an indoor space. The sunlight penetrates further until it reaches the floor level, and at the same time left a small amount of heat trapped on the attic space which is located above standard ceiling level. This glazing roofing system although brought beneficial to occupants, at instance it also creates green house effect inside indoor spaces. As agent of anti-thermal comfort, it tends to transfer the heat load downwards to occupants level using radiation concept. (HQ technology, 2009). Therefore, secondcomponent or crucial part is needed to balance this natural approach.

The second component is the hybrid turbine ventilator (HTV). It's a component which combines a conventional turbine ventilator and powered by 40watts PV panel. The HTV component is also a hybrid powered system which not just relies on solar radiation but also wind driven when there are abundance of sunlight. These make the innovative roofing is efficient system which save energy by utilising both approach natural illumination and ventilation to achieve occupant's thermal comfort. The HTV operates as to extract heat load that is trapped inside the attic space of a building design which consider as the main source of heat load inside a building.

As mentioned from Ismail.M (2009) by placing the HTV device at attic space level compare to occupants level will diminish the temperature inside a building efficiently. During high intensity of solar radiation the HTV component will kick start and extract the heat inside the attic space. From 0 m/s in existing level, it subsequently reach until max 0.48m/s the amount of upward induce air which flows directly upon HTV extraction. The innovative roofing system relies mostly between two components mentioned. If using only one component, efficiency will not occur. The innovative roofing system is one of a kind. It implies by giving us free natural illumination using glazing roofing and yet induced indoor air quality.

2. METHODOLOGY

In this paper, method of research relies mostly in the usage of quantitative approach which is an experimental data. Experimentation was carried out using a scaled model that is specifically design for data measurement of thermal comfort parameters. The thermal comfort parameters that were collected in this research are indoor temperature, ambient temperature, solar radiation, relative humidity, internal air velocity and lux intensity. The scaled model is made out of wooden framework material and plywood panels. The roof covering material is from profiled metal decking roofing attached with a 10mm Perspex clear sheet. It acts as glazing roofing in real condition. The materials that have been chosen definitely will create serious heat gain due to non insulation aspects. Its function is to determine or simulate on worst case scenario, to see any impact factor differential in thermal comfort parameters.

The main data will be presented by experimental measurements. The instrumentation or equipment use in this paper is Data Acquisition System (DAQ). The DAQ system is connected via electrical cable and at the end of the cables probes or sensors is attached to measure readings of thermal comfort parameters. The HTV system which consists of a 450mm or 18" in diameter modified turbine ventilator is placed on top of the scaled model. This component is attached with PV panels that are tilt until it reaches 30° angle. The orientation of the model is set to east side orientation as mentioned by Ismail.M (2009), the HTV tend to start extraction when it is placed towards this orientation. The duration of this research measurements is about 1 month from Feb to Mac 2011 and data are taken 9.00am till 5.00pm at every 5 minutes intervals. In this paper selected data are taken from 3 days of perfect clear weather condition.

Measurements that have been done during these months are considered as the driest period of the year referred to Meteorological Department Bayan Lepas Penang. The data collection or measurements also will be conducted in 3 phase. Phase 1 is measurements on existing situation where as data will be collected on the scaled model existing condition. Only natural illumination will penetrate inside the indoor of the model itself. In Phase 2, application of the HTV system will be implement thus will see any differential data regarding thermal comfort parameters in enclosed situation. The last phase is considered measurement to induced stack ventilation by creating small opening on the side of the scaled model. P. Priyadarsini et al (2003) mentioned that by commencing this last phase of creating an opening, it will lead to internal air velocity fluctuations as it induces stack ventilation. However, in this paper, the only phase 1 &2 experimental programmes will be explained in detail.

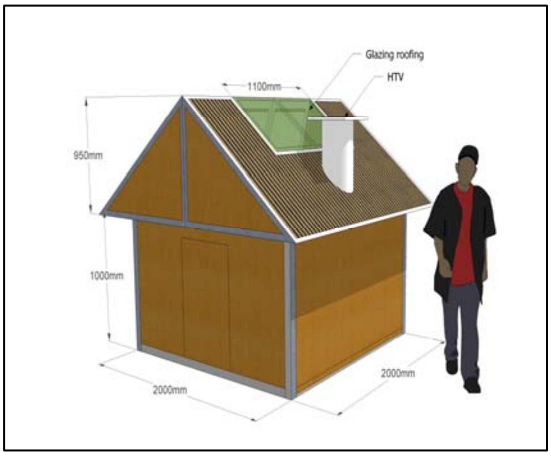

Figure 1 & 2. *Prototype of scaled model to measure thermal comfort parameters.*
(Source author)

2.1 Experimental Programmes

a. Test 1: Existing Condition

Data for phase 1 and phase 2 are included to see differential of thermal comfort parameters. Recent studies indicated that indoor air temperature was lower than ambient temperature. However for the scaled model existing condition, the indoor temperature presents higher than the ambient temperature or outside temperature. This is due to material selection which has no insulation of heat as to investigate worst case scenario. During the investigation, the internal temperature projects around maximum of 45°C.The internal air velocity presents a stagnant 0 m/s as there is no air movement occur inside the scaled model.

***Table 2.** indicates existing thermal comfort parameters from scaled model measurements (source author)*

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	14335.64	953.06	35.33	43.76	0	78.58
Day 2	12602.56	950.14	36.94	45.75	0	76.56
Day 3	12766.5	1002.84	36	45.65	0	66.84
Mean.max	13234.9	968.68	36.09	45.05	0	73.99
Max	14335.64	1002.84	36.94	45.75	0	78.58

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	4218.2	115.66	26.33	28.82	0	34.68
Day 2	3216.99	171.29	27.22	30.02	0	34.9
Day 3	4505.09	363.07	29.61	33.08	0	34.68
Mean min	3980	216.67	27.72	30.64	0	34.75
Min	3216.99	115.66	26.33	28.82	0	34.68

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	10212.4	584.62	31.42	38.17	0	53.8
Day 2	9866.3	584.62	32.96	40.76	0	48.8
Day 3	8213.3	732.16	33.83	41.74	0	44.95
Average	9430.7	633.8	32.73	40.22	0	49.18

b. Test 2: Application Condition

For phase 2, after existing measurements have been taken, the application of HTV system will be carried out for the scaled model. As observed in phase 2 for application condition for enclosed case, the readings illustrated slight lower or drop in internal temperature. While the internal air velocity increase gradually until max 0.48 m/s. This means when higher the air change rate (ACH) the lower the temperature will become. When sunlight penetrates through the glazing roofing and reaches to a certain amount of lux, it also brings along solar radiation. These intense solar radiations from 900-1000 W/m2 ignite the HTV to start rotate as well as extract the heat inside the scaled model.

Table 3. *Indicates application of HTV system in thermal comfort parameters from scaled model measurements (source author)*

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	10242.99	1002.84	36.78	41.47	0.37	72.5
Day 2	16595.66	1020.41	34.91	38.93	0.35	73.32
Day 3	60016.34	1045.3	36.18	41.25	0.48	66.22
Mean.max	28951.7	1022.9	35.95	40.55	0.4	70.68
Max	60016.34	1045.3	36.78	41.47	0.48	73.32

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	3158.44	102.48	27.14	29.37	0	35.86
Day 2	3158.44	109.8	26.12	28.87	0	39.32
Day 3	2526.1	84.91	29.76	33.64	0	40.34
Mean min	2947.7	99.06	27.67	30.62	0	38.5
Min	2526.1	84.91	26.12	28.87	0	35.86

	Lux meter (lux)	Solar radiation (W/m2)	Outdoor temp. ('C)	Indoor temp. ('C)	Indoor air vel. (m/s)	Indoor Rhum (%RH)
Day 1	7279	711.34	34.01	37.79	0.18	46.64
Day 2	10978	550.3	31.76	35.67	0.13	52.64
Day 3	26877	647.73	32.63	37.35	0.21	48.45
Average	15044.7	636.45	32.8	36.93	0.17	49.24

From the experimental tables observed above, it shows data comparison from existing to the application of the innovative roofing system. For existing condition in Table 2, the maximum lux penetrates inside the indoor spaces at 14335.6 lux. The average lux intensity is estimated 9430.7 lux. Therefore the lux intensity is efficient enough to replace artificial lighting fixtures in broad day light. The existing internal temperature reaches until maximum level of 45.75 °C while the average value is 40.22°C. The internal air velocity projects a stagnant 0 m/s. This means that there are no air change rate (ACH) occurs inside the scaled model at existing level since there was no air infiltration. The different value between maximum indoor-outdoor temperatures for existing condition is around 8.81°C.

With reference from the trend in table 3, it shows a list of parameters which present significant value after implement the HTV system onto the scaled model. The lux intensity reached until maximum level of 60016.34 lux considers being the highest lux during data measurements. The maximum indoor temperature fluctuates until 41.47 °C while average readings show 36.93°C. The average drop of indoor temperature after inserting the HTV system is 3.29°C or 8.18%.Further investigations presents an astonishing reading where as by using single HTV system it is capable of achieved a maximum drop of 4.28°C.

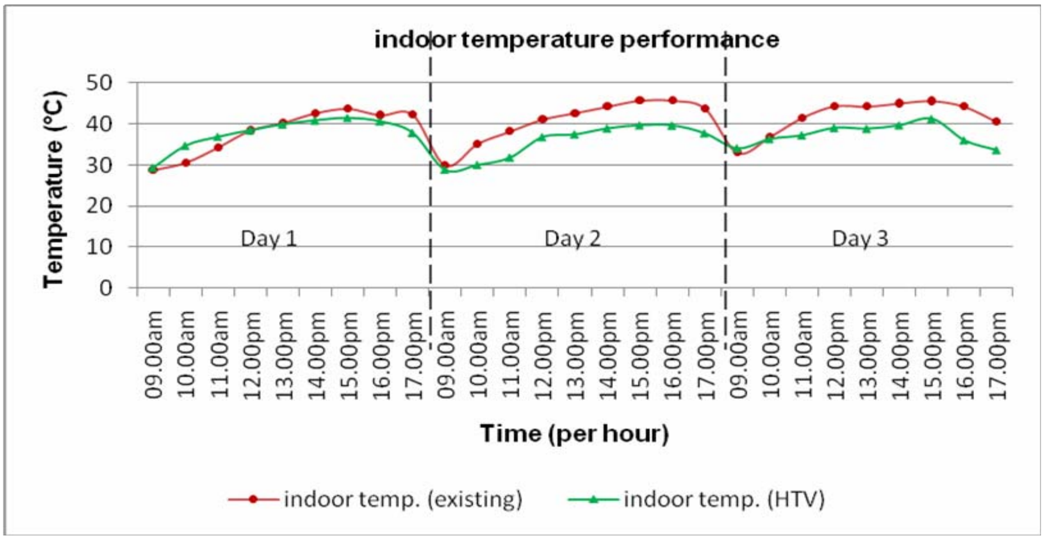

Figure 3. *Comparison of internal temperature differences for existing and application of HTV situation. (Source author)*

The indoor air velocity increased from 0m/s (existing) to a maximum of 0.48 m/s (HTV) in day 3 when solar radiation reached 1045 W/m2. The mean average of fluctuation increase is about 0.17 m/s. This shows that the innovative roofing system has the potential to create exceptional occupants thermal comfort by induce the upward air velocity and also extract the heat load from the attic space before it reaches occupants level. Szokolay (1998) informed that the standard effective of velocity is 0.25m/s. Although the average presents a lower rate, still it is consider an effective approach since it is based on single HTV device. The average lux intensity projected from the scaled model is around 9000 lux – 15000 lux. During observations, the innovative roofing system triggers itself and started to induced air velocity when solar radiation is set from 800-1000 w/m2.

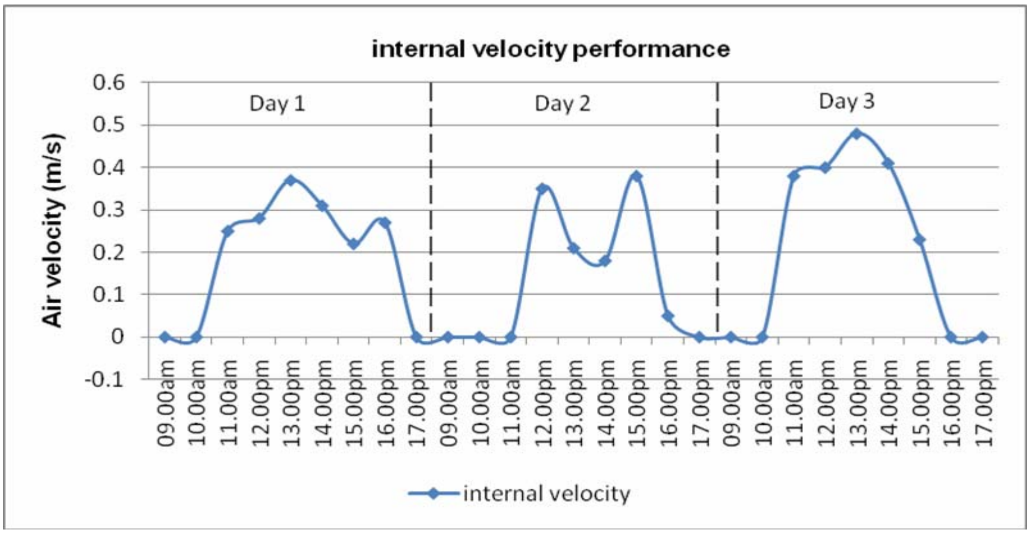

Figure 4. *Internal velocity performance application using HTV situation.(Source author)*

3. CONCLUSIONS

From experimental results, it was also found that by using the glazing roofing itself not only created a green house effect but also brings non exceptional thermal comfort inside an indoor space. The investigation proves that the system has the potential to reduce energy consumption by saving electricity from lighting fixtures and over usage of air conditioning system during office hour. The innovative roofing system reduced the temperature in terms of internal attic space and induced good air velocity. The ability to induce upward air velocity up to a maximum of 0.48m/s and diminished heat load at estimated of 3-4°C are an astonishing findings due to enclosed situation and non insulation material wised. Even though the achievement of enhancing natural day lighting has been sort out but still lowering temperature until the occupants standard effective temperature (SET) has been an issue. The SET level of 28.5°C was introduced by Ismail M (2009) which contradicts with Abdul Rahman (1999) and Zain-Ahmed et al. It is said that people of Asian countries are more tolerable to acclimatization of hot humid conditions nowadays. The scaled model itself is a non insulation material which generally built up heat load quickly than conventional material such as brickwork. As mentioned it acts as a worst case situation to identify minimum and maximum requirements of parameters to achieve thermal comfort. Due to the small cubic size, the capacity of 1 nos HTV device is needed to balance the natural illumination that penetrates the glazing roofing. The application of using this system alone is considered minimal efficiency. The results shows that in future by adding multiple solutions such as improved material wised and quantity of HTV will possibly create an effective level for occupant's thermal comfort and thus exceed the standard effective temperature (SET).

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THE PERFORMANCE OF THREE DIFFERENT SOLAR PANELS FOR SOLAR ELECTRICITY APPLYING SOLAR TRACKING TIMER DEVICE UNDER THE MALAYSIAN CLIMATE CONDITION

Azhar Ghazali. M[1] and Abdul Malek Abdul Rahman[2]

[1, 2]Universiti Sains Malaysia, Penang, Malaysia

[1]azhar.dtyco@gmail.com , [2]malik@usm.my

ABSTRACT: Excessive usages of fossil fuel as main energy source have become the main factor that contributes to the global warming. Carbon dioxide emissions from the combustion of fossil fuel increase the temperature of the atmosphere. Besides that, the depletion of this energy source also became a major factor to the world's energy crisis. As Malaysia move towards a developed country, energy requirement will remain very intensive. To prevent future problems of world energy crisis, Malaysia has embarked into renewable energy by introducing it as the Fifth Fuel Policy. The government would help out in terms of incentives, enabling policies and tax cuts for energy saving and efficient appliances and machinery. Since 2005 the Malaysian Energy Center (Pusat Tenaga Malaysia now known as the Green Tech Office) has been campaigning for the use of solar electricity with the help of the Danish Government via its implementation arm program known as DANIDA (from 2005 to 2010). This paper focuses on the three common types of solar panels available in the Malaysian market and to identify the most efficient in term of power conversion using the timer-tracking system by optimizing the sunlight and also to compare the performance between static system and dynamic system. Primarily for the Engineers and Architects, this research will help them identify critical technical factors in maximizing the energy output of different types of PV technologies at the same time consider the engineering and aesthetic issues associated with the roof mounted grid-connect PV system.

Keywords: **photovoltaic system, building integrated photovoltaic (BiPV), tropical climate, solar tracker**

1. INTRODUCTION

Excessive usages of fossil fuel as main energy source have become the main factor that contributes to the global warming. Carbon dioxide emissions from the combustion of fossil fuel increase the temperature of the atmosphere. As Malaysia move towards a developed country, energy requirement will remain very intensive. At present, almost half the energy consumption in the nation is consumed by the industrial, residential and commercial sector (APEC, 2006). This means Malaysia has a strong need to apply energy efficient strategies in lowering energy consumption in buildings.

Solar energy has been acknowledged as a free and infinite source of energy and provides an alternative energy where there is no pollution of the environment and its use will decreases the rate of depletion of energy reserves (A.M. Sharan, 2008). As one of renewable energy, solar energy represents a massive energy potential greatly exceeding the fossil fuels. Malaysia lies in the tropical region between 1^oN and 7^oN, and 100^oE and 119^oE (Kamaruddin, R, *el at*, 2002). Its weather condition in Malaysia is very suitable for photovoltaic implementation, this is because the weather condition is almost predictable and the availability of sunlight for more than ten hours daily with almost six hours of direct sunlight with irradiation of between 800 W/m^2 and 1000 W/m^2 (N. Amin *et al*, 2009).

According to Deo Prasad & Mark Snow (2005), photovoltaic are solid-state devices that simply make electricity out of sunlight, silently and with little to no maintenance, no pollution and no significant depletion of material resources. According to T. Markvart, (1994), the design of the photovoltaic system relies on the input of measured data close to the site of the installation which is the regular daily and yearly variation due to the apparent motion of the sun, irregular variations are caused by the climate condition (cloud cover), as well as by general composition of the atmosphere. Besides that, different materials and different structures of photovoltaic panels provide different efficiency in term of power conversion.

According to A.J. Carr, T.L. Pryor (2004), photovoltaic technologies have different seasonal patterns of behavior and these differences are due to the variations in spectral response. There is evidence that modules of differing technologies could be more suited to certain specific climates. Mieke (1998) for example, has reported that in this tropical climate, with high ambient temperatures and high humidity during the wet season, the a-Si array produces up to 20% more energy than the p-Si array. Akhmad *et al*. (1997) have also indicated that a-Si modules may be more suited to tropical climates. N. Amin *et al*, (2009), found that Amorphous silicon and Copper Indium Diselenide (CIS) solar cell have shown better performance ratio than mono- and multi-crystalline silicon solar cells in Malaysia climate condition.

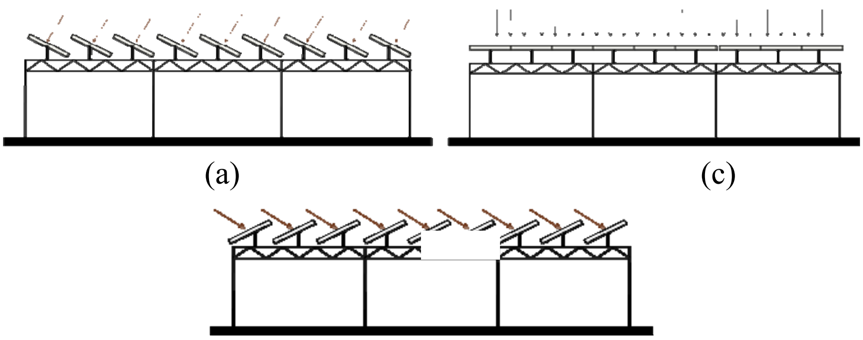
Commonly in Malaysia, PV panels were installed fix to the building either on the roof this installation considered as *static system*. These static systems only maximize sun availability at certain time during daytime. While a dynamic system use Solar Tracker; a device for orientating Photovoltaic Panel (PV) towards the sun. This is to ensure that the concentration sunlight is directed to the focal point of the PV and this will improve the amount of power produced by the system by enhancing morning and afternoon performances. The use of tracking technology allowing solar modules to follow the course of the sun (and optimize the incident angle of sunlight on their surface) can increase electricity production by around a third, and some claim by as much as 40% in some region, compared with modules at a fixed angle (Appleyard, D. 2009).

This research focuses on the three common types of solar panels available in the Malaysian market and to identify the most efficient in term of power conversion using single-axis time/date tracking system by optimizing the sunlight and also to compare the performance between static system and dynamic system. Primarily for the Engineers and Architects, this research will help them identify critical technical factors in maximizing the energy output of different types of PV technologies.

2. METHODOLOGY

In Malaysia, the sun is always directly overhead and its solar intensity do not varies by season, and for this matter Single-axis time/date solar tracker is used based on method suggested by Abdallah *et al* (2008) and Francisco Duarte *et al* (2010). Therefore, the movements of the PV panels during daytime are perpendicularly toward the sun and followed it from east to west. This type of method was selected because it's cheaper compare to the other method due to simpler mechanism (step motor and gear speed), low power consumption from the motor and electric system and as sensors are not advisable to be used in Malaysian climatic conditions because of the heavy cloud cover. Sudden change in sky condition might interrupt the signal to the sensor.

The system in this experiment is design to support the three types of PV panels which allowed it to move with single degree of freedom (horizontally) according to the sun's orientation (sunrise to sunset) from east to west. Studies on sun's movement in equator countries have been conducted to indicate the sun's position on site (azimuth and altitude angle) using the Stereographic diagram. Figure 1 show, the basic diagram of the panel's movement according to sun's position in morning, noon and evening. This movement allowed the PV panels to set perpendicularly toward the sun insolation (incoming solar radiation) so that solar rays can be optimized from morning until evening.



(b)

Figure 1. *(a) Panels oriented towards morning sun. (b) Panels oriented directly under mid afternoon sun. (c) Panels oriented towards afternoon sun*

With the clock running, the electrical motors control the drives of the panel's support structure to the desired position. The electrical motor run at every 30 minutes, in order to orientate the panel support structure to the position. The runtime of this process was done between 09:00 and 16:00 at local time (daylight saving) or in local time between 8:00 and 15:00. The standard position of the panel was programmed in the motor control mode, starting from 09:00 (local daylight saving) at 60^o East. At 23:00 (daylight saving) of each day it commanded the panel's structure to the standard position and starting at 09:00 (local daylight saving) on the next day, the microcontroller sends the command every 30 minutes to the motor, to move them in the desired position. With this control mechanism, the PV panels will be orientated toward the sun to maximize the insolation on the PV panels at every 30 minutes between 09:00 and 16:00 every day. The three types of PV installed are exposed to condition which are exactly identical, which is during the same period, with identical stand and near to each other, so all the three type of PV panels received the same solar radiation, so readings can be compared to verify the efficiency of the panels under the northern region climatic conditions. All the reading or parameters produced by the installed photovoltaic panels was stored in data logger in the control room every 15 minutes. In this installation, the right three rows are installed with amorphous solar panels, the middle three rows are the mono-crystalline and the left three rows are installed with polycrystalline panels with its own specification as mention in Table 1.

Table 1. *Specification of three type of PV panels installed*

	Detail	Poly Crystalline	Mono Crystalline	Amorphous
PANEL SPEC	Module Efficiency (lab test)	12.11%	13.7%	8.10%
	Maximum Power Output	80 W	180 W	115 W
	Dimension (WxHxD)	545 x 1214 x 35 mm	1,318 x 994 x 46mm	1,009 x 1,409 x 46 mm
	Front Cover Material	Low iron temperd glasses	white tempered glass	Cover glass-less
	Type of Cell	Square 126 mm, Poly Crystal	Monocrystalline silicon solar cells, 155.5mm square	a-Si/μc-Si (Tandem)
	Weight	9 Kg	16.0kg	19kg
	Open Circuit Voltage : Voc	21.6 V	30 V	238 V
	Short Circuit Current : Isc	5.15 A	8.37 A	0.810 A
	Max Power Voltage : Vpm	17.3 V	23.7 V	174 V
	Max Power Current : lpm	4.63 A	7.6 A	0.661 A
	Price			
	Supplier	SHARP (NE-080T1J)	SHARP (NUS0E3E)	SHARP (NA-F115A5)
INSTALLED	Maximum Power Output Installed	160 W	180 W	230 W
	Area of Modul Installed, Pmp (m^2)	1.32m^2	1.31m^2	2.84m^2
	Number of Panel Installed	2 panel	1 panel	2 panel

There are 4 parameters were collected from each of the photovoltaic module for each stage of this experiment, that is Current (A), Voltage (V), Module temperature (^{o}C) and Power (W). While parameters to determine the environmental condition that influences the performance of photovoltaic module is Solar Radiation (W/m^2) and Ambient Temperature (^{o}C). Power output efficiency, average daily module efficiency and performance ration have been used to measure the performance of each type of the panels installed by using the equation below:

$$\eta_p = (P_{mea} / P_{max}) \times 100\%$$

η_p	Power output efficiency (%)
P_{mea}	Average power output (W) measured on site in the given period.
P_{max}	Maximum power output (W) of panel.

$$\eta_{eve} = \frac{P_{eve \cdot mea}}{PV_A . H} \text{ X } 100\%$$

η_{eve}	Average module efficiency (%)
$P_{eve \cdot mea}$	Average power output (W) measured on site in the given period.
H	Average incident radiation (W/m2) on site in the given period.
PV_A	Surface area (m^2)

$$PR = \frac{P_{eve.mea}}{P_{max}} \Big/ \frac{H}{G_{STC}}$$

PR	Performance Ratio
$P_{eve \cdot mea}$	Average power output (W) measured on site in the given period.
P_{max}	Maximum power output (W) of panel.
H	Average incident radiation (W/m2) on site in the given period.
G_{STC}	Irradiance at STC (W/m^2) = $1000W/m^2$

3. RESULT AND DISCUSSION

The performance of each solar module cannot be obtained by observing the power output of the module because each type of photovoltaic module has different efficiency, Therefore, the power output efficiency have to be normalize to the ratings of each module should be use in order to compare them. The output efficiency will show how much power is actually generate at a specific time over the installed capacity of the module. (N.Amin et al, 2009)

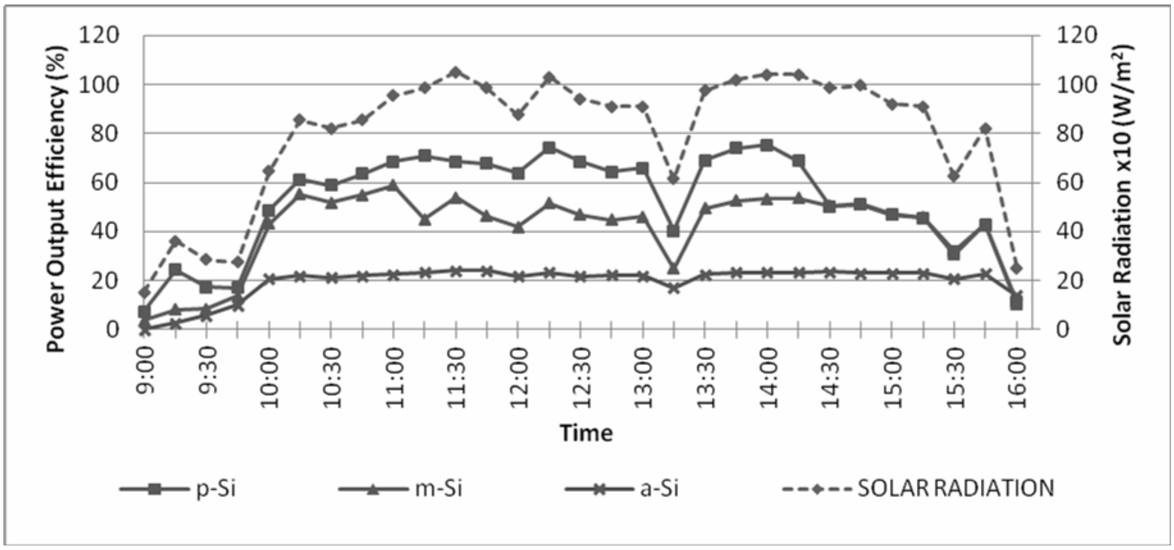


***Figure 2**. Power Output Efficiency, Solar Radiation – Time for day 1.*

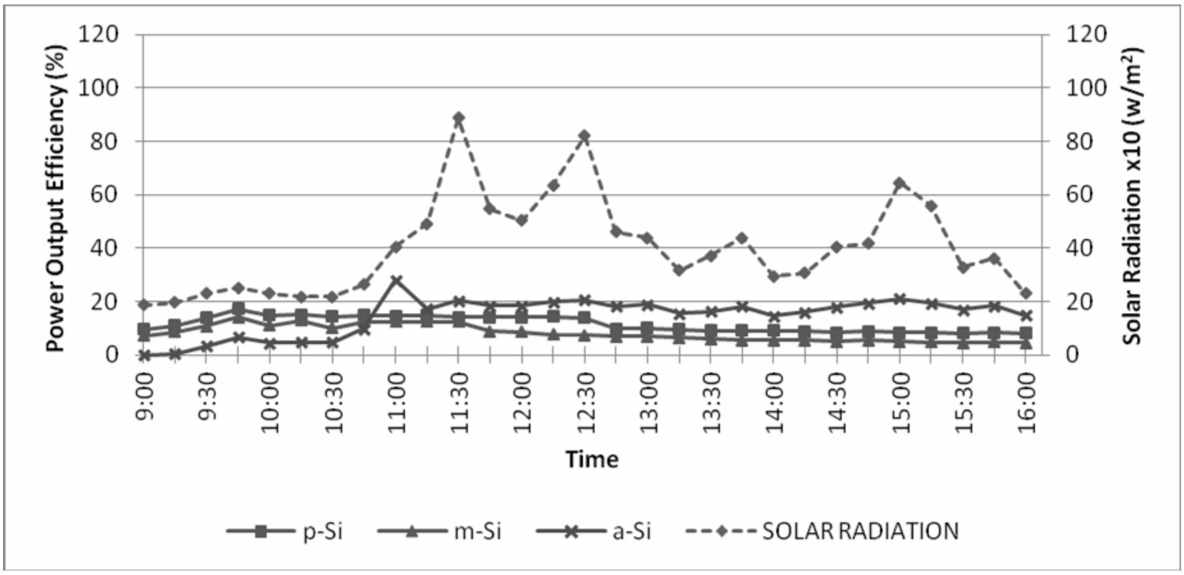


***Figure 3**. Power Output Efficiency, Solar Radiation – Time for day 2.*

Figure 2 shows the normalized power output efficiency for day 1 with an average solar radiation 796.16W/m^2. With this amount of average solar radiation per day, it was consider a sunny day when most of the time the solar radiation are above 600W/m^2. It can be seen that, poly-crystalline panel produced more power than mono-crystalline and amorphous panels in high insolation. Poly-crystalline and mono-crystalline power outputs are inclining almost at the same rate as solar radiation, while amorphous power output is at low level almost at the same level from 10:30 to 15:15. The average performances of poly-crystalline, mono-crystalline and amorphous on day 1 are 52.35%, 40.90% and 19.65% (respectively).

Meanwhile in Figure 3, average solar radiation in day 2 was below 600W/m^2 and it was considered a cloudy day. By looking at the solar radiation trend line, the amount of solar radiation was fluctuated and slowly decreased between 11:30 to15:00. The result of power output efficiency in day 2 shows that amorphous silicon cell created more energy than poly-crystalline and mono-crystalline cell with average power output efficiency 14.63% for amorphous, 11.58% for poly-crystalline and only 8.09% for mono-crystalline panel.

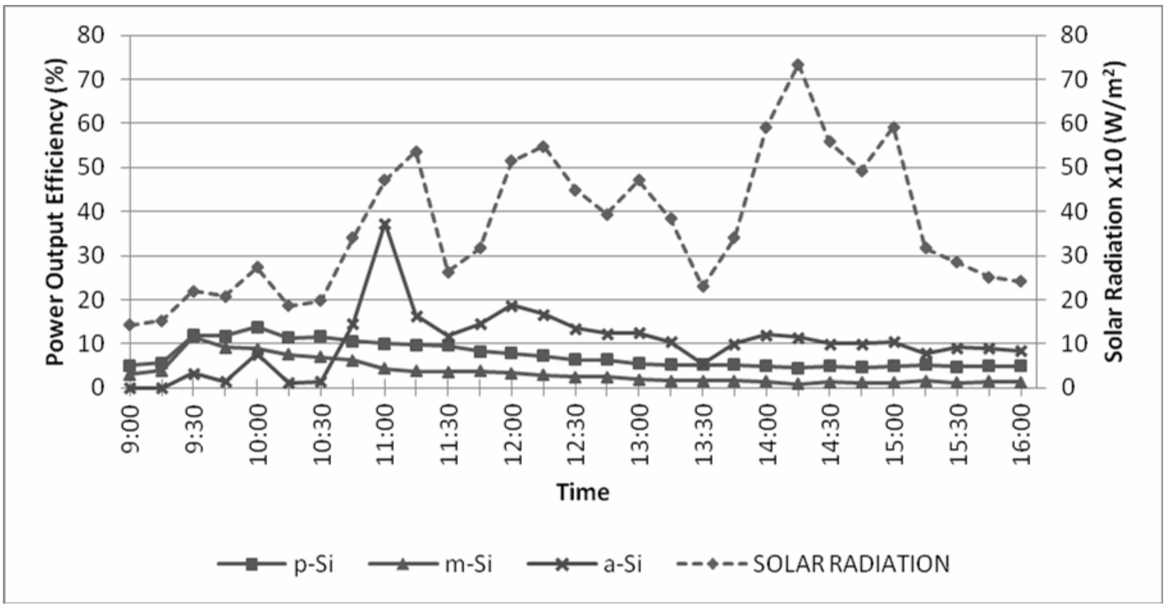


***Figure 4.** Power Output Efficiency, Solar Radiation – Time for day 3.*

The same trends also occur in day 3 regarding the power output efficiency for each type of photovoltaic panels. Figure 4 shows, that amorphous panel (10.28% in average) produced higher power output compared to poly-crystalline (7.33%) and mono-crystalline panels (3.53%). The average of solar radiation is also below 600W/m^2, which is considered a cloudy day. From the graph it can be seen that, in morning hour before 10:45, the power output of poly-crystalline was much higher than mono-crystalline and amorphous panel, but after 10:45 the trend was reversed. Amorphous power output increased gradually while poly-crystalline and mono-crystalline panels decreased slowly toward that evening.

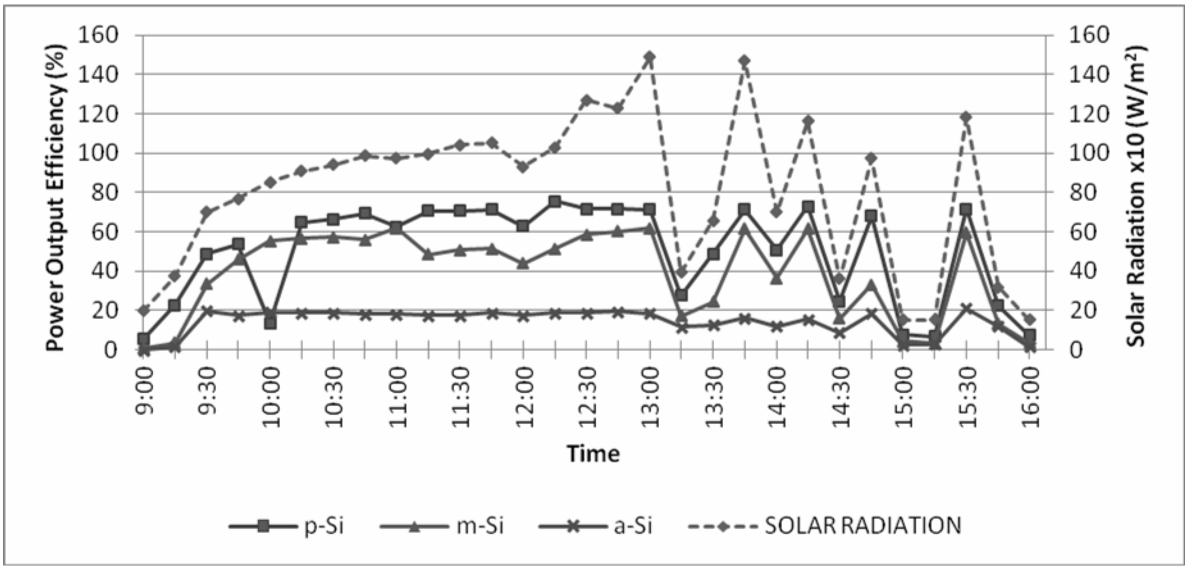


Figure 5. *Power Output Efficiency, Solar Radiation – Time for day 4.*

By observing the graph in Figure 5, it can be seen that the solar radiation in day 4 were very high most of the time with average 807.49W/m^2 and it was increase gradually from 9:00 until 13:00, but after 13:00 the solar radiation level was fluctuated until 16:00 due to the present of scattered cloud cover. The trend line of solar radiation also affected the power output efficiency for poly-crystalline and mono-crystalline. During day 4, poly-crystalline panel produced higher power output with average efficiency of power output 50.08%, while mono-crystalline and amorphous (39.02% and 14.28% of average power output efficiency, respectively).

Efficiency in photovoltaic solar panels is measured by the ability of a panel to convert sunlight into usable energy for human consumption. Knowing the efficiency of a panel is important in order to choose the correct panels for photovoltaic system. The panel efficiency determines the power output of a panel per unit of area. Figure 6 shows the bar graph for daily module efficiency for each type of solar panels by using single axis time/date solar tracker in 4 days period. It can be seen that, the daily module efficiency for poly-crystalline module is the highest compare to mono-crystalline and amorphous solar module in 4 days period with efficiency 7.97% (day 1), 3.49% (day 2), 2.41% (day 3) and 7.52% (day 4). However there are differences in module efficiency between mono-crystalline and amorphous solar module. As shows in graph, mono-crystalline module efficiency in day 1 (7.06%) and day 4 (6.64%) shows a significant higher efficiency compared to amorphous, which is 2.00% and 1.43%, respectively. While in day 2 and day 3, amorphous solar module shows a slightly better efficiency than mono-crystalline solar module.

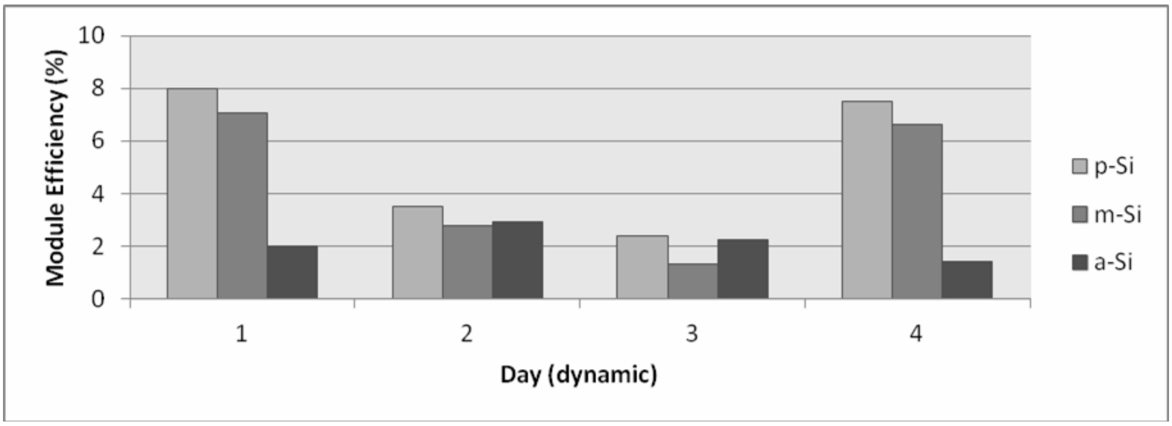


Figure 6. *Module efficiency – Day.*

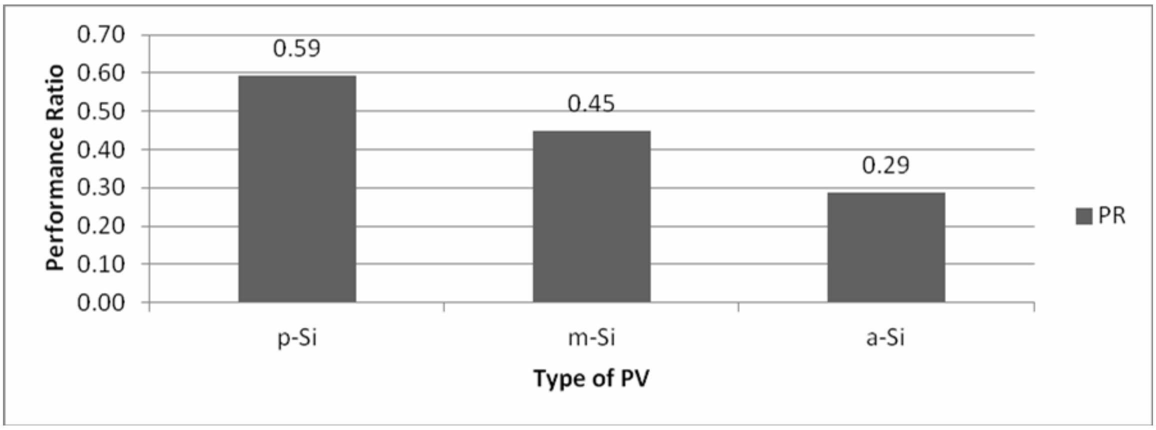


Figure 7. *Performance ratio of each type of photovoltaic panel using single-axis time/date solar tracker.*

The performance ratio is one of the most important variables for evaluating the efficiency of a PV. The performance ratio is the ratio of the actual and theoretically possible energy outputs. It is largely independent of the location of a PV being measure and the incident solar irradiation on the PV. Figure 7 shows the Performance Ratio of each type of photovoltaic panels in this experiment by using single-axis time/date solar tracker. From the graph, it can be seen that, the Performance Ratio for poly-crystalline solar module is 0.14 higher compare to mono-crystalline. While Performance Ratio for amorphous solar module is the lowest. From this experiment, it can be conclude that poly-crystalline solar module are most suitable type of photovoltaic module to be used under Malaysian climate condition when applying single-axis time/date solar tracker.

4. CONCLUSIONS

Developing a clean and renewable energy helps Malaysia safeguard its depleting energy resources. With the introduction of photovoltaic system and Malaysian climate condition which is almost predictable with the availability of 6 hours of direct sunlight per day with average solar radiation between 800 W/m^2 to 1000 W/m^2, provide a suitable condition to consider solar power as a promising renewable energy for Malaysia. Poly-crystalline, mono-crystalline and amorphous are types of PV panels most commonly use in Malaysia with different characteristic and efficiency. In this research, a solar tracker device (single-axis time/date) are being used to maximize the usage of sunlight in morning and afternoon performance to create more efficient solar energy by oriented the PV panels towards the sun's rays which incident on them from the normal direction. Poly-crystalline solar cell performed better in high level of solar radiation per day compared to amorphous and mono-crystalline. Meanwhile, during low intensity of solar radiation, amorphous solar cells efficiency is higher than the other two types of PV panel.

In built environment, by replacing the conventional type of solar installation with solar tracking installation, it's not only increase the performance of the PV panels installed but also, it can be integrate to the overall building design as part of Building Integrated Photovoltaic (BiPV) to perform specific function to the building envelope (e.g., provide shades on top of parking lot, minimize direct heat from sun at roof top, minimize direct sunlight for natural lighting). Besides that, the quantity of PV panels installed at the building can be reduced by using the solar tracking installation compare to the conventional type of installation (fixed installation). Hence these will influence the cost of the overall construction of the building. The combination of new green technology and well-design building will provides opportunities for sustainable building that not only energy efficient but also create a better environment.

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FINITE ELEMENT MODELLING OF RC BEAMS WITH LARGE OPENING AT CRITICAL FLEXURE AND SHEAR STRENGTHENED WITH CFRP LAMINATES

Chin S.C[1], Shafiq N.[2] and Nuruddin M.F.[3]

[1, 2, 3]Universiti Teknologi Petronas, Perak, Malaysia

[1]brigitchin@gmail.com, [2]nasirshafiq@petronas.com.my, [3]fadhilnuruddin@petronas.com.my

ABSTRACT: Pipes and ducts are provided to accommodate essential services that are required in a building, namely power supply, sewerage, air-conditioning, water supply, telephone cable and computer network. Traditionally, pipes and ducts are usually hung below the beams and formed a “dead space” in a building. An option is to pass these pipes and ducts through openings in reinforced concrete (RC) beam. In the design stage, design engineers provide sufficient strength and ensure serviceability in beams with opening. However, M&E contractors usually request to drill an opening in order to simplify the pipe arrangements during construction stage which were not considered in the design. This causes problem to structural engineers especially when the required opening is located at critical section in an existing beam. This paper presents a two dimensional non-linear finite element analysis to study the behaviour of externally strengthened RC beams with large opening at section with critical flexure and shear using carbon fiber reinforced polymer (CFRP) laminates. The numerical results were validated with the experimental testing of nine beams which included a control beam, four un-strengthened and four strengthened beams. The numerical modelling was conducted using a non-linear finite element (FE) program, ATENA. In this FE program, tensile behaviour of concrete is modelled by nonlinear fracture mechanics combined with the crack band method in which smeared crack concept is adopted. All the beam specimens had a cross section of 120x300 mm with an effective span of 1800 mm. The beams were loaded to failure under four point bending. The parameters included the size, location and shape of opening. The crack patterns, load deflection curves at mid-span and ultimate load were compared with the experimental results. Generally, good agreement has been obtained between the numerical analysis and experimental results. The finite element analysis can be useful to design a suitable strengthening configuration for RC beams with large openings at critical location by optimal application of CFRP laminates.

Keywords: CFRP Laminates, Finite Element Modelling, Large Opening, RC beams

1. INTRODUCTION

Utility pipes and ducts are provided to accommodate essential services in a building namely electricity cable, sewerage, air-conditioning, water supply, telephone cable and computer network. Openings are provided in RC beams to allow for the passage of pipes and ducts to eliminate dead space and produce a more compact and economical design. However, providing openings in a RC beam changes the simple beam behaviour to a more complex behaviour. The presence of openings in the web of a RC beam reduces beam stiffness, cause excessive cracking and deflection and severely affects the strength of the beam (Mansur & Tan, 1999). Thus, sufficient strength and serviceability provision need to be ensured by design engineers during the design stage. In order to reduce cost and time, M&E contractors often prefer to simplify the pipe and ducts arrangements during construction. Problems arise when M&E contractors request to create an opening at a location in the existing structure which is not considered in the design.

External strengthening with fiber reinforced polymer (FRP) laminates is well known as external reinforcement for the strengthening or rehabilitation of reinforced concrete structures. Numerous experimental investigations have shown that externally bonded FRPs can be used to improve flexural and shear capacities, significantly increase the stiffness and load carrying capacity of a member (Wong & Vecchio, 2003). FRP laminates have many advantages including excellent corrosion resistance, high strength-to-weight ratio, good fatigue properties and low in weight which ease the installation process and time saving. The most common types of FRP available in the concrete industry are made with carbon, aramid or glass fibers. The FRPs are usually in the form of sheets or laminates. These materials were applied by bonding it to the external surfaces of the beams with different configurations and layouts (Allam, 2005; Madkour, 2009).

Various investigations (Li et al., 2001; Allam & Ebeido, 2003; Islam et al., 2005) have been carried out by using FRP laminates as external strengthening at flexure and shear in reinforced concrete beams. However, very little research effort (Mansur et al., 1999; Abdalla et al., 2003; Allam, 2005; El Maaddawy & Sherif, 2009) focuses towards the study of strengthening of beams provided with openings. Also, little attention has been paid to web opening strengthening in beams in terms of numerical approach (Pimanmas, 2010). According to literature, it is essential to investigate and predict the complex behaviour of the strengthened reinforced concrete structure before and after using FRP with the usage of highly sophisticated numerical model (Madkour, 2009).

In this paper, a two dimensional non-linear finite element analysis was conducted to study the behaviour of strengthened RC beams with large circular and square openings at critical flexure and shear sections using carbon fiber reinforced polymer (CFRP) laminates. The crack patterns of the four un-strengthened beams were analyzed and a suitable strengthening configuration for each beams were designed and modelled in the numerical modelling. An experimental study was performed to validate the modelling results. The main investigated parameters were shape, size and location of opening.

2. NONLINEAR FINITE ELEMENT ANALYSIS

A two dimensional (2D) nonlinear FE analysis was carried out using ATENA, a nonlinear FE program developed by Červenka Consulting (V. Červenka et al., 2010). In the FE program, concrete was represented by the SBETA material model. The tensile behaviour of concrete is modelled by nonlinear fracture mechanics combined with the crack band method, whereby the smeared crack concept is adopted. The rotated crack model was used in this study (Franca et al., 2007). The beam specimens were modelled in full scale mode. Stirrups, reinforcement bars and CFRP laminates were represented by bar reinforcement elements. Meshing was performed using quadrilateral "CCIsoQuad" element type. Newton-Raphson solution method was used to solve for the nonlinear behaviour of the full scale beams (V. Červenka, 1999; J. Červenka, 2001; V. Červenka et al., 2002; V. Červenka & J. Červenka, 2006).

2.1 Numerical Modelling

The FE modelling was conducted on simply supported RC beams with dimensions of 120x300x2000 mm. The reinforced concrete beams were comprised of (i) a control beam, (ii) four un-strengthened beams with large circular and square openings in flexure and shear location and (iii) four strengthened beams with large circular and square openings in flexure and shear location with CFRP laminates. The openings provided were circular and square with the size of ø230 mm and 210x210 mm respectively. The openings were placed at a critical location; zero distance from the support at both ends for shear. Meanwhile, the opening was placed at mid-span of the beam for flexure. The loads were applied to a spreader beam that distributed to two point loads which located symmetrically.

3. RESULTS AND DISCUSSIONS

Finite element modelling was conducted onto control beam and four un-strengthened reinforced concrete beams with large openings. The results of FEM were compared with the experimental results. Crack pattern results of un-strengthened beams with large openings at flexure and shear were presented and discussed. The un-strengthened beams with large circular and square openings at flexure respectively were known as "C-flexure" and "S-flexure" while beams with large circular and square openings at shear respectively were "C-shear" and "S-shear".

3.1 Control beam

The comparison of crack patterns is shown in Figure 1(a). A good match of crack patterns was observed. The analysis predicts several flexural cracks at the mid-span and large shear cracks were found at shear similar as the experiment result. The load deflection curve is illustrated in Figure 4(a). It was observed that the load deflection curve trend of FEM matches well with the experimental result.

3.2 Un-strengthened beams

a. *Beam C-flexure*

The crack patterns of beam C-flexure was compared in Figure 1(b). From the FEM simulated results, it was observed that stress was highly concentrated around the opening provided with concentrated flexural cracks formed at the bottom chord of the opening and along the mid-span. The cracks eventually appeared as diagonal shear cracks formed from the point load towards the support. A good agreement with the experimental crack patterns was observed.

b. *Beam S-flexure*

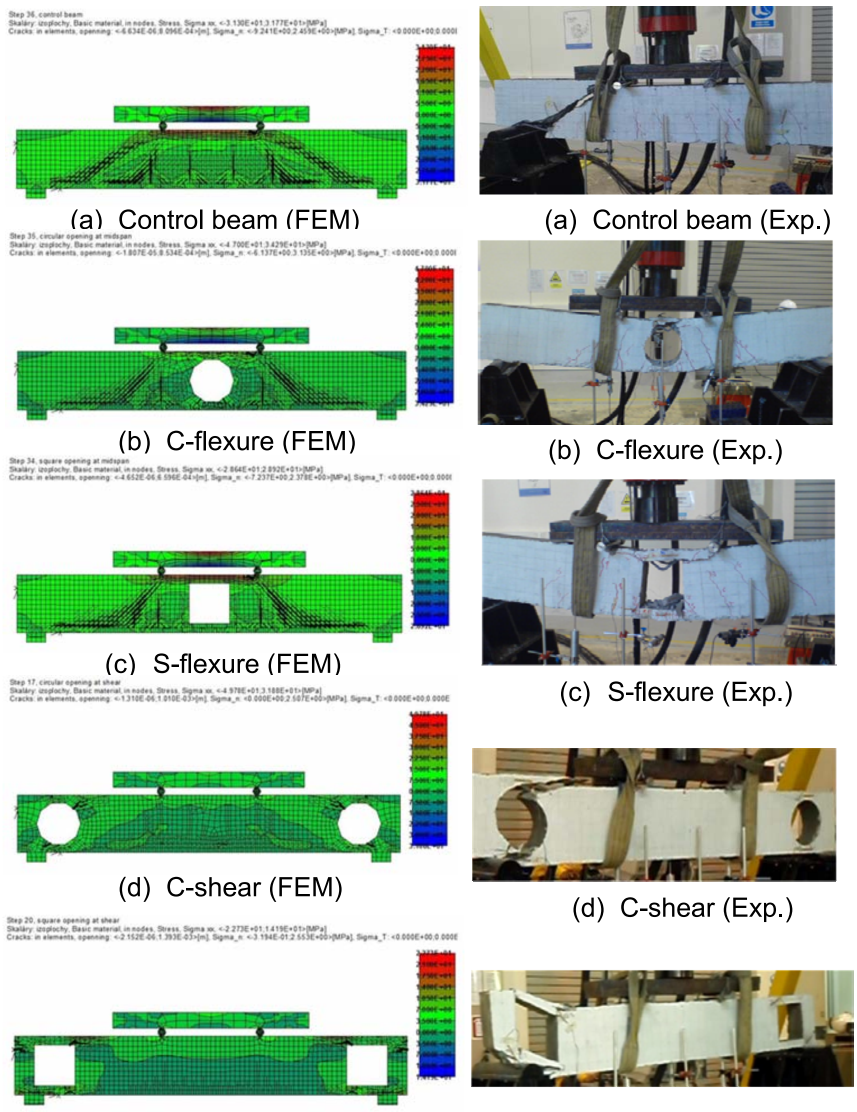
The FEM crack pattern results of beam S-flexure were compared and shown in Figure 1(c). Concentrated stress were seen at the top chord above the square opening in FEM, between two point loads, similarly in the experimental result. Cracks were found at the bottom chord of the opening and eventually diagonal shear cracks were formed. The crack patterns in FEM matches well with the experimental results.

c. *Beam C-shear*

The crack pattern results of beam C-shear in FEM were compared to the experimental results as shown in Figure 1(d). It was observed that the FEM simulated cracks penetrated from an angle diagonally through the opening and to the support, at both the circular openings at shear. The crack pattern of FEM is in good agreement with the experimental results.

d. *Beam S-shear*

The crack pattern of FEM and experimental results of beam S-shear is shown in Figure 1(e). It was found that the FEM predicted cracks were first initiated at four corners of the square opening and propagated through the top and bottom chord. Similarly in the experimental results, the cracks caused crushing of concrete at the top and bottom chord of the opening. The crack pattern of FEM matches well with the experimental crack pattern.



(a) Control beam (FEM) (a) Control beam (Exp.)

(b) C-flexure (FEM) (b) C-flexure (Exp.)

(c) S-flexure (FEM) (c) S-flexure (Exp.)

(d) C-shear (FEM) (d) C-shear (Exp.)

(e) S-shear (FEM) (e) S-shear (Exp.)

Figure 1. *Crack pattern results of un-strengthened beams in FEM and experiment*

3.3 Strengthened beams

Based on the crack pattern results, a suitable strengthening configuration was designed for "C-flexure", "S-flexure", "C-shear" and "S-shear". The strengthening configuration for each beam is shown in Figure 2. In this paper, the strengthened RC beams with large circular and square openings in flexure using CFRP laminates were known as "C-flexure cfrp" and "S-flexure cfrp"; while the strengthened beams with large circular and square openings in shear were "C-shear cfrp" and "S-shear cfrp". The results of crack pattern and load deflection curve of the strengthened beams with FEM and experiment were presented and discussed. The results of FEM were compared with the experimental results.

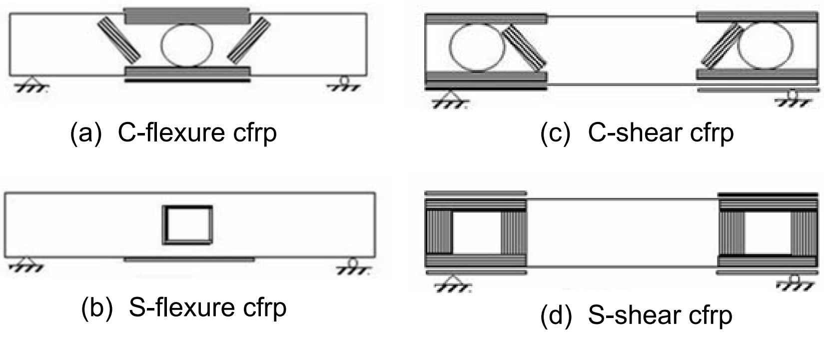


Figure 2. *Strengthening configuration of beams with large openings in flexure and shear*

a. *Beam C-flexure cfrp*

The beam C-flexure cfrp was strengthened externally by CFRP laminates around the opening with the design configuration shown in Figure 2(a). A comparison between FEM and experimental crack patterns are shown in Figure 3(a). It was observed that the failure in the experiment was initiated from the point load penetrated diagonally towards the support away from CFRP laminates. Similar to the experiment, the FEM predicted that the diagonal cracks propagated from the point load towards the support. This is due to CFRP laminates disturb the path of crack propagation which required a higher energy to extend the crack in a diverted direction. The comparison of FEM and experimental load deflection curve is illustrated in Figure 4(b). The load deflection curve of control beam is shown in Figure 4(a) for comparison purposes. With the strengthening configuration, both FEM and experimental beam have significantly restored the control beam original capacity. However, both ultimate load of FEM and experimental beam exceeded the original beam capacity, about 52% and 42% respectively.

b. *Beam S-flexure cfrp*

Beam S-flexure cfrp was strengthened by placing the CFRP laminates inside the square opening and at the tension zone of the beam. The strengthening configuration is shown in Figure 2(b). The comparison of crack pattern is shown in Figure 3(b). FEM predicted large diagonal shear cracks formed from the point load to the area without the existence of CFRP laminates and predicted wide cracks to appear at the edge of CFRP laminates located at the tension zone. Similarly, in the experimental results, the beam failed with wide diagonal cracks formed from the point load and peeling of concrete with CFRP at the tension zone was observed. The FEM also predicted that the stress were concentrated at the top chord above the opening which caused crushing of concrete in the experimental program. The crack pattern of FEM is in good agreement with the experimental results. The results of load deflection curve are compared in Figure 4(c). It was observed that the load deflection curve of FEM was almost similar to the load deflection curve of experiment, with a difference of 7%. Comparing to the load deflection curve of control beam in Figure 4(a), strengthening with the design configuration in the experiment was able to re-gain 74% of the control beam original capacity.

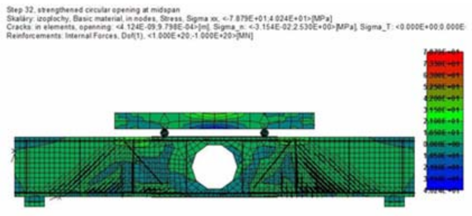
c. *Beam C-shear cfrp*

CFRP laminates were placed around both circular openings at shear location at the support according to the design strengthening configuration as shown in Figure 2(c). The comparison of FEM and experimental results is illustrated in Figure 3(c). As simulated by FEM, diagonal cracks appeared at the top chord above the circular opening and at the bottom chord near the support. Stress was concentrated at the diagonal cracks at the top and bottom of openings while concentrated flexural cracks were observed along the mid-span of the beam. As predicted in FEM, the failure of experimental beam was horizontal crack near the point load which propagated through the opening at the top chord and a diagonal crack was observed from the opening towards the support. The diagonal crack eventually turned into a horizontal crack due to the resistance of CFRP laminates at the top chord. The formation of diagonal crack at the bottom chord caused peeling of CFRP laminates. The flexural cracks at the mid-span of the experimental beam were not significant. The comparison of FEM and experimental load deflection is shown in Figure 4(d). It was found that the ultimate load of FEM almost double the experimental result. The possible reason may due to the mix mode of shear and flexure failure in FEM as massive flexural cracks were observed, as shown in Figure 3(c). In the experiment, strengthening with the design configuration was only able to re-gain 35% of the control beam original capacity.

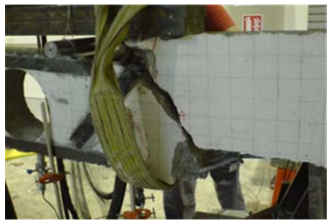
d. ***Beam S-shear cfrp***

The strengthening configuration designed to strengthen beam with large square openings at shear located at the support is illustrated in Figure 2(d). The CFRP laminates were wrapped around both the square openings. The crack pattern results of FEM and experimental results were compared and illustrated in Figure 3(d). Based on the FEM results, horizontal cracks were seen internally at the top chord near the point load and at bottom chord near the support. It was observed that minor flexural cracks formed at the mid-span of the beam. In the experimental results, the beam failed in a different mode compared to the un-strengthened square opening, S-shear. With the presence of CFRP around the opening in this strengthening configuration, the beam failed at a weaker point at a connection between two CFRP laminates, as shown in Figure 3(d) S-shear (Exp.). The beam failed with a diagonal crack penetrated near the point load to the corner of the square opening and crushing of concrete with CFRP laminates at the bottom chord of the same square opening at shear, as predicted by FEM. Similar in the FEM, minor cracks were observed in the mid-span of the beam. The comparison of FEM and experimental results of load deflection is illustrated in Figure 4(e). From the figure, it was observed that the curve trend of FEM was almost similar to the curve trend of experiment. With the design strengthening configuration, the simulated beam could achieve about 57% of the simulated control beam capacity while in experiment; the beam could only re-gain 33% of the control beam original capacity.

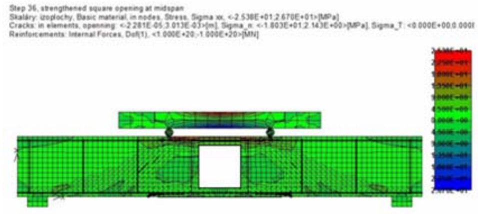
In general, the load deflection curve of FEM in all the beams showed a higher stiffness compared to the load deflection curve of the experimental beam. The possible reasons may be due to the highly heterogeneous nature of the concrete and the inaccurate position of the steel reinforcement bars in the experimental program (Franca et al., 2007).



(a) C-flexure cfrp (FEM)

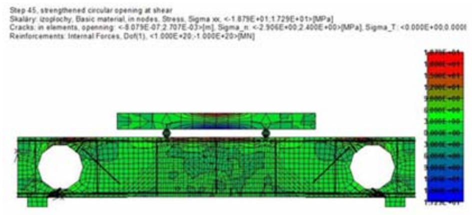


(a) C-flexure cfrp (Exp.)



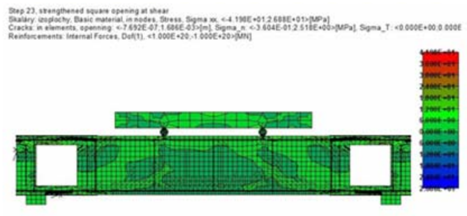
(b) S-flexure cfrp (FEM)

(b) S-flexure cfrp (Exp.)



(c) C-shear cfrp (FEM)

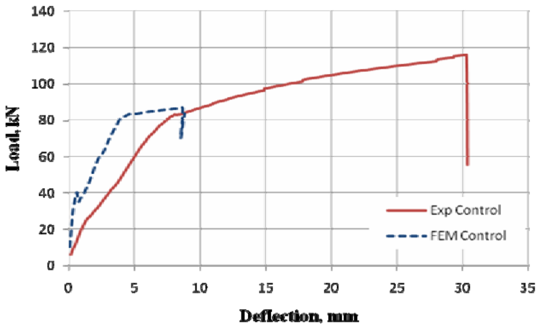
(c) C-shear cfrp (Exp.)

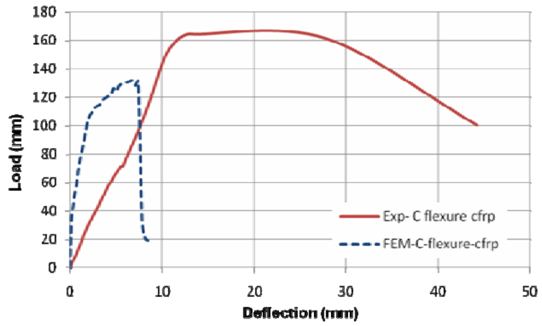


(d) S-shear cfrp (FEM)

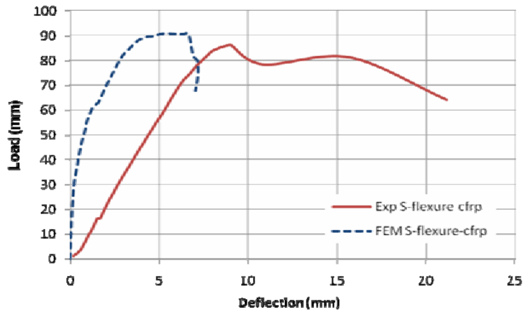
(d) S-shear cfrp (Exp.)

Figure 3. *Crack pattern results of strengthened beams in FEM and experiment*

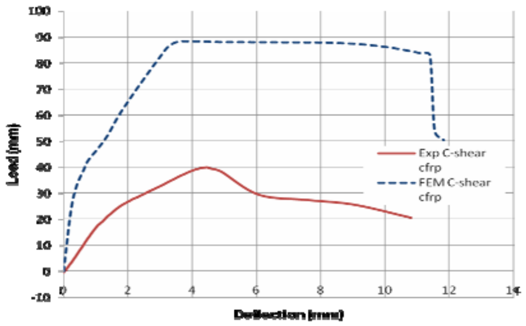




(b) C-flexure cfrp



(c) S-flexure cfrp



(d) C-shear cfrp

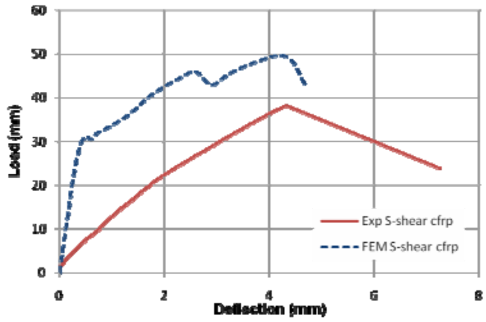


Figure 4. *Comparison of load deflection curves between finite element and experimental control and strengthened beams*

4. CONCLUSIONS

Based on results and discussion, the following conclusions are made:

1. The strengthening behaviour of RC beams with large opening at critical flexure and shear using CFRP laminates was modelled by a non-linear finite element programme, ATENA. The crack pattern results were found in good agreement with the experimental results.
2. Strengthening of large circular and square openings in flexure zone could significantly restore 74-100% of the beam original capacity, while strengthening of large circular and square openings in shear zone could only re-gain the beam capacity to about 33-35% of the original beam strength.
3. The beam with large circular opening in flexure zone with the design strengthening configuration could significantly restore 100% of the beam original capacity compared to beam with large square opening in flexure, which could only re-gain 74% of the control beam capacity.
4. The strengthened beams with large circular and square openings located at both critical shear locations at the support could not restore the control beam original capacity.
5. The load deflection curve of FEM simulated beams were almost similar as

load deflection curve of tested beams with a slightly higher in stiffness.

6. Optimum strengthening can be designed using FE simulation to achieve a beam capacity similar to the control beam with the optimal usage of CFRP.
7. The modelling that was simulated by ATENA non-linear finite element program can be useful as it is time saving and cost effective compared to experimental work. In addition, the errors during the testing can be prevented.

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PRELIMINARY STUDY ON STRUCTURAL BEHAVIOUR OF PRECAST LIGHTWEIGHT FOAMED CONCRETE SANDWICH PANEL

N. Mohamad[1], A.A.A Samad[2], W. Adnan[3]

[1,2,3]Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn, Parit Raja, 86400 Batu Pahat, Johor.

[1]noridah@uthm.edu.my

ABSTRACT: Affordable quality housing is vital in developing countries to meet its growing population and demands arising from migration of people to urban areas. Development of a new cost effective system is crucial to fulfill these demands. In view of this, a study is carried out to develop a Precast Lightweight Foamed Concrete Sandwich Panel (PLFP), as a new affordable building system. Preliminary experimental investigation to study the structural behaviour of the PLFP panel under axial load is undertaken. Six number of panels (PLFP-1 to PLFP-6) consist of two lightweight concrete wythes and a polystyrene insulation layer in between the wythes are cast. Four panels are reinforced with 6 mm mild steel BRC with 200mm x200 mm openings and two panels are reinforced with 9mm diameter high yield steel bars. The reinforcements of both wythes in each panel are tied to each other through the insulation layer by shear connectors of 6mm and 9 mm mild steel bars bent to an angle of 45º. Panels PLFP-5 and PLFP-6 are capped with normal concrete at its top and bottom end to avoid end crushing during axial loading. Axial load test was conducted and the results are presented here, which include the crack pattern and mode of failure, ultimate loads achieved, and load-deflection profiles. It is observed that the strength of the panels are affected by the compressive strength of the foamed concrete forming the wythes and the presence of concrete capping at panel ends. The results also indicate that wythes in panels with capping at both ends tend to deflect together in the same direction.

Keyword: Shear connector, ultimate strength, load-deflection curve, strain distribution, crack pattern, compositeness

1. Introduction

Precast concrete sandwich panel technology has advanced gradually over the past four decades. This type of panels have been in use for more than 40 years in North America. Prior to 1960, sandwich technology had been confined almost entirely to aerospace applications. By 1960 increasing numbers of alternative uses were discovered, such as in the building, refrigerated storage, automobile and shipbuilding industries. This period was also the beginning of a worldwide boom in prefabricated building elements for diverse applications. (Davies, 1997)

The aim of this study is to develop a load bearing Precast Lightweight Foamed Concrete Sandwich Panels (PLFP) for use as structural component in low-rise building construction. In order for this type of panel system to be viable as an Industrial Building System (IBS), it must satisfy all requirements such as strength and integrity under imposed loads for the intended storey heights such as its ultimate strength capacity and its composite action. Factors that contribute to the strength and composite action are investigated which include the wall slenderness ratio and the shear connector's rigidity. The degree of compositeness achieved is determined from the capacity of the shear connectors to transfer the applied load which depends on the diameter of the connector used and its orientation.

2. Precast Lightweight Foamed Concrete Sandwich Panel (PLFP)

The PLFP panels tested consist of two wythes which are made of foamed concrete and polystyrene as a core. The thickness of each concrete wythe is fixed at 40 mm for all panels. The gap between the concrete wythes had been varied from 20 mm to 70 mm to obtain a variation of overall wall thickness from 100 mm to 150 mm. Square welded mild steel BRC was used as the longitudinal and transverse reinforcement for both wythes in all panels. Steel shear connectors are tied up to the BRC in each wythe to strengthen the panel. A typical PCSP with a truss type shear connectors is shown in Figure 1. To take full advantage of the strength of the two wythes and to prevent individual wythe buckling, the shear connectors should be designed to provide for full shear transfer between the two concrete wythes. The test carried out on connector's orientation by Pfeifer and Hanson (1964) showed that the diagonal orientation was the most efficient orientation of the shear connectors in taking up the applied load. Therefore, the diagonal orientation of the shear connectors is adopted in the experiment.

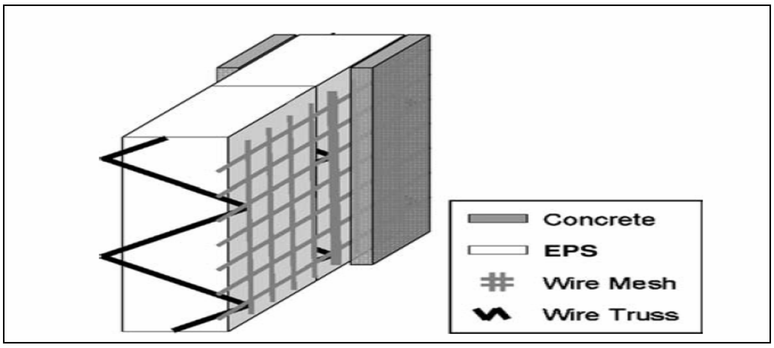


Figure. 1. *Sandwich precast concrete with truss shaped connectors (Lee et al., 2006)*

Generally there are three types of PCSP panels. (Shutt ,1997). A non-composite sandwich panel is one in which each concrete wyhte acts independently to resist bending. Plane section behaviour is obtained in each wythe, but not through the entire panel depth. A fully composite sandwich panel is one in which the two concrete wythes act integrally to resist bending allowing the entire panel to perform as a single unit. In theory, a fully composite panel exhibits plane section behaviour throughout its entire depth at all locations along its span. Full composite action is achieved by providing sufficient horizontal shear transfer between the wythes.

A partially composite sandwich panel is one in which concrete wyhtes act atleast partly together to resist bending. Thus, a partially composite panel resists bending to a degree between that of a fully composite panel and a non-composite panel. The degree of composite action exhibit by a panel may change throughout the loading history of the panel. The distribution of stress and strain for three types of sandwich wall panel is shown on Figure 2.

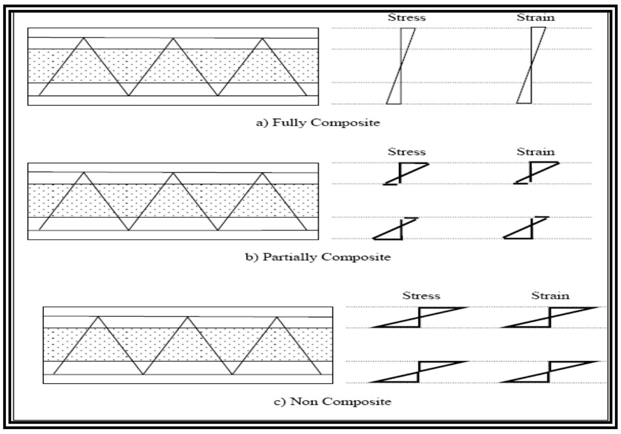


Figure 2. *Stress and Strain Distribution Of Sandwich Panel (Shutt, 1997)*

3. Foamed Concrete

Foam concrete is a lightweight material which has been widely used especially in the western countries. Nowadays, foam concrete technology has been used extensively in construction industries especially for non-structural building wall panels or as partitions. It is considered as an attractive material for its lightweight, better thermal properties and ease of construction. Based on research carried out at British Cement Association, an indication on properties of foamed concrete at various dry densities is as shown in Table 1. The drying shrinkage for foamed concrete is higher than that of dense concrete. As Table 1 shows, the thermal conductivity is between 5% and 30% of that for dense concrete. The modulus of elasticity is also of a similar order when compared with dense concrete

Table 1. *Properties of Foamed Concrete*

Dry Density (kg/m^3)	Compressive Strength (N/mm^2)	Thermal Conductivity (W/mK)	Modulus of Elasticity (kN/mm^2)	Drying Shrinkage (%)
400	0.5-1.0	0.10	0.8-1.0	0.30-0.35
600	1.0-1.5	0.11	1.0-1.5	0.22-0.25
800	1.5-2.0	0.17-0.23	2.0-2.5	0.20-0.22
1000	2.5-3.0	0.23-0.30	2.5-3.0	0.18-0.15
1200	4.5-5.5	0.38-0.42	3.5-4.0	0.11-0.09
1400	6.0-8.0	0.50-0.55	5.0-6.0	0.09-0.07
1600	7.5-10.0	0.62-0.66	10.0-12.0	0.07-0.06

4. Experimental Programme

The experimental programme includes six PLFP specimens with various height and thickness under axial load. All panels tested consist of two wythes which are made of foamed concrete and polystyrene as a core. The thickness of each concrete wythe is fixed at 40 mm for all panels. The gap between the concrete wythes had been varied from 20 mm to 70 mm to obtain a variation of overall wall thickness from 100 mm to 150 mm. Square welded mild steel BRC was used as the longitudinal and transverse reinforcement for both wythes in all panels. Steel shear connectors are tied up to the BRC in each wythe to strengthen the panel. The slenderness ratio, H/t, was in the range of 13.33 to 20. Each specimen was designated by a name, i.e. PLFP-1 to PLFP-6. Further details of all the PLFP specimens with their designation, compressive strength and slenderness ratio are tabulated in Table 2

Table 2. *Dimension and Properties of Pilot Test Specimens*

Specimen	H x W x t	f_{cu}	H/t	t_1	t_2	c	Reinforcement	Shear
PLFP-1	2000x750x125	7	16	40	45	15	6mmΦ@200mm c/c	R6
PLFP-2	2000x750x150	6.5	13.33	40	70	15	6mmΦ@200mm c/c	R6
PLFP-3	2000x750x100	4	20	40	20	15	6mmΦ@200mm c/c	R6
PLFP-4	1800x750x100	5	18	40	20	15	6mmΦ@200mm c/c	R6
PLFP-5	2000 x 750 x 125	8	16	40	45	15	6mmΦ@200mm c/c	R6
PLFP-6	2000 x 750 x 125	9	16	40	45	15	6mmΦ@200mm c/c	R6

Where

- W Panel's width (mm)
- H Panel's height (mm)
- t Total thickness of Panel (mm)
- t_1 Thickness of wythe (mm)
- t_2 Thickness of core layer (mm)
- c Concrete cover (mm)
- f_{cu} Compressive strength of panel at 28 days (N/mm^2)

4.1 Materials

The materials described in this section include the materials used as the wythe, core, reinforcement bars and shear connectors of the panels. A description on each material is presented below.

Wythe: Inner and outer wythe were made of foamed concrete with wet density from 1600 kg/mm^3 to 1700 kg/m^3 to get the target compressive strength of 15 MPa to 17 MPa for panels PLFP-1 to PLFP-6 as the preliminary test specimens.

Core: Polystyrene is used as the insulation material in the core as it is economical and readily available. The polystyrene sheet were cut into pieces and inserted between the inner and the outer wythes and between shear connectors.

Reinforcement: A square welded mild steel BRC mesh of 6 mm diameter bars with 200 x 200 mm openings was used as the longitudinal and transverse reinforcement for both wythes in all panels.

Shear connectors : Steel shear connectors in the shape of trusses of 6 mm diameter mild steel bar bent to an angle of 45º were used in both wythes so that load can be transferred from one wythe to the other. Four connectors were used over a width of 750 mm

4.2 Foamed Concrete Mixing Process

Before casting the full-scaled specimen of PLFP for pilot test, a trial mixture was first carried out to determine the characteristic properties of the foamed concrete and to get the right mixture for foamed concrete with strength 15 N/mm^2 to 17 N/mm^2 at the age of 28 days. The trial process of mixing was based on typical mix details for foamed concrete as given in British Cement Association (British Cement Association, 1994). Table 3 gives the ratio of materials used, the wet and dry density and the compressive strength at age 7, 14 and 28 days.

Table 3. *Ratio of material and characteristic properties for trial mixture*

Ratio of material						Compressive strength (N/mm^2)		
Mix	Sand: Cement	Foam: Cement	Water: Cement	Wet Density	Dry Density	7 Days	14 Days	28 Days
M1	2:1	0.6	0.6	1680	1570	9	11.9	12.3
M2	2:1	0.7	0.6	1635	1520	6.8	9.7	10.9
M3	2:1	0.8	0.6	1506	1437	4.9	5.6	8.2
M4	2:1	0.9	0.6	1429	1363	3.8	4.98	5.6
M5	2:1	1.0	0.6	1364	1244	2.8	3.6	3.8
M6	2:1	1.1	0.6	1291	1200	1.95	3	3.2

For the preliminary test, the full-scaled specimens were cast using mixture M2 in order to get the dry density of 1600 kg/m^3. In this mixture, 200 kg sand and 100 kg cement were used. The wet density was measured by taking out 1 liter of the foamed concrete from the mixer and weighed. The dry density will be measured after three days and it is expected to be slightly lower than the wet density. Therefore, the wet density looked for during the mixing was around 1650 kg/m^3. The dry density of panels recorded for pilot test are in the range of 1433 kg/m^3 to 1590 kg/m^3 for panel PLFP-1 to PLFP-4 and from 1610 kg/m^3 to 1650 kg/m^3 for panel PLFP-5 and PLFP-6 as recorded in Table 4

Table 4. *Foamed Concrete Properties for Panel PLFP-1 to PLFP-6*

Foamed Concrete	f_{cu} (MPa)	f_t (MPa)	E_c (kN/mm^2)	Wet density (kg/mm^3)	Dry density (kg/mm^3)
PLFP-1	7	0.62	10.34	1590	1365
PLFP-2	6.5	0.51	9.4	1483	1275
PLFP-3	4	0.48	8.76	1433	1260
PLFP-4	5	0.5	9.08	1479	1268
PLFP-5	8	0.65	10.88	1610	1400
PLFP-6	9	0.7	10.97	1650	1420

4.3 Material Properties

Material properties of the foamed concrete were determined from the tests carried out in the laboratory. The compressive strength was determined from the compression test which was carried out using Universal Testing Machine, UTM, based on BS 1881: Part 116: 1983. The tensile strength was determined from the split cylindrical test according to BS 1881: Part 117: 1983 and ASTM C496-90. The Young Modulus, E, was determined from the compression test on the foamed concrete cylinder using the Universal Testing Machine according to BS 1881: Part 121 and ASTM C39-86. The properties for steel used as reinforcement and shear truss connectors were also tested using UTM machine in the laboratory. The properties are as shown in Table 5 and Table 6 for foamed concrete and steel, respectively.

Table 5: Foamed Concrete Properties

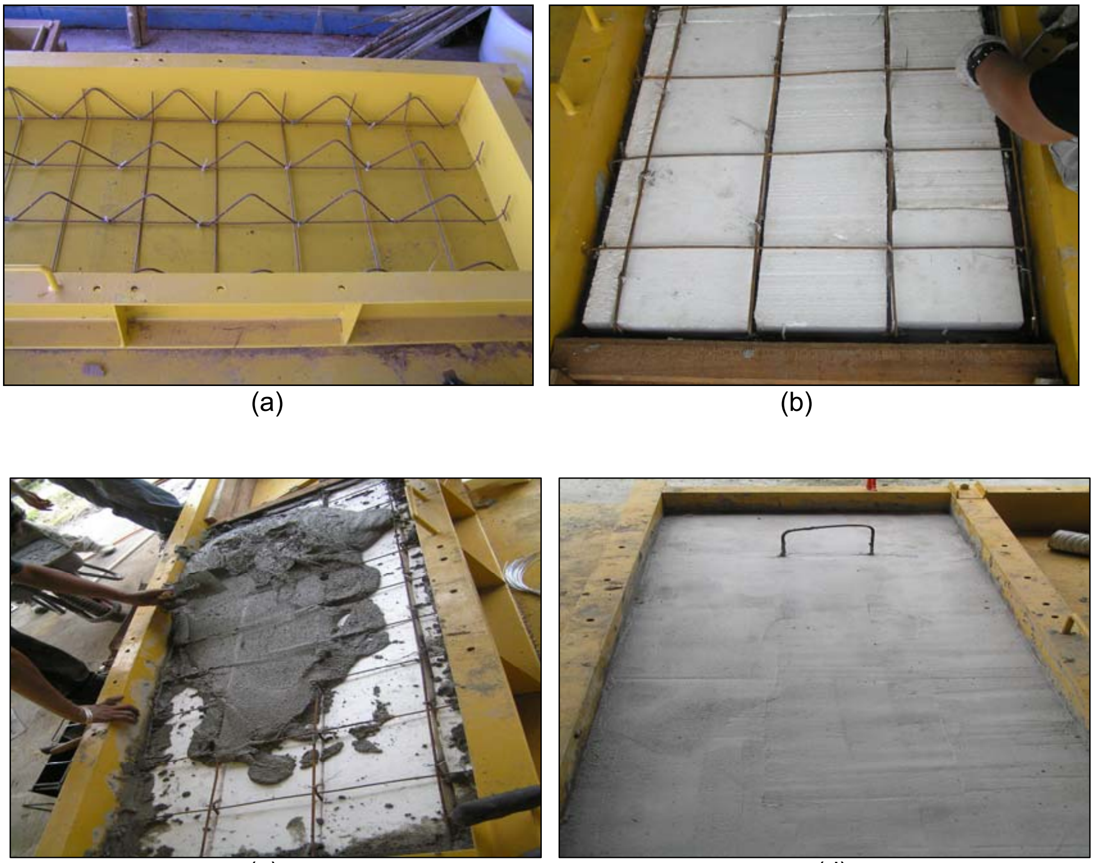
Foamed Concrete	f_{cu} (MPa)	f_t (MPa)	E_c (kN/mm^2)	Wet density (kg/mm^3)	Dry density (kg/mm^3)
PLFP-1	7	0.62	10.34	1590	1365
PLFP-2	6.5	0.51	9.4	1483	1275
PLFP-3	4	0.48	8.76	1433	1260
PLFP-4	5	0.5	9.08	1479	1268

Table 6: Properties of Steel

Steel	Yield Stress σ_y (MPa)	Tensile Strength,σ_t (MPa)	Strain at Failure	E_s (kN/mm^2)
6 mm truss connector bars	518	544.28	0.0478	197.8
9 mm bar reinforcement	559	626.5	0.1934	203.68

4.4 Fabrication and Casting

Figure 3 shows the fabrication and casting of panel specimens PLFP-1 to PLFP-4. The reinforcement was placed in the formwork before the concrete was poured. A BRC of 6 mm mild steel was tied to the 6 mm steel truss connectors as in Figure 3(a). Another BRC was tied to the steel truss connectors at the top. The BRC and steel truss connectors were placed in the formwork. The foamed concrete was poured into the formwork until the required thickness of the lower wythe was achieved. The polystyrene was cut and placed on top of the lower wythe and in between the shear connectors to form the insulation core as in Figure 3(b). The foamed concrete was again poured onto the top of the polystyrene to construct the upper wythe as shown in Figure 3(c)



(a) (b) (c) (d)

Figure 3. *(a) BRC and Shear Connectors placed horizontally in the formwork ; (b) The polystyrene was cut and placed on top of the lower wythe; (c) Foamed concrete poured on the top of polystyrene layer as the upper wythe; (d) Finish of the PLFP panel specimen*

The results obtain from the preliminary test on PLFP panels PLFP-1 to PLFP-4 shows that the panels failed prematurely due to the lower compressive strength. It is also found that the panels did not behave compositely from the load-deflection curve which shows both wythe deflected apart from each other with the rear surface deflected more than the front surface. There are few possibilities that cause the premature failure other than the lower compressive strength such as the load was not evenly distributed and the inability of the reinforcement to take up and transfer the load. It was decided that the panels need capping at both ends. Therefore, panels PLFP-5 and PLFP-6 were cast with normal concrete capping of 100 mm thickness at both ends to prevent from premature cracking near loading and support areas. The compressive strength of the normal concrete used as capping is 30 MPa. The capping was strengthened by the horizontal bars of 9mm diameter which were bent at the ends and links of 6 mm diameter. The panels were reinforced with 6 mm BRC top and bottom which were tied up to 6 mm shear trusses as shown in Figure 4.

(a) (b) (c) (d) (e) (f) (g) (h)

Figure 4. *Fabrication of Panel PLFP-5 and PLFP-6 (a) (b) and (c) : Bars and links for the end capping (d) and (e) : BRC and shear truss were placed in the formwork before foamed concrete for the bottom layer is poured (f) polystyrene were cut and placed on the bottom layer (g) top BRC was placed before the top concrete layer is pored (h) top layer of foamed concrete is poured*

4.5 Test procedure

The testing on this panel under axial load was carried out using the 2000 kN capacity Magnus Frame. The support conditions were designed as pinned at the top and fixed at the bottom of the panel. A total of 12 strain gauges were used in each specimen for surface strain measurement. The strain gauges was glued to both panel's surface at different locations and connected to the data logger which records the reading of the vertical strains. The horizontal displacement was measured by Linear Voltage Dislacement Transducer (LVDT) located at mid height and placed on opposite sides of the panel. The LVDT is also connected to the data logger to record the reading. The front and rear view of the panel are as shown in Figure 5

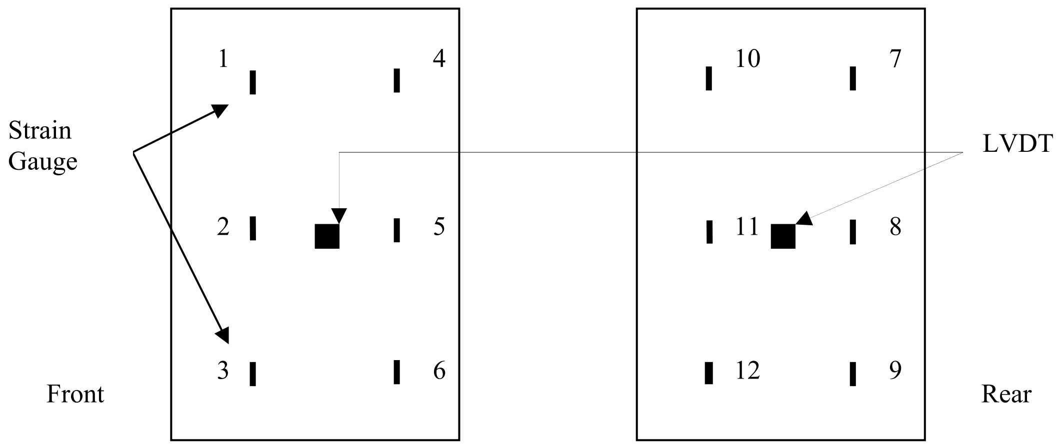


Figure 5. *Front and rear view of the panel*

A small load of 1 kN was first applied to make sure that all the instruments were working. The load is then increased gradually with an increment of approximately 25 kN till the failure of the specimen. At each load stage, strains and deflection values were recorded automatically by a Data Logger which is connected to a computer. The crack pattern was also noted at each load stage. The general behaviour of the specimen was carefully observed during the load application.

5. Discussion of Test Results

Panels PLFP-1 to PLFP-4 for the pilot test was tested under axial load. The failure mode and crack pattern of the panels were observed and the ultimate strength achieved in each panel was recorded. The load-lateral deflection profiles were drawn based on the deflection measurements recorded. The results were studied before casting panels PLFP-5 and PLFP-6.

(a) Failure Mode and Crack Pattern

From the data gathered, it can be seen that the failure in panels PLFP-1 to PLFP-4 is controlled by the material failure as shown in Figure 6. The results show that these panels failed prematurely from local buckling near the supports. This is due to the lower compressive strength achieved by the panels. PLFP-3 and PLFP-4 with lowest compressive strength failed from crushing at the bottom edge.

(a)

(b)

Figure 6. *(a) Crushing and cracking at top part of panel PLFP-3; (b) Crushing and cracking at bottom part of panel PLFP-3*

Figure 7 shows the failure mode and cracking pattern of panels PLFP-5 and PLFP-6. It can be seen from the figure that cracks occurred at the lower part of top capping in panel PLFP-5. In panel PLFP-6, crack and crush was observed near its bottom edge.

(a)

(b)

Figure 7. *Failure mode in panel PLFP-5 and PLFP-6 (a) Crack and crush at the lower part of top capping PLFP-5 (b) Crack near the bottom edge of PLFP-6*

(b) Ultimate Strength Capacity

The ultimate strength achieved in panel specimens is shown in Table 7. It can be seen that the ultimate strength achieved is influenced by the compressive strength of foamed concrete and the slenderness ratio of the panel. Panel PLFP-4 with lower compressive strength of 5 MPa achieved lower ultimate strength of 136 kN. Meanwhile, panel PLFP-3 with lowest compressive strength of 4 MPa achieved higher ultimate strength of 208 kN compared to panel PLFP-4. It is seen here that the results of ultimate strength of panels are quite scattered. Therefore, the results need to be studied further in term of the load-deflection curves of the panels. The ultimate strength achieved in panels PLFP-5 and PLFP-6 shows higher ultimate strength recorded.

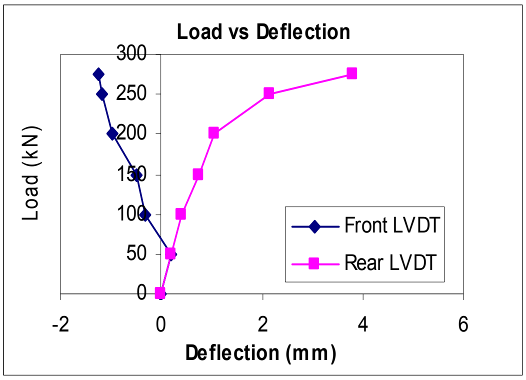
Table 7. *Ultimate Strength Results of Pilot Test Specimens*

Panel	Slenderness Ratio, H/t	Ultimate Strength, f_{cu} (kN)
PLFP-1	16	275
PLFP-2	13.33	345
PLFP-3	20	208
PLFP-4	18	136
PLFP-5	16	280
PLFP-6	16	365

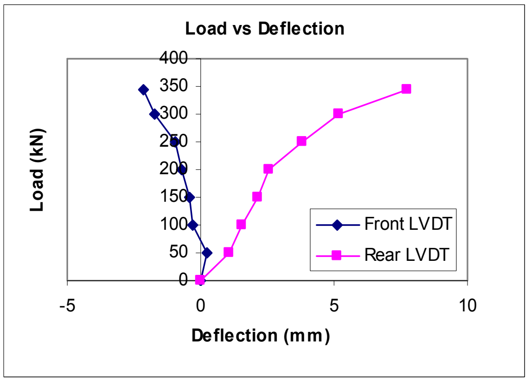
(c) Load-deflection Profiles

The load-deflection profiles for panels PLFP-1 to PLFP-4 are shown in Figure 8. The trend of the deflection curve for panels PLFP-1 to PLFP-4 are similar where under initial loading, the out-of-plane displacement behaviour shows very small deflection. Upon reaching a load of 50kN, panels PLFP-1 and PLFP-2 show some signs of cracking. As the load increases, the mid-depth displacement value also increases gradually. However, the curve for rear and front face of both panels tend to move farther apart from each other. For panel PLFP-3 and PLFP-4, both wythes started to move apart from each other even since the early loading stage.

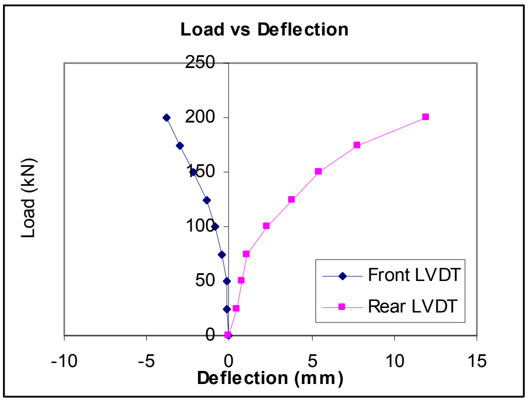
It is seen that the front and rear curve for PLFP-1 to PLFP-4 panels are all heading to the negative and positive direction. The overall trend shows that both wythe in the panels only deflected together until the first crack load occurred. After that the wythe deflected in different direction until maximum load was achieved. Figure 8 presents the load-deflection profiles for panels PLFP-1 to PLFP-4



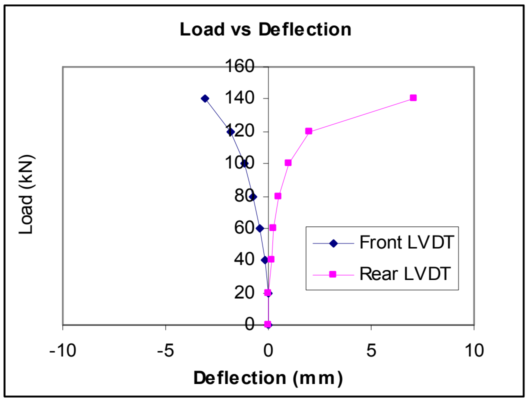
(a) PLFP-1



(b) PLFP-2



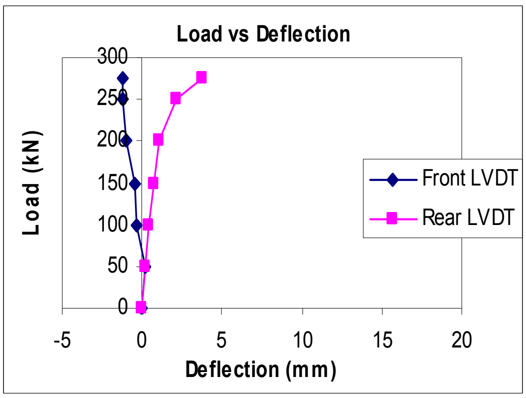
(c) PLFP-3



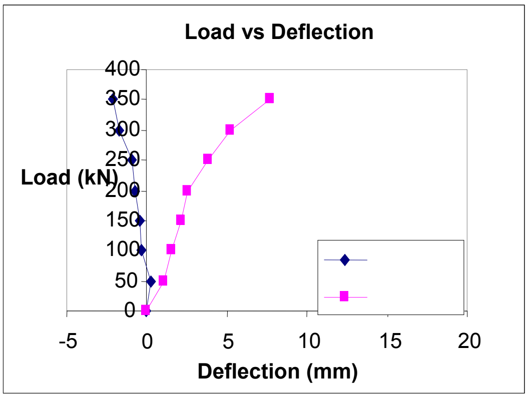
(d) PLFP-4

Figure 8. *Load-deflection profile for panels PLFP-1 to PLFP-4*

Figure 9 shows the load-deflection profile of panels PLFP-5 and PLFP-6. From the figure, it is seen that the wythes in both panels recorded small deflection measurements at the early stage of loading. The wythes is noticed to move in the opposite direction. However, both panels tend to buckle at the later stage where the front LVDT indicates that the front wythe tend to move towards the same direction as the rear wythe and this trend remains unchanged until it reaches a maximum load. This proved that capping at both ends of panels has improved the performance of the PLFP panels where certain extent of compositeness has been achieved.



(a) PLFP-5



(b) PLFP-6

Figure 9. *Load-deflection profile for panels PLFP-5 and PLFP-6*

6. CONCLUSIONS

The findings from the preliminary experiment showed that the strength achieved in the panel specimens is much lower than expected. This was due to the imperfection during the mixing and casting work. These findings have led to the mixture ratio proposed for foamed concrete wythe in PLFP panel with targeted strength of 12 MPa to 17.5 MPa. The mixture ratio which was used during the preliminary experiment need to be enhanced in the actual experimental work by reducing the foam:cement ratio from 0.7 to 0.65 and the water:cement ratio from 0.6 to 0.55. The reduction of foam:cement ratio is to get a more dense foamed concrete. Meanwhile, the water:cement ratio is reduced to take into consideration the amount of water used in the foaming process.

The premature cracking near loading and support areas indicated that the applied load was not well distributed to the whole area of panel during the pilot testing. Therefore, asides from enhancing the mixture used for foamed concrete, it was decided to use capping at both ends of the panels to avoid the same type of failure mode from occurring in the panel specimens in the actual experiment. The capping was made of reinforced normal concrete of 100 mm thickness with strength of 30 N/mm^2.

From the load-deflection profiles of the panel specimens in the preliminary work, it was found that the wyhtse in panels with no capping at both ends tend to deflect in different direction far from each other especially at the later stage of loading. However, the trend was different with panels with capping at both ends where both wythes in the panels tend to deflect together in the same direction. This can be seen in panel PLFP-5 and PLFP-6.

The proposed PLFP panel with capping at both ends is found to be practical either during casting and fabrication work or during handling and placing work before testing. Two to three panels could be cast in one day depending on the weather condition. This panel is easy to handle due to the reduction in its weight. The size makes it possible to be lifted and placed in the test frame using a forklift. The thickness and length of the PLFP panel chosen is similar to the usual size for wall panel. The width of 750 mm is used to fit the testing frame. The size and design of the PLFP panel is chosen to suit the limitation of loading capacity of the hydraulic system which is up to 1000 kN

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DISTRIBUTION OF LIVE LOAD ON SKEWED MULTICELL BOX-GIRDER BRIDGES UDER TRUCK LOADS

Mohseni, Iman[1] and Khalim A, R[2]

[1]Department of Civil Engineering, Behbahan Branch, Islamic Azad University,Behbahan, Iran

[2]Universiti Kebangsaan Malaysia, Bangi, Selangor Malaysia

[1]mohseni1979@gmail.com , [2]khalim@vlsi.eng.ukm.my

ABSTRACT: Current AASHTO LRFD Bridge Design Specifications provide several equations for distribution factor of moment and shear of multicell box- girder bridges. The equations do not account for changing in straining actions due to skew angle, continuity and effect of diaphragms. While these parameters greatly influence transverse distribution of live load on superstructure of bridges. The objective of this study is to evaluate the effects of the skew angles and continuity on the moment distribution factors of skewed multicell box- girder bridges. A parametric study is carried out on three equal span continuous bridges to determine the effective parameters on lateral distribution of loads. A total of 50 multicell box-girder bridges are modeled using finite element and grillage method. The prototype bridges are analyzed under four HS-20 design trucks. The trucks are positioned close the mid span and support to produce maximum bending moment and shear of each spans, respectively. The results are compared AASHTO Specifications and AASHTO LRFD (2008) procedures. The results indicates that live distribution factor obtained from equations of AASHTO Standard and AASHTO LRFD(2008) give very conservative results for moment distribution of exterior girder compare to grillage analysis and finite element methods. It also can be observe the grillage analysis determine rational results for skewed bridges with skew angle more than 30 degrees.

Keyword: multicell box, girder, distribution factor, skews bridges, Grillage method, truck load

1. Introduction

The responses of a bridge under live load or live load distribution factor are important for both design and evaluation purposes, because they enable the engineer to find the strength and Serviceability of a given superstructure. However, determining the accurate maximum responses and load distributions is difficult because of the complexity of bridge structures. Because of the advances in computer technology and modern finite element (FE) programs with user-friendly graphical interfaces, the finite element method (FEM) is replacing other methods, such as grillage analysis, even for more straightforward bridge analyses. Recently, Several researches were conducted on the subject of load distribution in skew composite bridges. Khaleel and Itani(1990) revealed that for a skew angle of 60°, the maximum ending moment in the interior girders is approximately 71% of that in a normal bridge, whereas, the reduction in maximum bending moment is only 20% in the exterior girders. Ebeido and Kennedy (1996) presented reaction and shear distribution factors for two-span continuous skew composite bridges subjected to AASHTO standard truck loading. The bridges were assigned partial loading in case of calculating maximum positive moment and full loading in case of calculating maximum negative moment. They revealed that the presence of the skew causes significant reductions in both girder span and support moments. They observed that the skew angle has insignificant effect on the distribution factors for angles less than 30°. However, the distribution factors changed significantly with skew angles over 30°. Moreover, it was concluded that the span length of the bridge had a major effect on its load distribution characteristics. Its effect was best reflected by the bridge aspect ratio defined by the span length/bridge width. Zokaie (1991) presented the research work on which the AASHTO LRFD (2007) distribution factors are based. The work assumed equal girder spacing, uniform girder cross sections, and constant angle of skew. The research did not incorporate the effect of cross diaphragms nor the continuity. Murat Dicleli (2009) determined the live load distribution formula for single span prestressd concrete integral abutment bridge girder. the objective of the present study was investigation the behaviour of skew continuous multicell box bridges and a parametrical study on multicell box bridge with unequal span and determination accurate moment distribution factor used FEM and Grillage method and compare with AASHTO standard and LRFD Codes.

2. METHODOLOGY

This study presents a parametric study on two continuous skew MCB bridges. The varying parameters in this study included, skew angle, span length, number of box, number of lane. The effect of the skewness on live load distribution factor are evaluated. The modeling bridges are analysis using FEM and grillage method to determine moment distribution factor. Then analytical results are compare to AASHTO LRFD (2007) and AASHTO (2002) specification to investigate the accuracy of code specifications.

3. LIVE LOAD DISTRIBUTION FACTOR AASHTO SPECIFICATION FOR MULTICELL BRIDGES

3.1 AASHTO Standard specification [2]

The distribution factors for the interior beams of cast-in-place concrete multicell beam bridges are calculated using the S-over equations from the AASHTO(2002) Table 3.23.1

For one lane loaded:

$$\mathrm{DF} = \frac{S}{8} \tag{1}$$

For two or more traffic lanes loaded:

$$\mathrm{DF} = \frac{S}{7}.0 \tag{2}$$

Range of Applicability:

$$S \leq 16\,ft \tag{3}$$

3.2 AASHTO LRFD FOURMULS [3]

For interior beams, Table 4.6.2.2.2b-1 lists the moment distribution factor equations and ranges of applicability for cast-in-place multicell beam bridges.

For one design lane loaded:

$$MDF = \left[1.75 + \frac{S}{3.6}\right]\left[\frac{1}{L}\right]^{.35}\left[\frac{1}{N_C}\right]^{0.45} \tag{4}$$

For two or more design lanes loaded:

$$MDF = \left[\frac{13}{N_C}\right]^{.3}\left[\frac{S}{5.8}\right]^{.35}\left[\frac{1}{L}\right]^{0.25} \tag{5}$$

Range of Applicability:

$$\begin{cases} \begin{cases} 7.0\text{ft} \le S \le 13.0\text{ft} \\ 60ft \le L \le 240ft \end{cases} \\ N_C \ge 3.0 \\ IF\ N_C > 8\ USE\ N_C = 8 \end{cases} \tag{6}$$

For exterior beams, Table 4.6.2.2.2d-1 lists the moment distribution factor equations and ranges of applicability for cast-in-place multicell beam bridges. The equation is:

$$MDF = \frac{W_e}{14} \tag{7}$$

Range of Applicability:

$$W_e \le S \tag{8}$$

Where S is spacing of beams (ft), N_c number of cells, L span length and W_e is equal to half the web spacing plus the total overhang and is measured in feet.

For skew bridges, a reduction factor SRF is then applied to the straight bridge distribution factors.

$$SRF = 1.05 - 0.25 tang\theta \le 1.0 \tag{9}$$

$$IF\ \theta > 60\ then\ use\ \theta = 60 \tag{10}$$

Range of applicability:

$$0 \le \theta \le 60 \tag{11}$$

4. Description of Bridge Prototypes

Two cast-in-place concrete multicell beam bridges were included in this study. The bridge consists of three varying span lengths of 91,119,140 feet. Each support is at a 26.23-degree angle from the vertical with the roadway as shown in Figure 1. The 50 ft wide bridge carrying three lanes of traffic is supported by a four-cell, cast-in-place box beam. Typical web thickness is 12 in spaced at 10.33 ft on center. Table 1 presents complete list of structural parameters for each bridges.

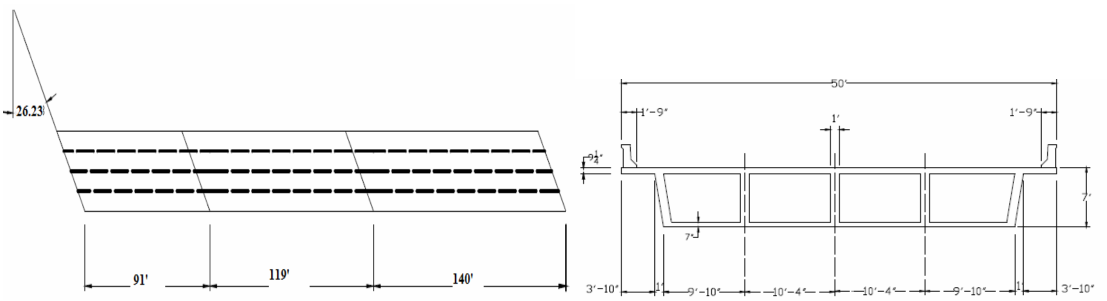

Figure 1. *Bridge No.3 multicell beam, plane view and cross section*

5. Modelling

5.1 Finite Element Model.

Sap2000 structural analysis software was used to determine the response of the bridge structure due to vehicle live loads. Bridge superstructures were typically modelled with frame elements for longitudinal and shell element transverse members (diaphragms) to form an integrated superstructure. the frame element includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations.

***Table 1**. Cast- in- place multicell beam bridge information*

Br. No.	No. span	L	ɵ	No. cells	Girder spacing	Slab Thickness (in)	Width (ft)
1	3	140.0	26.23	4	10.33	9.25	50
2	2	110.0	10.60	3	9.50	8.00	36

5.2 Grillage Model Geometric

The cross section show in Figure 1 was modelled with a set of longitudinal and transverse beam element. Figure 2 shows the placement of transverse and longitudinal grillage members adopted in this paper. The grillage mesh is in the plane of the principal axis of bending of the deck as a whole with the longitudinal members coincident with longitudinal webs, and one longitudinal grillage members were placed for each girder, representing each web of gorder [1]. Grillage analysis requires the calculation of the moment of inertia, I , torsional moment of inertia, J , and equivalent shear area of a transverse grillage member, a_s. Hambly(1990) suggested following equations for multicell box grillage member properties:

$i = (d^{\prime\prime} d^{\prime} h^{\prime 2})/((d^{\prime} + d^{\prime\prime}))$ Per unit length (12)

$j = (2h^{\prime 2} d^{\prime} d^{\prime\prime})/((d^{\prime} + d^{\prime\prime}))$ Per unit length (13)

$$a_s = \frac{\left((d^{\prime 3} + [d^{\prime\prime}]^{3}))/l^{2} \left[(d_w^{3} \ast l)/(d_w^{3} l + (d^{\prime 3} + d^{\prime\prime 3})h)\right)\right] E}{G}$$ Per unit width (14)

Where: $d^{\prime\prime}$ and $d^{\prime}$ are the top and bottom slab and $h^{\prime\prime}, h^{\prime}$ are distances from their centriod of deck. d_w, l and h are web thickness , distance between two adjacent web and total depth of cells, respectively. For bridge No.3 grillage properties are $i_s = 2.10, c_s = 4.185$ and $a_s = 0.004$.

In both methods, support conditions were defined using hinges at the beginning of the bridge and rollers at the other supports. The live load applied to the bridges was an HL-93 truck. The bridge was loaded with series 1, 2 and 3 trucks moving load respectively. The AASHTO(2002) Standard intensity reduction factors were used for three and four truckload results (0.9 and 0.75 respectively). The maximum responses of interior and exterior girders were determined by moving the trucks on a bridge to various locations in the longitudinal and transverse directions until the maximum response was obtained.

6. RESULTS OF ANALYSIS AND COMPARISON

The analytical results obtained include the maximum live load moment on the exterior and interior girders, as shown in table 2. Maximum distribution factors of moment at each girders were determined by dividing the maximum shear and moment from each bridges analysis by the maximum shear and moment from a single beam line analysis at the corresponding location.

Table 2. *Max. Moment distribution factor for bridges 1 and 2*

Bridge No	Skew angle	Max. Span length(ft)	Beam	DLF.m FEM	DLF.m LRFD	FEA/ LRFD	DLFm. AASHTO	FEM/ AASHTO
1	26.23	140	Exterior	0.490	0.616	0.795	0.678	0.722
1	26.23	140	Interior	0.689	0.670	1.020	0.678	1.010
2	10.60	110	Exterior	0.570	0.616	0.920	0.678	0.841
2	10.60	110	Interior	0.810	0.570	1.420	0.678	1.190

6.1 EFFECT OF SPAN LENGTH ON MOMENT DISTRIBUTION FACTORS.

Figures 2 and 3 show the moment distribution factor vs. span length for exterior and interior beam of bridges. That was seen that grillage method matches with that of FEA very well, but AASHTO, Standard and LRFD were very conservative for interior and exterior beams. LRFD specification results were unconservative for exterior girder while span length exceeded of 170ft.

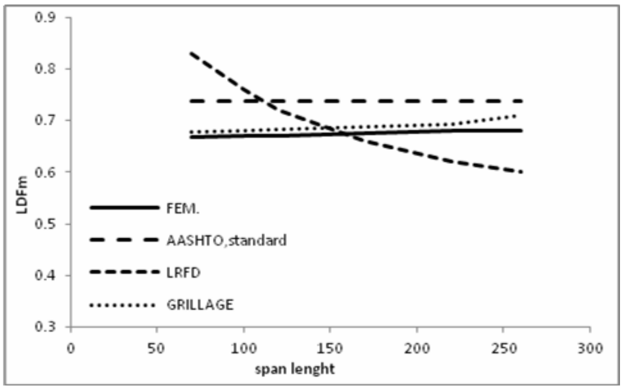

Figure 2. *Moment Distribution Factor vs. Span Length For exterior beam*

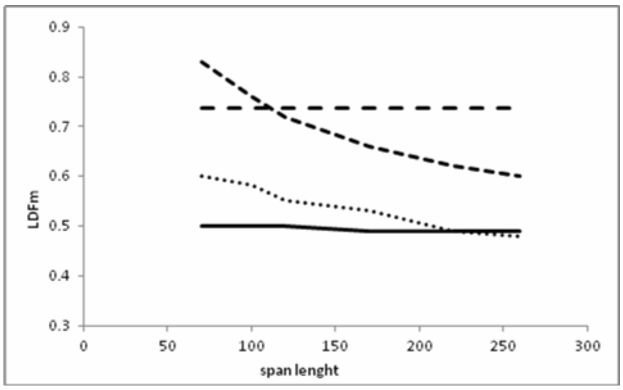

Figure 3. *Moment Distribution Factor vs. Span length for interior beams*

6.2 EFFECT OF SKEW ANGLE ON MOMENT DISTRIBUTION FACTORS

Figures 4 and 5 show the moment distribution factors obtained for all four methods versus the skew angle. The skew angle for bridges in this study ranged from zero to 60 degrees. By comparing each method of calculation with FEM, it can be seen that all methods predicted moment distribution factors very conservative for exterior girders, but,for interior girders AASHTO-LRFD predicted moment distribution factors partly unconservative for skew angle more than 30 degree.

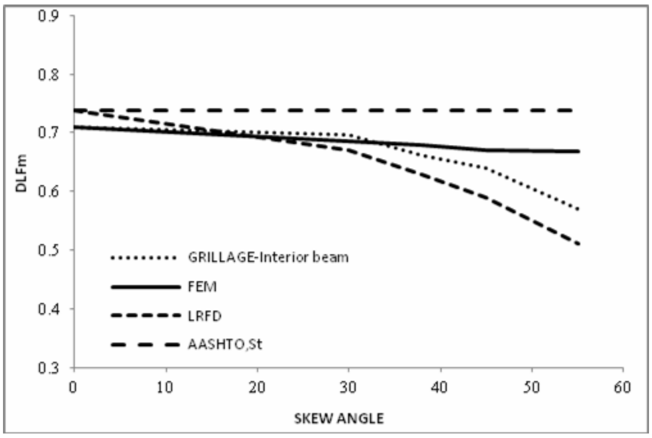

Figure 4. *Moment Distribution Factor vs. Skew for interior beams*

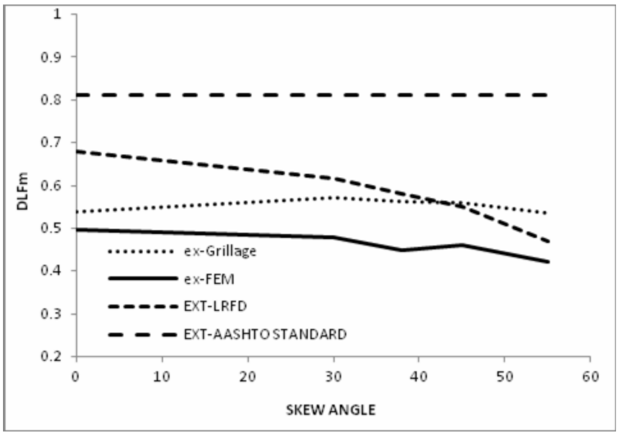

Figure 5. *Moment distribution factor vs. Skew for exterior beams*

Figure 6 shown that the presence of skew causes significant reductions in both span and support girder moment. It should be noted that in the case of right bridge, $\theta = 0$, the span and support girder moments for an interior girder are higher than those for an exterior girder. However, the factor LDFm for an interior girder increases more significantly due to skew than in the case of an exterior girder.

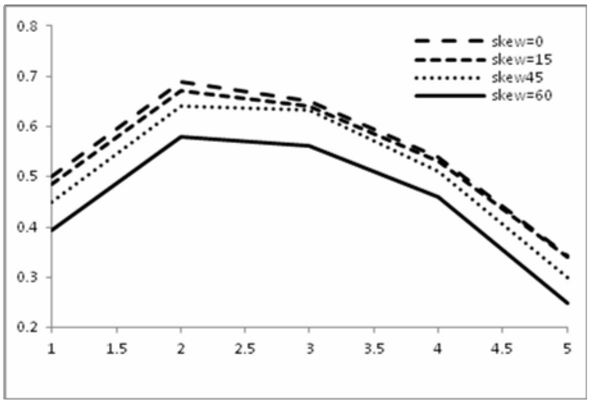

Figure 6. *Moment distribution factor vs. transverse Location of beams*

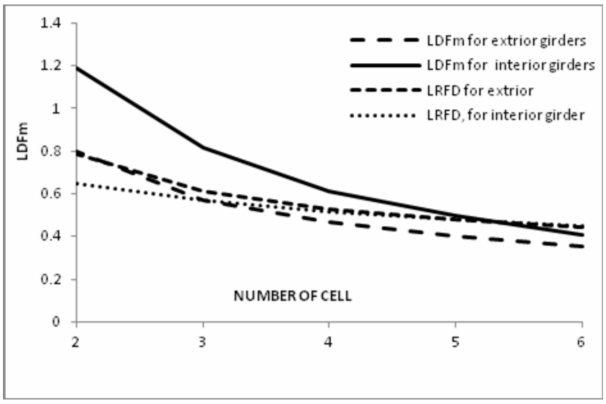

Figure 7. *Moment distribution factor vs. number of cells*

6.3 EFFECT OF NUMBER OF CELLS ON MOMENT DISTRIBUTION FACTOR

Figure 7 shows the influence of changes in the number of cells on the distribution factors for maximum positive moment distribution factor. It can be observed that as the number of cell becomes more, the distribution factors for maximum positive moment decreased. However, LRFD estimated higher values for LDF of interior girder, it estimated fairly close to values of DLF for exterior girders.

7. CONCLUSIONS

The load distribution in continuous multicell beam bridges was examined by carrying out an extensive parametric study on prototypes of such bridges. It was deduced that the main parameters affecting the load distribution are the span length, number of cells and skew angle of deck. A finite-element model was used in this study. The effect of continuity was compared with current practice using AASHTO (2002), LRFD (2007) and Grillage method. The comparison revealed that The AASHTO Standard method for calculating live load moment distribution factors produces conservative results compare to FEM in nearly every case. When compare to each other, the AASHTO LRFD (2007) and Grillage methods produce very similar results.

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EXPERIMENTAL COMPARATIVE ANALYSIS OF SOLAR CHIMNEY HOUSE PERFORMANCE WITH ENERGY SAVING HOUSE AND GENERAL HOUSE IN THAILAND

Jaran Ratanachotinun[1], Jongjit Hirunlabh[2], Nat Kasayapanand[3], Sopa Visitsak[4], Sombat Teekasap[5] and Joseph Khedari[6]

[1, 2] Building Scientific Research Center, Institute for Scientific and Technological Research and Services,

[3] Energy Technology Division, School of Energy Environment and Materials, King Mongkut's University of Technology Thonburi, Thailand

[4] Faculty of Architecture , Kasetsart University, Thailand

[5] Faculty of Engineering , Eastern Asia University, Thailand

[6]Rattanakosin College for Sustainable Energy Environment, Rajamangala University of Technology Rattanakosin, Thailand

jaran07@yahoo.com[1] , ijonlabh@kmutt.ac.th[2]

ABSTRACT: The objective of this research was to conduct field comparison of performance of Solar Chimney house (SCH) with Energy Saving house (ESH) and a regular general house (GH) used as a reference during the 3 seasons of Thailand: summer, rainy and winter. SCH designed by the Building Scientific Research Center (BSRC) was composed mainly of a Modified Trombe Wall and a Roof Solar Collector built using common construction materials. Air gap in the wall and roof allow air flow from inside to outside. The Energy Saving House (ESH) was built with lightweight autoclave concrete blocks and well insulated roof. The General House (GH) was built as usual in Thailand by concrete blocks and concrete tiles. The dimensions of house models are 1.3x1.3x2.5 meters. Experimental observations indicated that SCH average indoor temperature has good performance in all seasons and thus confirmed past researches. In summer, the average diurnal and nocturnal ambient temperature were 31.9 C and 29.1 C respectively. When compared to ESH and GH houses, the average daily indoor temperature decrease was about 1 and 2 C respectively. During rainy season and winter, the average diurnal and nocturnal ambient temperature were 29.22, 26.04 C and 28.83, 27.14 C respectively. The corresponding average daily indoor temperature decrease was about 1 C when compared to both ESH and GH for both seasons. Construction cost for the Modified Trombe wall and Roof solar collector (SCH) are 19.7 and 67 % higher than the cost of GH but 23 % lower than the wall cost of ESH and 15.5 % higher than the roof cost of ESH. However SCH still represents a good choice as the average daily electrical consumption of air conditioner will be reduced by 7-10% for every 1 C decrease of indoor temperature. Additionally solar chimney design has several practical advantages such installing electric wires and telephone cables in the air gap that would represent an additional saving too. The outcomes of this research will result into the development of a prototype of SCH for real construction in the future.

Keywords: solar chimney, modified trombe wall, roof solar collector, indoor temperature, energy saving

1. INTRODUCTION

The trends in energy saving at the present time have gained a lot of attention due to the fact that energy from the natural resources is decreasing, that the energy cost is increasing and that the world is encountering global warming. House or domicile is the largest unit in the world, resulting in the consumption of large amount of energy.

Published researches about solar chimney houses worldwide including those reported by our team (BSRC) showed that the temperature inside the solar chimney house when compared to general

house is lower and such it saves electrical energy for air-conditioner. However, few researches have been done for all seasons. Most of them were done in a specific season or at specific time and few of them considered simultaneous comparison with lightweight autoclave brick house and well insulated roof. This is being the main research objective of this work.
The objective of this research was to conduct field comparison of performance of Solar Chimney house (SCH) with Energy Saving house (ESH) built with lightweight autoclave concrete blocks and well insulated roof and a regular general house (GH) used as a reference during the 3 seasons of Thailand: summer, rainy and winter. SCH designed by the Building Scientific Research Center (BSRC) was composed mainly of a Modified Trombe Wall and a Roof Solar Collector built using common construction materials. For the experiment, 3 house models were built in Bangkok (Thailand) to compare the temperature inside the houses to prove that solar chimney house show good perfomance compared to other types of houses and that solar chimney house is suitable for all seasons or all year round. The outcomes of this research will be useful for the development of a prototype of an economic bioclimatic energy saving house which is suitable for real world in Thailand in the future.

2. METHODOLOGY

This research consists of 2 phases: data collection and analysis of temperature inside the houses from all seasons and a design layout of a general real bio-climatic energy saving houses.
Based on past research works of the Building Scientific Research Center (BSRC) about inducing natural ventilation by the use of Solar Chimney (SC) concept (J. Khedari et al. (2003), B. Boonsri et al. (2000), C. Lertsatitthanakorn et al. (1997), J. Waewsak et al., 2003, S. Chaima et al., 1997, S. Wachirapuwadon et al., 2001, T. Bunnag et al., 1997)) Three house models (figures 1-3) were built with the same dimensions: 1.3x1.3x2.5 meters. The first house was (SCH) with modified double-layer Trombe wall facing south. The outer wall was brick and the inner wall was gypsum board. Air gap between walls was 10 centimeters and the air gap to allow air flow from inside to outside the house was 5 centimeters. The roof was made from concrete tiles (CPAC monier) and gypsum board, spaced 10 centimeters. The second house was built using energy saving materials (ESH) with lightweight autoclave concrete blocks and gable-shaped well insulated roof. The third house was a general house (GH) with red bricks and concrete tiles, gable-shaped roof and cement boards as ceiling. Data collection was done in summer, rainy season, and winter. Data were collected for 7 days in each season, totaling 21 days. The average temperature for each house was analyzed for performance comparison.

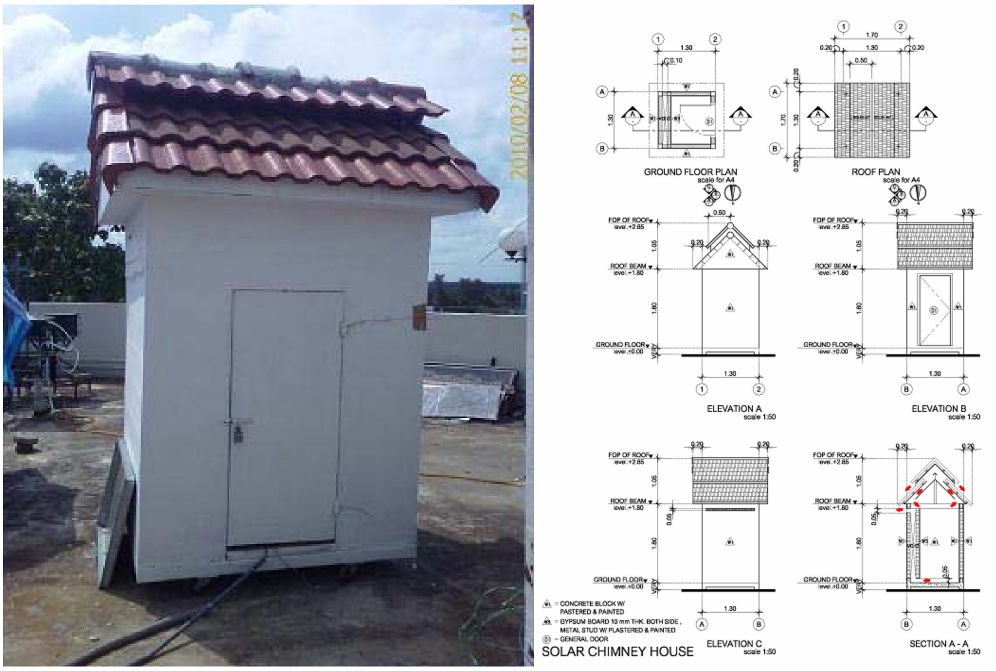

Figure 1. *Solar Chimney House (SCH)*

Figure 2. *Energy Saving House (ESH)*

***Figure 3**. General House (GH)*

3. CONCLUSIONS

3.1 Comparison of average hourly indoor temperature during the measured period of SCH, ESH and GH from all 3 seasons: summer, rainy season and winter is shown in figures 4 to 6.

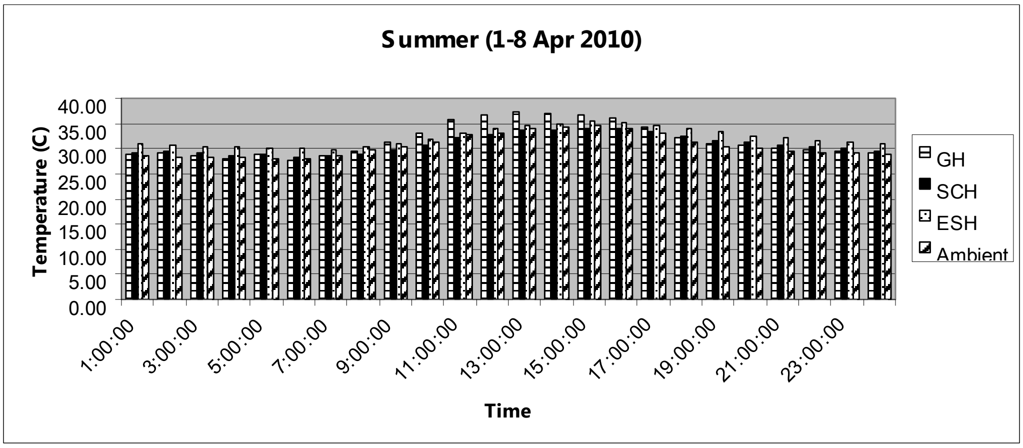

***Figure 4.** Comparison of average temperature inside 3 house models in summer (1-8 April 2010)*

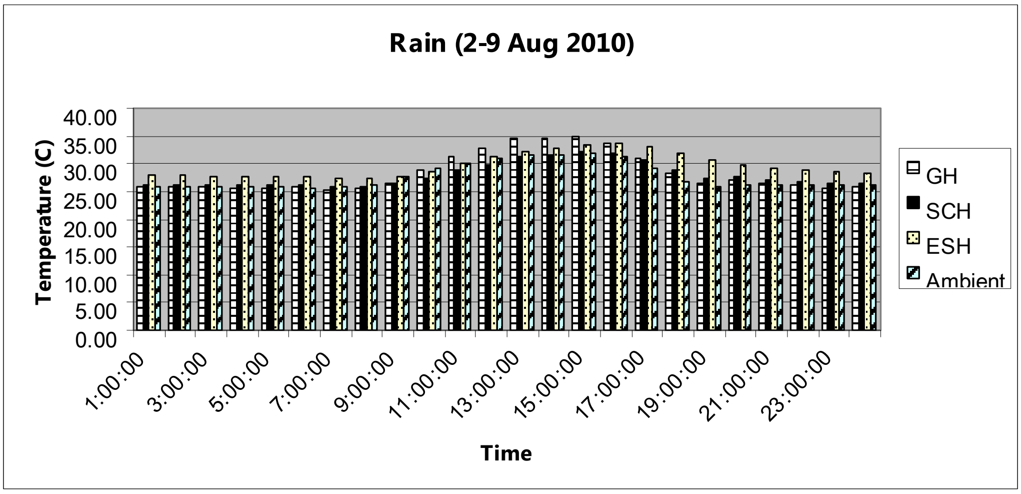

Figure 5. *Comparison of average temperature inside 3 house models in rainy season (2-9 August 2010)*

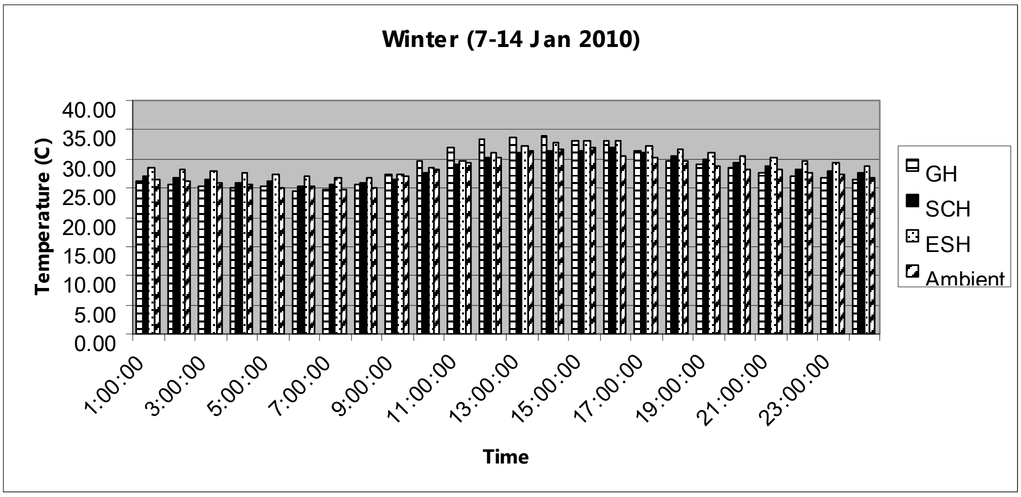

Figure 6. *Comparison of temperature inside 3 house models in winter (7-14 January 2010)*

To simplify comparison, a summary of average indoor temperature inside the 3 house models and ambient temperature is shown in the table 1 during daytime (6.00am-18.00pm) and nighttime periods (18.00pm-6.00am) separately.

Table 1. *Average indoor temperature of the house models and ambient temperature (AMB) during daytime and nighttime for 3 seasons (Celsius degree)*

	SUMMER				RAINY				WINTER			
	AMB	SCH	ESH	GH	AMB	SCH	ESH	GH	AMB	SCH	ESH	GH
day time	31.99	31.68	32.90	33.65	29.22	28.99	30.42	30.42	28.83	28.90	30.07	30.19
night time	29.13	30.08	31.51	29.64	26.04	26.77	28.83	26.26	27.14	27.88	29.27	26.91

Experimental observations indicated that SCH average indoor temperature has good performance in all seasons. In summer, the average diurnal and nocturnal ambient temperature were 31.9 C and 29.1 C respectively. When compared to ESH and GH houses, the average daily indoor temperature decrease was about 1 and 2 C respectively. During rainy season and winter, the average diurnal and nocturnal ambient temperature were 29.22, 26.04 C and 28.83, 27.14 C respectively. The corresponding average daily indoor temperature decrease was about 1 C when compared to both ESH and GH for both seasons.

3.2 The cost for the construction of modified Trombe wall is 26.7 $/m2 approximately and the cost of Solar Collector roof is 54.7 $/m2 approximately. When compared to the construction of specific parts in the general house, the cost increase is 19.7 and 67% respectively and 23 % lower than the wall cost of ESH and 15.5 % higher than the roof cost of ESH. However, the maximum temperature difference is about 2 Celsius. According to the research published by J. Khedari. et al. (2003) which is a field study of performance of solar chimney with air-conditioned building, solar chimney house could save electrical energy from air-conditioner by 7-10% when compared to general house when the indoor temperature decreases by 1 Celsius. In a previous research work (J. Ratanachotinun et al. 2009), on the study of cost and economic feasibility for energy saving house in Thailand, we reported that 57.7% of the sampling group of house owners preferred solar chimney house due its various advantages.

As a summary of the overall study on solar chimney house model, it can be claimed that SCH average indoor temperature has good performance in all seasons. SCH is practical in real world and could maintain the temperature to be suitable for living year round (except when ambient temperature exceeds human body temperature). The average indoor temperature difference is around 2 Celsius. This research could be useful for SCH performance confirmation and help to gain approval from general house owners who would like to live in solar chimney house. Besides solar chimney design has several practical advantages such as installing electric wires and telephone cables in the air gap that would represent an additional saving too and reduce the puncture in the wall as done in general house.

3.3 The design of solar chimney house according to past research and the results from this research could be presented in formulation of real energy saving house for construction and living. The fixed model is 2-story single house with 180 m^2 area as shown in figure 7 to 10. The materials for construction are common ones. The wall is made of red bricks and the roof is made of concrete tiles. As for the wall facing south, it will be a double layer wall and the roof will have two layers.

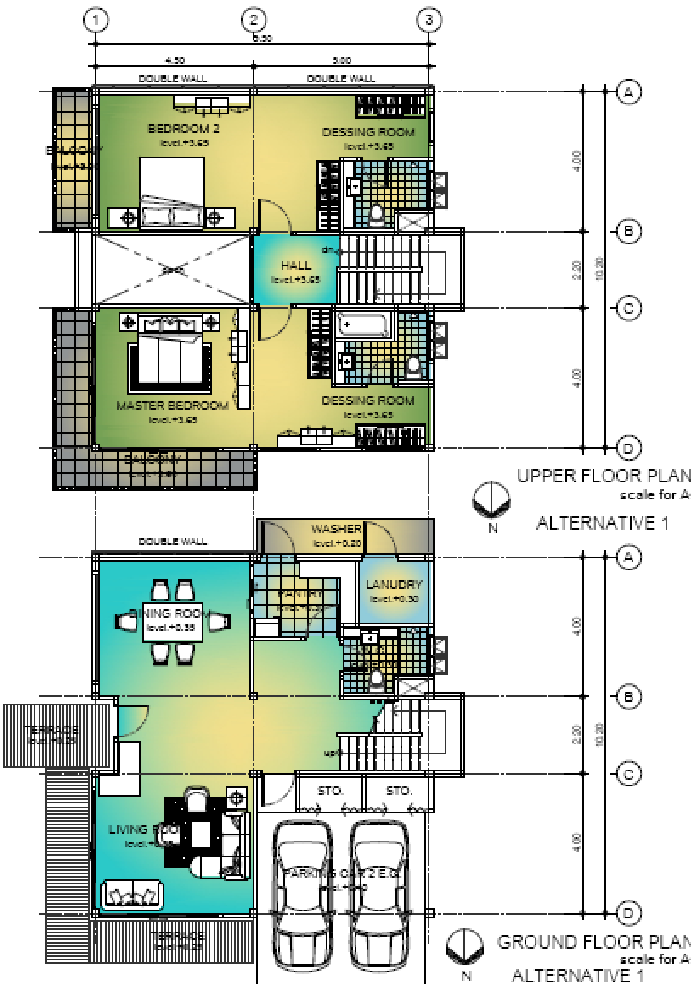

***Figure 7.** House plan of bio-climatic energy saving house prototype*

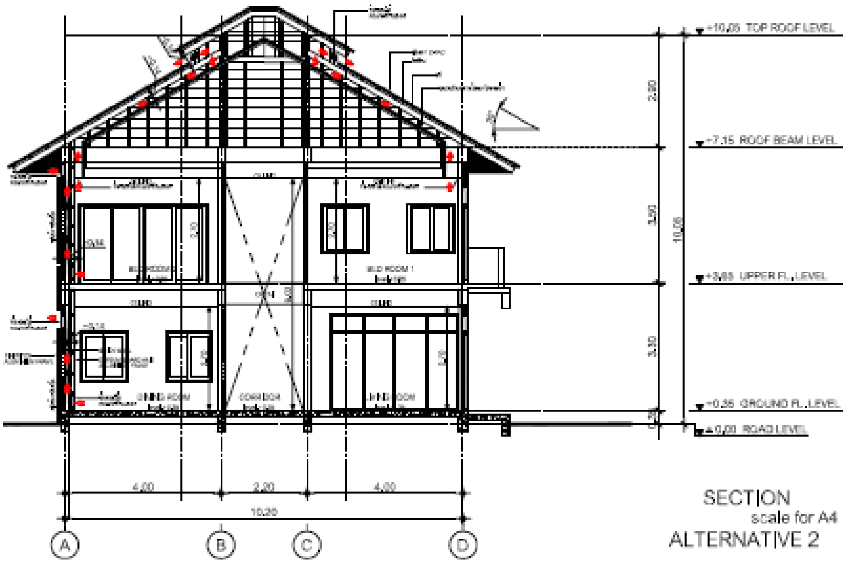

Figure 8. *Front section of the bio-climatic energy saving house prototype*

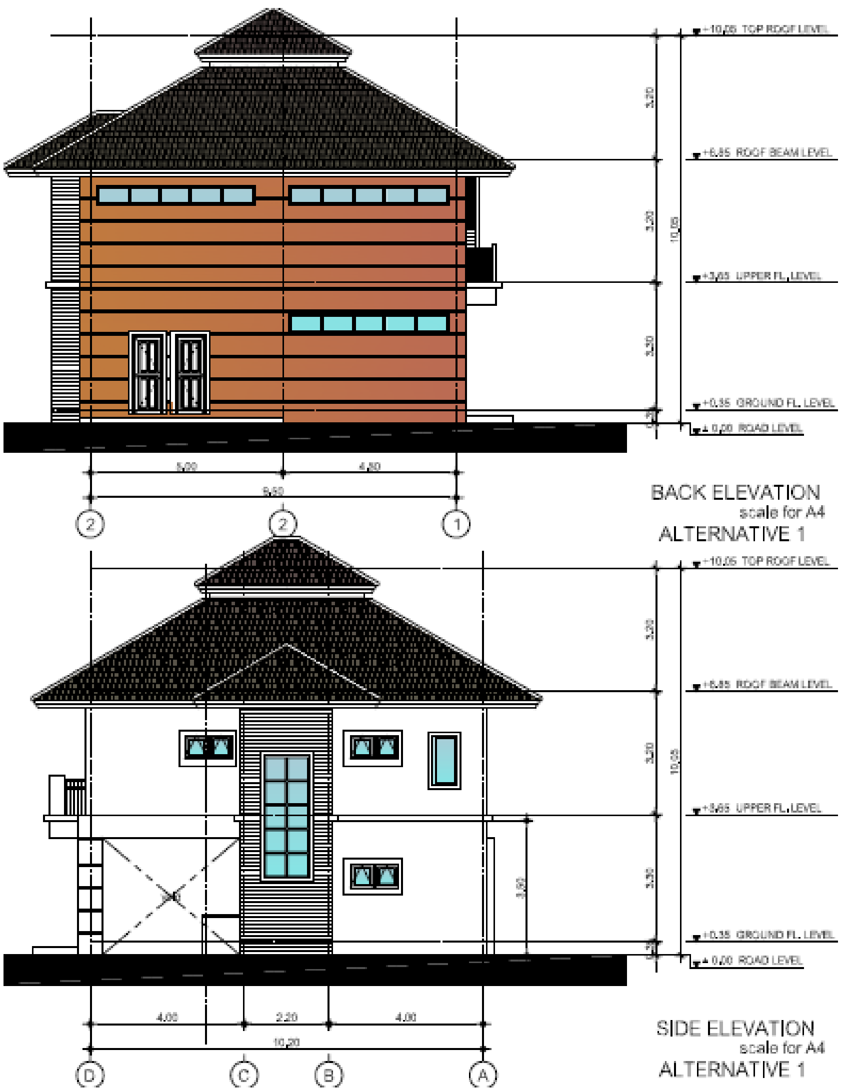

Figure 9. *Side elevations of bio-climatic energy saving house prototype*

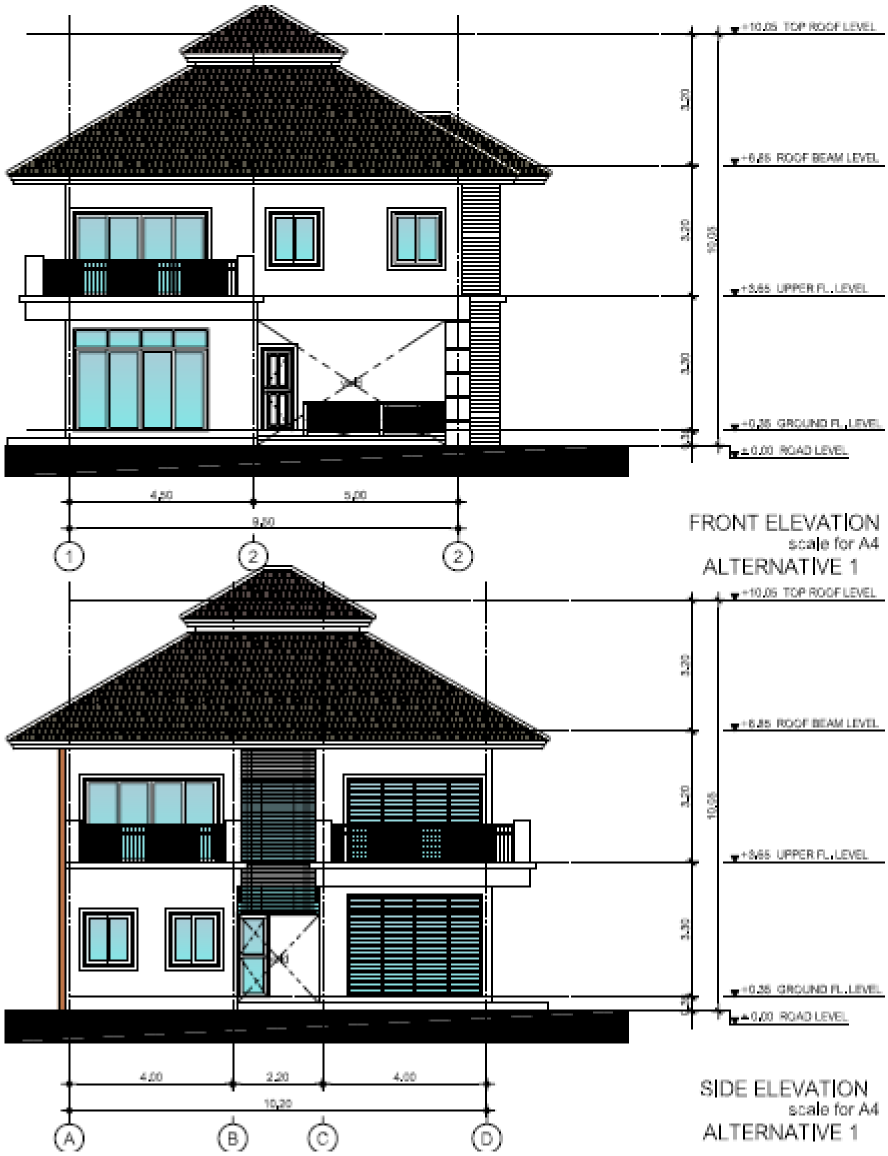

***Figure 10.** Side elevations of bio-climatic energy saving house prototype*

The proposed house plan of solar chimney house will be useful for the real construction in order to study guideline the efficiency of the energy saving and economic value as well as the suitable technique for construction. The house plan may be adapted to suit the real performance and the demands of the house owners.

ACKNOWLEDGEMENT

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T145

THE POTENTIAL OF USING BOTTOM ASH (BA) ADDITIVES IN ROAD BASE

Norazlan K.[1], Norbaya S.[2], Mohd Fadzil A.[3]

[1,2,3]Faculty of Civil Engineering, UiTM Shah Alam

[1]aln_kh82@yahoo.com

ABSTRACT: This paper presents the results of study on the potential of using bottom ash (BA) in road base crushed aggregate. The bottom ash used in this study from Jimah Power Plant. The bottom ash been tested and the sizes range from 0.075mm to 2mm, indicating sand size (about 90%). This sample of bottom ash also classified as type-F fly ash because bottom ash describe as siliceous and aluminous materials that possess little or no cementatious value because the lime (CaO) content is relatively low. The road base aggregate been used in this study is based on gradation limits for crushed aggregate road base from JKR specification. This study examines the potential of using bottom ash (BA) as additives to the crushed aggregate for road base and substitution the sand size with bottom ash for the crushed aggregate for road base. The objective of this study is to determine the optimum concentration additives of bottom ash (BA) based on the CBR value and the CBR value for road base aggregate mixed with the optimum concentration of bottom ash (BA). The second objective is to determine the CBR value for the substitution of sand portion (from 0.075mm to 2mm) in road base crushed aggregate with the bottom ash. This study involved the California Bearing Ratio (CBR) test to determine the strength of CBR value for base course materials. The entire test involved for unsoaked test. The CBR results for all testing were compared to the control of CBR value for crushed road base aggregate. The laboratory result shows, the optimum concentration of using bottom ash (BA) as additives in road base aggregate is about 9%. The addition of 9% bottom ash (BA) to the road base crushed aggregate were improve the CBR value and gives the best result for CBR value compared to the control. The substitution of sand size from 0.075mm to 2mm of road base crushed aggregate with bottom ash were gives 11% higher for CBR value compared to the control. These study shows, bottom ash more effective used as additives to the road base aggregate to enhance the CBR value.

Keywords: Bottom ash, California bearing ratio, road base

1. INTRODUCTION

Nowadays Malaysia promotes coal as a fuel of choice for power generation, to free up more natural gas for export. Coal was introduced as a raw material for power generation since 1988 in Malaysia. Coal consumption is expected to increase from 10 million tonne to 19 million tonne in year 2010 (Joseph, 2005). Consequently, a large volume of coal ash produced is typically disposed of as a waste in utility disposal sites. Coal ash mostly consists of fly ash and bottom ash. According to the American Coal Ash Association (ACAA, 2006), in the US, the general production ration of fly ash and bottom ash is approximately 80:20. The heavier ash that drops through the bottom of the furnace where it is collected in a funnel is called as bottom ash.

Some of the studies showed the engineering properties of bottom ash were more favorable than those of traditional highway material and has the capability to improve asphalt pavement performance when used to substitute a portion of the aggregate in asphalt mixes (Wei, 1990). The U.S. Environmental Protection Agency (EPA, 2005) encourage the use of coal combustion products in highway construction projects such as road base, embankments, flowable fill, and other beneficial applications. The increased use of these materials, would be discarded a waste and reduce greenhouse gases in the atmosphere, reduce energy consumption, and conserve natural resources.

According to American Coal Ash Association (ACCA, 1996), in 1996 an approximately 0.6 million metric tonne of bottom ash were used as road base or sub base materials. Bottom ash could be used as a road materials as additives or replacing certain size of aggregate to its characteristic (Maria

Izquierdo, 2007). Some of the engineering properties of bottom ash that are of particular interest when bottom ash are used as an aggregate in granular base applications are gradation, specific gravity and unit weight, moisture-density relationship, degradation under compaction, shear strength, bearing strength, and corrosivity. The gradations of bottom ash are considered fine aggregates (ASTM 1994). In order to improve the sizing characteristics of the bottom ash or boiler slag, a conventional aggregate may be blended with the ash prior to its use in a base or subbase.

This study examines the potential of using bottom ash (BA) as additives to the crushed aggregate for road base and bottom ash use as substitution for sand size to the crushed aggregate for road base. This study involved the determination of optimum concentration additives of bottom ash (BA) based on the CBR value and the CBR value of mixed road base aggregate with the optimum concentration of bottom ash (BA). The second objective is to determine the CBR value when the substitution of sand size from 0.075mm to 2mm of road base crushed aggregate size with the of bottom ash. The laboratory was conducted to determine the CBR value. It shown that bottom ash can be usefully used as additives to the road base aggregate.

2. MATERIALS AND METHODS

2.1 Road Base Aggregate

The aggregate sample for road base was collected from Hanson Quarry based on the gradation limit for road base aggregate from JKR road work specification shown in Figure 1. The properties of road base aggregate shown in Table 1.

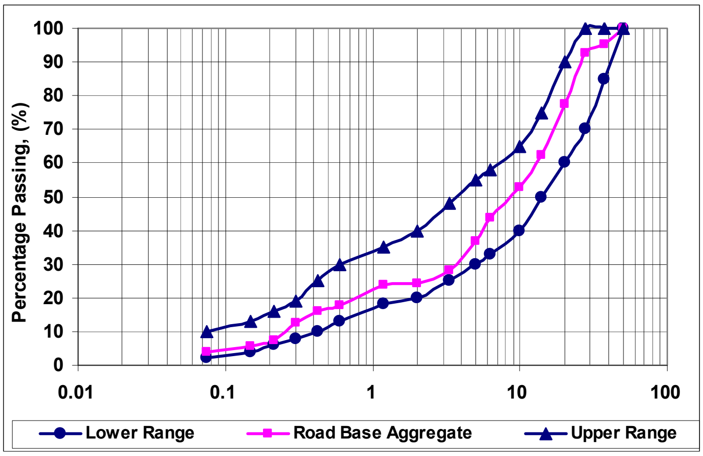

Figure 1. *Particle size distribution for road base aggregate based on gradation limit from JKR Specification for road work*

Table 1. *Physical properties of Road Base Aggregate*

Properties	Values
Specific gravity, (Gs)	2.67
Particle size distribution:	
Gravel (%)	70
Sand (%)	20
Fine/Silt (%)	10
Compaction Characteristic:	
Optimum water content, %	7.26
Maximum dry density, (Mg/m^3)	2.07

2.2 Bottom Ash

The main additives materials used in this study were bottom ash (BA) and collected from TNB Power Plant in Port Dickson, Negeri Sembilan. The bottom ash was sieved and the size ranging from 0.075mm to 2mm indicating sand size and the result shown in Figure 2. The bottom ash also tested for physical properties and the result shown in Table 2. The result for chemical properties of bottom ash determined by X-ray fluorescence (XRF) test shown in Table 3 and the bottom ash (BA) sample classified as Class-F ash. Table 4 shows the classification of BA compliance with ASTM C618. Based on the result, BA describe as siliceous and aluminous materials that possess little or no cementatious value consisting a little quantity of Calcium Carbonate (CaO) lower than 10%. The combination percentage composition of silicon dioxide (SiO_2), alumininа oxide (Al_2O_3) and iron oxide (Fe_2O_3) more than 70 percent. This BA considered as non self-cementing ash because having pozzolanic properties and no or small quantities of self-cementing properties sources of calcium and magnesium ions.

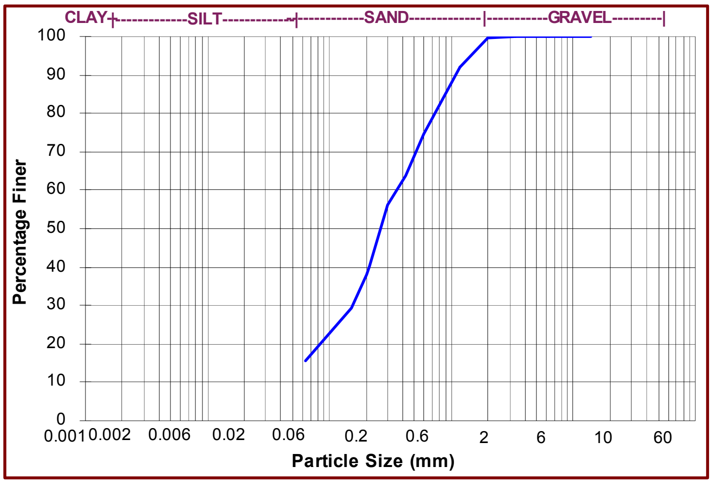

***Figure 2**. Particle size distribution for Jimah bottom ash*

***Table 2.** Physical properties of Jimah Bottom ash (BA)*

Properties	Values of BA
Natural Moisture content, (%)	
Specific gravity, (Gs)	2.06
Particle size distribution:	
Gravel (%)	0
Sand (%)	84.5
Fine/Silt (%)	15.5
Compaction Characteristic:	
Optimum water content, %	8
Maximum dry density, (Mg/m^3)	1.227
Classification (BS)	SAND with fine soils

***Table 3.** Chemical composition of Jimah Bottom ash (BA)*

Chemical Constituents	Chemical Composition of Jimah BA (%)
Silicon Dioxide (silica) , SiO_2	46.60
Alumunium trioxide, Al_2O_3	30.30
Iron oxide, Fe_2O_3	6.44
TiO_2	3.31
Calcium Oxides (lime) CaO	0.68
Potasium oxide , K_2O	0.50
Sodium oxide , Na_2O	0.26
Magnesium oxide, MgO	-
Sulphate, SO_3	0.09
L.O.I	0.64

Table 4. *Classification of PFA compliance with ASTM C618*

		Chemical Requirements for Fly Ash Classification (ASTM C618)		**Result of chemical composition of Jimah BA (%)**
Properties	Requirement	Class F	Class C	
SiO_2 + Al_2O_3 + Fe_2O_3	Min (%)	70	50	**83.34**
SO_3	Max (%)	5	5	**0.09**
Loss on ignition	Max (%)	6	6	**0.64**

2.3 Laboratory testing

There are several testing carried out to determine the physical properties of road base aggregate and bottom ash based on BS 1377: 1990. The main testing is the California Bearing Ratio (CBR) test carried out to determine the CBR value of the samples. The CBR test carried out based on the standard procedure given in BS1377-4: 1990 and ASTM D1883. This test method is used to evaluate the potential strength of sub-grade, sub-base and base course material. CBR defined as the ratio of the load sustained by the specimen at 2.5 or 5.0 mm penetration to the load sustained by standard load aggregates at corresponding penetration level. In this study, unsoaked CBR tests were performed on the samples. The samples were prepared with its OMC and were compacted at their MDD by using static compaction machine.

3. RESULT AND DISCUSSION

3.1 The optimum concentration of bottom ash as an additives in road base aggregate

Figure 3 shows the optimum concentration result of bottom ash (BA) been used as additives to the road base aggregate. It was found that, the optimum concentration of bottom ash is about 9% with the CBR value about 204%. The graph shows, more than 9% of bottom ash been used as an additives to the road base aggregate will give the lower of CBR value.

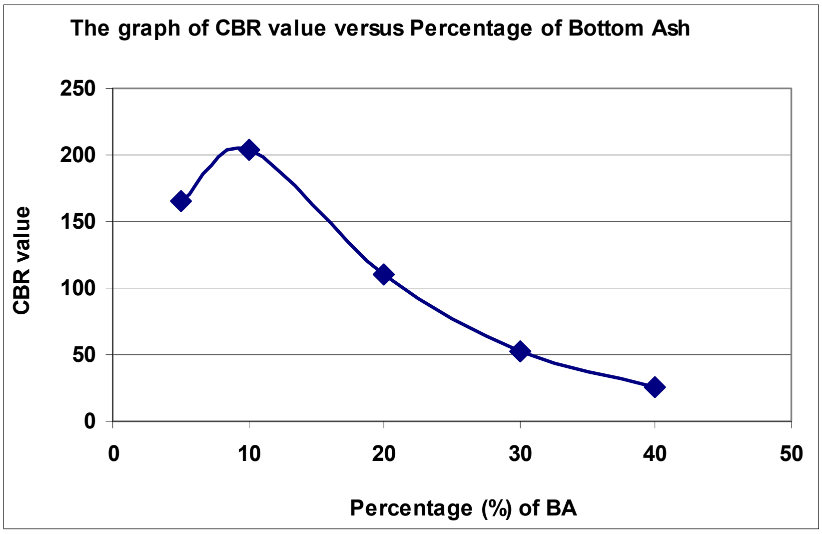

Figure 3. *The graph of CBR value versus the percentage of bottom ash*

3.2 Comparison of CBR value between road bases aggregate mixed 9% bottom ash with the control (road base aggregate)

Figure 4 shows the comparison result of CBR value between the control (road base aggregate) and road base aggregate mixed with 9% bottom ash (BA). The finding indicates that the addition of 9% additives of bottom ash was giving the best result in term of CBR value about 210% compared to the control only 115.9%. It shows an increment percentage about 81% by using the additive of bottom ash compared to the control. It can be seen that the CBR value of road base depend on the presence of additive such as bottom ash because bottom ash considered as fine aggregate and this will fill the void and space between the road base aggregate materials.

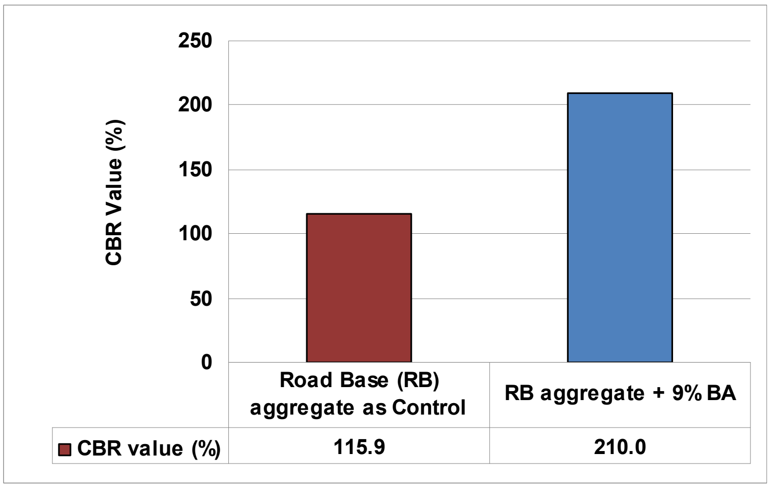

Figure 4. C*BR value for road base aggregate (control) and road base aggregate mixed with 9% bottom ash*

3.3 Comparison of CBR value between control (Road base aggregate) and substitutions of sand aggregate size in road base aggregate with bottom ash

Figure 5 show the comparison result of CBR value between the control (road base aggregate) and the substitutions of sand size (from 0.075mm to 2mm) in road bases aggregate with bottom ash. The results shows that the substitutions of sand size with bottom ash were give slightly give higher and best result of CBR value about 129.7 % compared 115.9% for control (road base aggregate only). This shows an increment percentage of CBR value about 12% to the control.

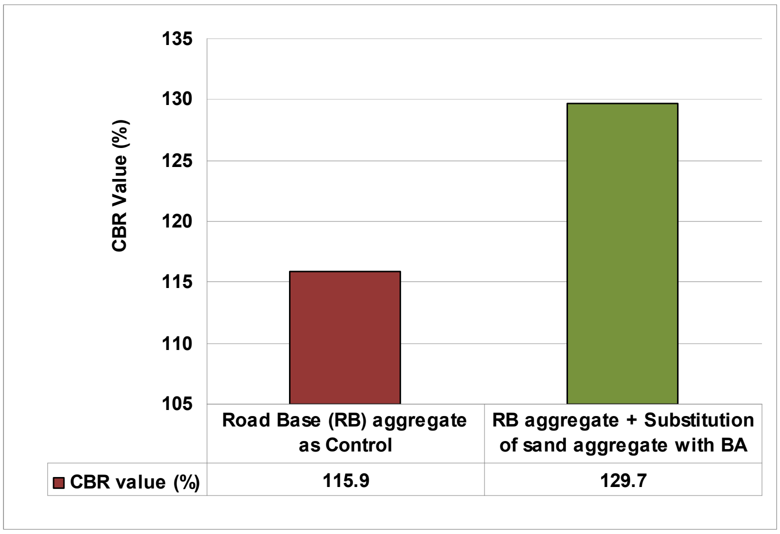

Figure 5*. CBR value for road base aggregate (control) and substitution of sand size in the road base aggregate with bottom ash*

3.4 Summarized of laboratory result

Figure 6 show the illustrated comparison of three laboratory result for CBR value between the control (road base aggregate) and road base aggregate mixed with 9% of bottom ash (BA) and bottom ash substituted to the sand size in road base aggregate. Among the three results shown, by using 9% additive of bottom ash were gives the better and higher result of CBR value about 210% compared to the control (about 115.9%) and compared to the substitution sand size with bottom ash (about 129.7%). It can be seen slightly increment percentage for substitution sand aggregate with bottom ash about 12% higher compared to control.

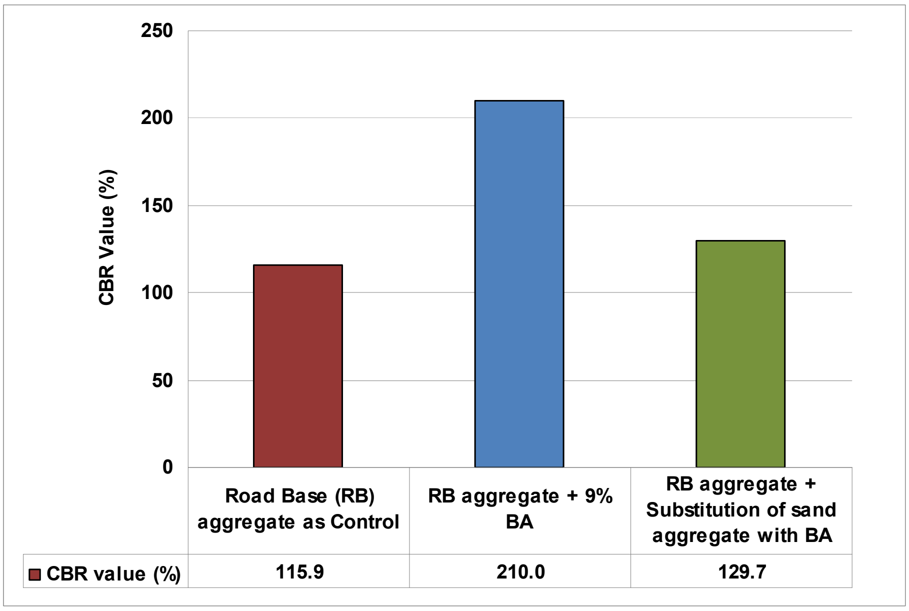

Figure 6. *CBR value for three experimental results*

4. CONCLUSIONS

Based on this study, bottom ash is suitable material to be used as road base additives. It was proved by experimental from laboratory result. By using of bottom ash (BA) as an additives material to the road base aggregate was improved the CBR value strength. Instead, bottom ash can be used as substitution of sand aggregate size (from 0.075mm to 2mm) in road base aggregate materials. These also were giving the higher of CBR value compared to control. Nevertheless, bottom ash (BA) more effective as additives and can be used as alternative additives materials to the road base aggregate to enhance and improve the CBR value. Thus this will reduce the construction cost and solving disposal problems.

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T146

STATIC LOADING ON DEEP BEAMS WITH LARGE OPENING WITH AND WITHOUT CFRP SHEETS AT BENDING ZONE

Preetpal Kaur A/P Ragbir Singh[1] and Nasir Shafiq[2]

[1, 2]Universiti Teknologi Petronas, Perak, Malaysia

[1]randhawa_preeti1985@yahoo.com.my, [2]nasirshafiq@petronas.com.my

ABSTRACT: In the construction of modern buildings, tall buildings and other industrial structures, pipes and ducts are installed to accommodate essential services such as water supply, sewerage, air conditioning, electricity, telephone and computer network. These pipes and ducts usually run horizontally and vertically. Pipes and ducts that have horizontal layout and run under the ceiling may sometimes penetrate through RC columns. These openings can cause RC beams to become weaker in load resistance due to shear and bending moment. This depends on the size, shape and location of the openings. The beam will reduce its stiffness under service load with higher deflection. Besides that, loads higher than the design service loads or chemical processes due to aggressive environmental conditions can damage the existing structures. This research focuses on static loading on reinforced concrete deep beams with openings. This research also studies the prospect of strengthening the beams by using external bonded CFRP in different combinations or arrangement to regain bending capacity that was lost due to the openings. The openings are circular, rectangular, square and elliptical shaped. Nine identical beams with and without opening were cast and tested under static loading with several different arrangements of CFRP sheets to regain the lost strength. Carbon Fibre Reinforced Polymer (CFRP) can increase the strength lost due to large opening with appropriate strengthening configuration and location of the CFRP. The usage is maximized by applying the CFRP perpendicular to the expected crack pattern on beams with large opening for bending zone.

Keywords: static loading, beam with opening, CFRP

1. INTRODUCTION

In the construction of modern buildings, tall buildings and other industrial structures, pipes and ducts are installed to accommodate essential services such as water supply, sewerage, air conditioning, electricity, telephone and computer network. These pipes and ducts usually run horizontally and vertically. Horizontal pipes and ducts that run under the ceiling may sometimes penetrate through RC columns. These openings will cause RC beams to become weaker in load resistance due to shear and bending moment. This depends upon the size, shape and location of the openings. The beam will have higher deflection under service load. This will occur when the beam stiffness reduces. In the continuous beam with opening the redistribution of moments and internal forces occurs. When a beam with opening has insufficient special reinforcement, the beam will reduce its strength to a critical degree.

Besides that, loads higher than the design service loads or chemical processes due to aggressive environmental conditions can damage the existing structures. The openings in the beams may be different in shapes and sizes. Prentzas (1968), in his extensive experimental study, considered openings of circular, rectangular, diamond, triangular, trapezoidal and even irregular shapes. Among all these openings, the most constructed ones are circular and rectangular. Circular openings accommodate service pipes, such as for plumbing and electrical supply. Rectangular openings accommodate rectangular air conditioning ducts. One of the ways to reduce possible stress concentrated at the sharp corners of rectangular openings is by rounding off these sharp edges. This will improve the cracking behavior of the beam in service. Mansur and Hasnat (1979) have defined openings such as circular, square or nearly square as small openings. According to Somes and Corley (1974), large circular opening has its diameter exceeds 0.25 times the depth of the beam web.

The carbon fibre reinforced polymer (CFRP) is one of the reinforced materials that are widely used for repairing all kind of RC structures. The CFRP is used as external reinforce to the concrete members, which is different from other reinforced material that is used to reinforce the concrete members internally. The material used in upgrading and strengthening structural capacity is CFRP. In the bridge construction, CFRP is most commonly used. CFRP strips are lighter, non-corrosive, and less intensive than the application of the steel plate or exterior post-tensioning. Structural elements can increase its strength by pasting CFRP externally. The lightweight of CFRP strips can provide ten times the tensile strength of steel. The capability of the CFRP to carry loads in tension can be a way to strengthen against flexural, shear or compression, depending on how the CFRP strips are oriented to the longitudinal reinforcing steel. Steel has superior qualities in terms of strength and compatibility with concrete where steel is an effective concrete reinforcement. However, steel is highly susceptible to oxidation when exposed to chlorides. Factors such as insufficient cover, poor design or workmanship, poor concrete mix and aggressive environment can break down the protection layer and may lead to corrosion of the steel rebars.

For several decades, Fiber Reinforced Polymeric (FRP) is a kind of polymer besides CFRP that is widely used in the aerospace industry. Recently both FRP and CFRP are becoming popular in the construction industry for strengthening purpose. This is due to CFRP has the advantages of compositing materials such as immunity to corrosion, a low volume to weight ratio, a high strength to weight ratio, and unlimited delivery length (in sheet form), thus eliminating the need for joints. The usage of CFRP as an effective mean of improving, upgrading and strengthening reinforced concrete beams. This CFRP improves the efficiency of construction work. This is due to CFRP is applied externally to the reinforced concrete beam whereas the reinforced can only be applied internally in the beam. By using conventional method, the beam has to be hacked of the reinforced before the reinforced can be applied internally in the beam. This will create problem, as it has to rectify the structurally deteriorated or functionally obsolete reinforced concrete beam. This method wastes a lot of money. The following are some advantages of using CFRP for bending strengthening in the critical bending zone of the beam with large opening:

i. Tailorability; the CFRP sheets can be arranged to the loading condition to optimize the performance.
ii. It has a low weight that reduces transportation expenses and allows for some prefabrication that consequently reduces time at the job site.
iii. CFRP does not have a yield limit and more or less elastic up to failure.
iv. CFRP has high elastic modulus and high strength in both tension and compression mode.

The strongest and stiffest reinforcing fibers for polymer composites are carbon fibers. The most commonly used are glass fibers. These fibers are made of pure carbon in form of graphite and the fibers are low in density. These fibers also have a negative coefficient of longitudinal thermal expansion. These carbon fibers are very expensive and can give galvanic corrosion in contact with metals. Therefore, they are generally used together with epoxy where high strength and stiffness is required. Generally, there are eight possible failure modes in CFRP strengthened reinforced concrete beams. Not all of this eight failure modes can be observed in this research. For a simply supported reinforced concrete beam strengthened by CFRP the following four modes will most likely occur:

i) CFRP rupture in tension zone
ii) Concrete crush in compression
iii) Delamination between CFRP and concrete
iv) CFRP peeling off in curtail zone resulting from a combination of shear and tensile stresses in the plane of the longitudinal steel bars.

This research focuses on static loading on reinforced concrete deep beams with openings. This research also studies the prospect of strengthening the beams by using external bonded CFRP in different combinations or arrangement to regain bending capacity that was lost due to the openings. The material used to develop the strength of the beam is CFRP sheets. The openings are circular, rectangular, square and elliptical shaped. The purposes of these large openings in floor beams are to facilitate the passage of utility pipes and service ducts. Nine identical beams with and without opening were casted and tested under static loading with several different arrangements of CFRP sheets to regain the lost strength.

Figure 1. *Experimental Setup*

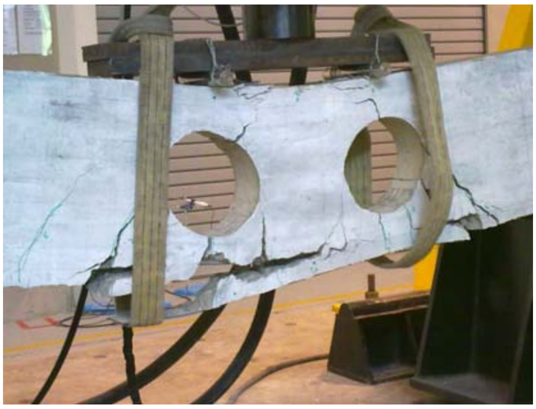

***Figure 2**. CFRP Debonding After Static Fracture of Reinforcing Steel*

2. METHODOLOGY

2.1 CASTING 9 CONCRETE DEEP BEAMS

In this research, the bending moment loss due to the large web opening is to be studied. By using CFRP it can be determined how much strength the beam regained even though there are large opening in the beam. As CFRP is an expensive material, the significance of using CFRP to regain the strength should be relatively feasible to be implemented in the industry. Nine beams were cast to test the strength of the beams. One of these beams is solid beams (reference beam) that have no openings. Two beams has circular opening (270mm diameter) in the middle of the beam and the other two has square openings (240mm X 240mm) in the middle of the beam. Two beams have rectangular opening (700mm X 500mm) in the middle of the beam and the last two has elliptical openings in the middle of the beam. All these nine beams were divided into two group that is CFRP and non-CFRP group. The CFRP group beams were tested under static loading that will have circular and square beams strengthen with CFRP sheets. The non-CFRP group beams tested under static loading that will have rest of the beams tested without strengthen with CFRP sheets. The criteria's for 9 Test Beams is shown in Table 2.

The concrete strength, fcu is 35+/- 5Mpa. The beams have 2T12 rebar as the bottom rebars and 2T10 rebar at the top of the beam. These rebars act as the main bars in the beams. The beams also have stirrups of R6 rebar with spacing of 300mm center to center. The beams were cast using the ready mix concrete with the strength of 35Mpa. The concrete cover used was 10mm top and bottom. First, the formwork of the beams was prepared following the dimension of the beams. The dimensions of all 9 beams are 2500mm X 500mm X 150mm. For the square opening, wooden framework was used to ensure the hollow section is maintained during casting. As for the circular opening, the hollow section was maintained by using a pipe of external diameter Φ270mm. Formwork must be applied with grease before the pouring of concrete. When the formwork for all the beams is done, with all the rebar and stirrups are placed as designed, then ready mix concrete is poured in the formwork. A poker was used to apply vibration the mixture in the formwork. Three layers of mixture were placed in the each formwork and poker was used to vibrate after each layer was placed. A poker vibrator was used to ensure air is not entrapped and to avoid honeycomb in the concrete. However, the vibrator should not be used too long (more than 1 minute) which could lead to bleeding and segregation problems in the concrete later on. The top layer should not be vibrated, and surface to be finished. Then the beams were left for three days in the formwork to harden.

After three days, the formwork of the beams was open and the beams were place at the corridor of the lab for curing purpose by water. The beams are big in size and there are no curing tanks big enough for these beams. Therefore, the beams were cover with sacks and water daily. The functions of the sacks are to keep water so that the beams will not over dry and to avoid surface cracks at the surface of the beams. The extra ready mix concrete were placed in 15 cubes' moulds 100mm X 100mm. A poker was used to apply vibration to the mixture in the mould. Three layers of mixture were place in the each mould and poker was used to vibrate after each layer was place. The vibration was only given for 2 seconds. It cannot be more than 2 seconds, as it will enhance watering and segregation in the mixture. This will give wrong result later when tested under compression test. Then the cubes were left one day in the mould to harden. The next day, the moulds were open and the cubes were place in water for curing purpose. On the 3rd day three cubes were tested under compression test. Then the other three cubes were tested on the 7th day. The rest nine cubes were tested on the 28th day. The major purpose of this cube testing is to find out the concrete strength, fcu on the 28th day. The concrete strength, fcu supposedly should be 35+/-5Mpa and this was achieved in this experiment.

The beams were tested after 28 days using the self straining loading frame machine. The solid beam is tested first. Then the beams with the circular and square opening in the middle were tested continued with the rest of the beams. The CFRP is pasted based on the crack pattern that is obtained from the circular and square opening beams tested under static. This will increase the strength of the beams with circular and square opening. Beams with rectangular and elliptical opening have higher static load then the solid, therefore no need to paste CFRP sheets. Below shown compression test result for cubes in Table 1 and average compression strength of concrete increases as the Day Increases in Figure 3,

Table 1. *Compression Test Result for 15 Cubes*

Days	Cube	Maximum Load (kN)	Compressive Strength (N/mm^2)
3	A	147.40	14.74
	B	143.10	14.31
	C	141.10	14.11
7	A	185.90	18.59
	B	189.90	18.99
	C	189.50	18.95
28	A	350.52	35.05
	B	350.85	35.08
	C	351.42	35.14
	D	350.02	35.00
	E	35.12	35.12
	F	350.74	35.07
	G	350.94	35.09
	H	350.53	35.05
	I	351.26	35.13

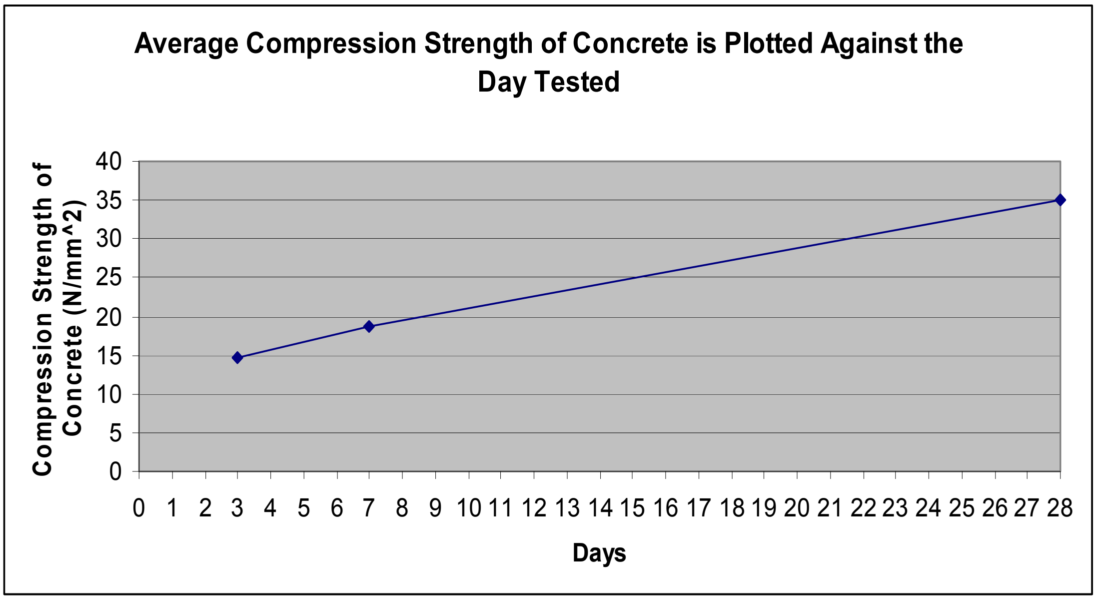

Figure 3. *Average Compression Strength of Concrete increases as the Day Increases*

Table 2. *Criteria's for 9 Test Beams*

Beam No	CFRP Strengthening	Shape of Opening	Spam of Beam (mm)	Design Strength (MPa)	Number of Openings	Location of Opening	Type of Loading
1	No	None	2300	+/- 35	0	No	Static
2	No	Square (240 X 240 mm)	2300	+/- 35	2	850mm from the support	Static
3	Yes	Square (240 X 240 mm)	2300	+/- 35	2	850mm from the support	Static
4	No	Circular (Φ270 mm)	2300	+/- 35	2	850mm from the support	Static
5	Yes	Circular (Φ270 mm)	2300	+/- 35	2	850mm from the support	Static
6	No	Elliptical (Φ125 mm, h=500mm, d=250mm)	2300	+/- 35	1	875mm from the support	Static
7	No	Elliptical (Φ125 mm, h=500mm, d=250mm)	2300	+/- 35	1	875mm from the support	Static
8	No	Rectangular (700mm x 250mm)	2300	+/- 35	1	900mm from the support	Static
9	No	Rectangular (700mm x 250mm	2300	+/- 35	1	900mm from the support	Static

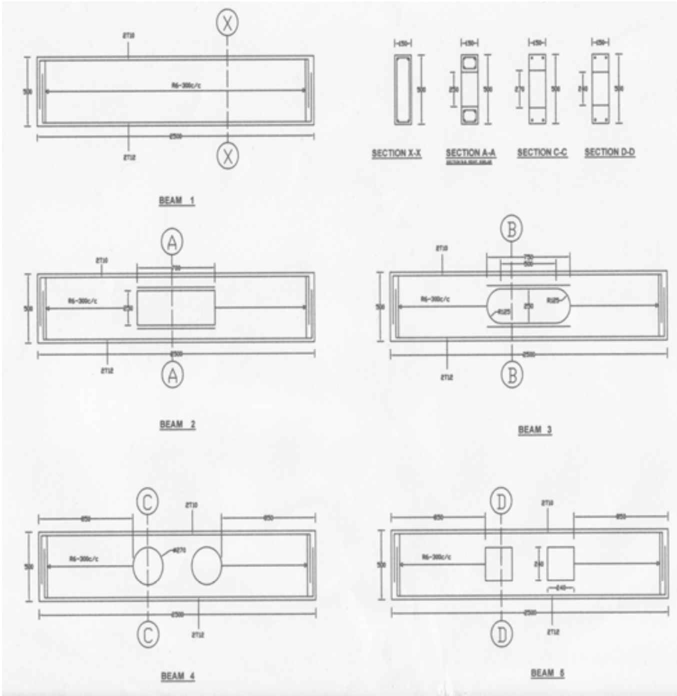

Figure 4. *Drawings of the 9 tested beams*

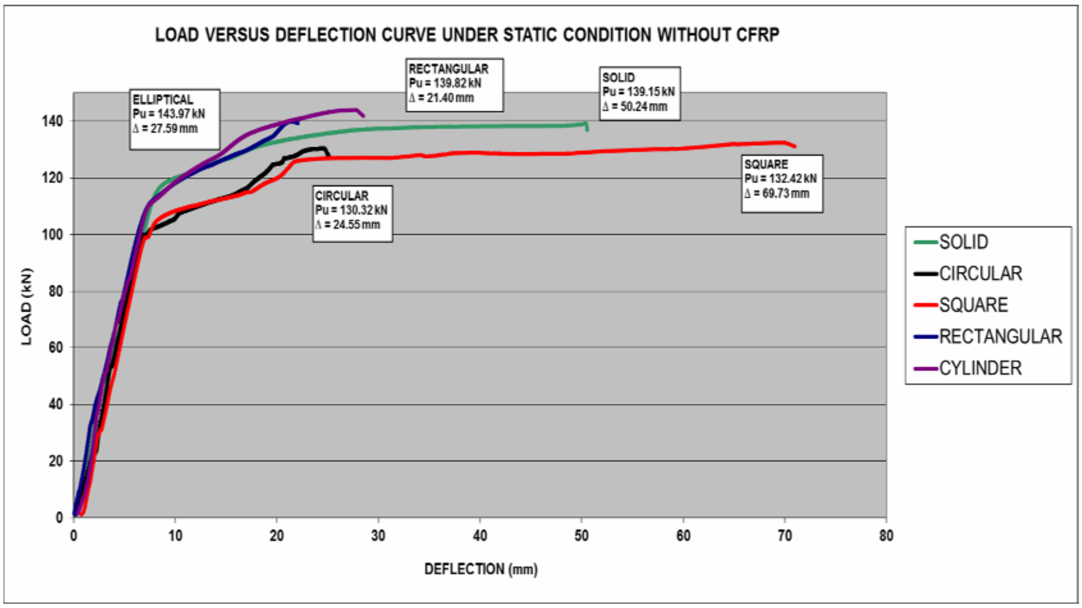

Figure 5. *Graph Plotted for All Tested Beams Under Static Condition Without CFRP Sheets*

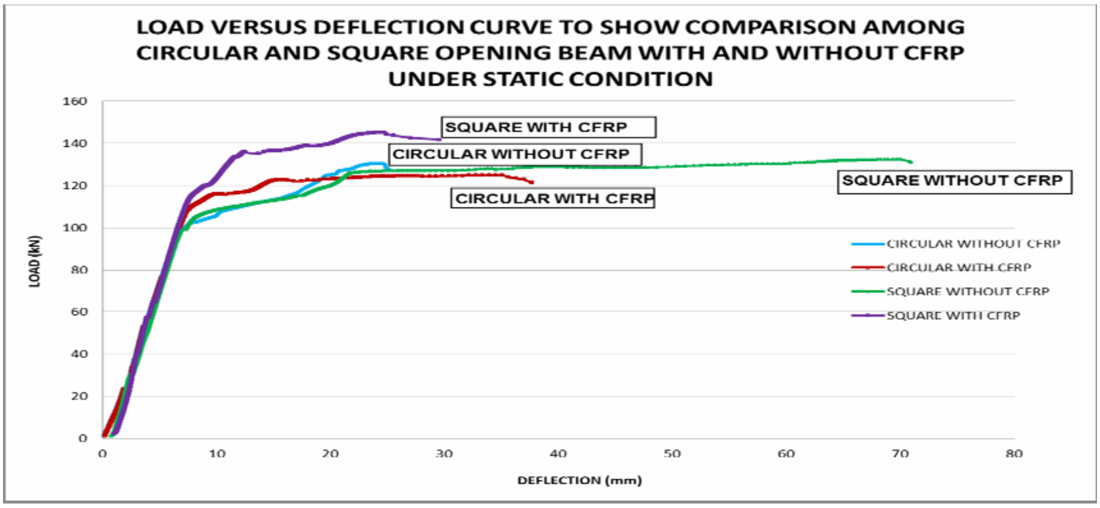

Figure 6. *Graph Plotted to Show Comparison Among Circular and Square Opening Beam With and Without CFRP Sheets Under Static Condition*

Figure 7. *Failure of Solid Beam Under Static Without CFRP Sheet Under Static Condition*

Figure 8. *Failure of Square Opening Beam Condition Without CFRP Sheet*

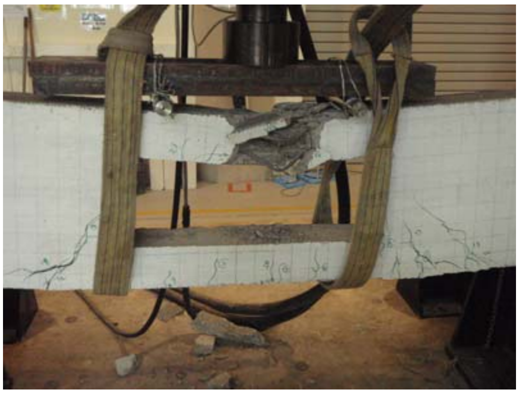

Figure 9. *Failure of Rectangular Opening Under Static Condition Without*

Figure 10. *Failure of Square Opening Beam Beam Under Static Condition With CFRP Sheet CFRP Sheet*

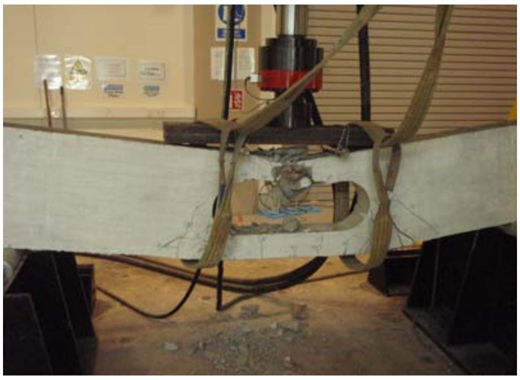

Figure 11. *Failure of Elliptical Opening Beam Under Static Condition Without CFRP Sheet*

3. DISCUSSION

This research is focused on the effects of large opening in deep beams under static condition for bending zone as well as the configuration on CFRP used for strengthening the beams with openings. From the results obtained after carrying out the lab work for these nine beams, the summary of ultimate load is summarized in Table 3 below:

Table 3. *Summary of Static Load Test (Beams Without CFRP Sheets)*

Beam Characteristic (Beams without CFRP strengthening)	Ultimate strength under static condition (kN)	Deflection (mm)	% lost or gain in strength due to opening in critical bending zone
Solid beam (Reference Beam)	139.15	50.24	Nil
Beam with Circular Opening	130.32	24.55	-6.5%
Beam with Square Opening	132.42	69.73	-6.3%
Beam with Elliptical Opening	143.97	27.59	+3.5%
Beam with Rectangular Opening	139.82	21.40	+0.5%

From Table 3, it can be observed that the deflection increase when there is square opening in the beam. The other opening beams have less deflection compared to solid beam. Deep beams are suitable to have large openings compared to solid beam. Deep beams with elliptical and rectangular opening can stand higher ultimate load and has less deflection compared to solid beam. The only opening beam that is weak is square opening beam. Therefore, deep beam with square opening is not suitable to have square opening in the middle. From Table 3, it is shown that circular and square opening beams lost its strength about 6-6.5% whereas elliptical and rectangular opening beams gain its strength about 0.5-3.5% compared to solid beam. Here can be conclude that deep beams are suitable to have large opening in the middle of the beam because the beam acts as a spring and allow energy to dissipate when load is being applied on it. The lost of strength is not much therefore, large opening in deep beams works well.

Solid beam is heavy in mass and there is no opening for the energy to dissipate when load is being applied on it. Since square opening beams are weak, the only way to reduce the stress concentrated at sharp edges is by rounding off the sharp edges. Rectangular and elliptical opening beams have higher yield strength compared to square and circular opening beams. All the beams have good ductility from the elastic point compared to solid beam by referring the Figure 5. Since square and circular opening beams lost its strength, therefore CFRP sheets are paste to gain its lost strength. The configuration of CFRP sheets pasted for square opening beam is shown in Figure 10. The CFRP sheets are paste front, back and bottom of the square opening beam. For circular opening beam, the CFRP sheets are only paste at the bottom (minimum usage of CFRP).

From Figure 6, it shows that square opening beam gain its strength up to 10% (145.24 kN) compared to non-CFRP sheets pasted square opening beam. CFRP pasted for square opening was perpendicular to the crack pattern obtain from the beam without CFRP sheets, which was tested earlier. The CFRP sheet pasted for circular was in minimum quantity therefore, not much strength increased for this beam. CFRP pasted for circular opening was not perpendicular to the crack pattern obtain from the beam without CFRP sheets, which was tested earlier. Carbon Fibre Reinforced Polymer (CFRP) can increase the strength lost due to large opening with appropriate strengthening configuration and location of the CFRP. The usage is maximized by applying the CFRP perpendicular to the expected crack pattern on beams with large opening for bending zone.

4. CONCLUSIONS

The conclusions of this study are listed below:

1. The shape of opening does affect the loading capacity of the beam under bending zone. Elliptical and rectangular opening beams lost less strength compared to solid, square and circular opening beams. Square and circular opening beams lost more strength compared to solid beam. It means that the critical openings are square and circular for deep beams under static loading at bending zone. Rectangular and elliptical opening beams are not critical for deep beams under static condition at bending zone.
2. The bending cracks for beams with square and rectangular opening were massive compared to the beams with circular and elliptical openings.
3. Carbon Fibre Reinforced Polymer (CFRP) can increase the strength lost due to large opening with appropriate strengthening configuration and location of the CFRP. The usage is maximized by applying the CFRP perpendicular to the expected crack pattern on beams with large opening for bending zone.
4. The beams increase its strength by increasing the bending capacity by using CFRP sheets even though there are large opening in the deep beam.

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SUBURBAN NEIGHBORHOODS SATISFACTION TOWARDS CREATION HIGH QUALITY OF URBAN LIFE THROUGH NEW URBANISM

Rozina Farajollahzdeh[1], Hasanuddin Lamit[2]

[1] Universiti Teknologi Malaysia, 81310 UTM Skudai, Johor, Malaysia

[2] Landscape Departments, Faculty of Built Environment, Universiti Teknologi Malaysia, Skudai, Malaysia

[1]rozarc2000@yahoo.com, [1]frozina2@live.utm.my , [2]inadean@hotmail.com

ABSTRACT: This paper examine the characteristics associated with implementing the principles of new urbanism through creation of a walkable neighborhood centre towards neighborhood satisfaction in Taman Sutera Utama, Skudai-Johor. The main purpose of this research is to explore and evaluate the main elements of creating a neighborhood centre towards achieving neighborhood satisfaction based on walkability. Results analyzed through qualitative and quantitative methods which show the high level of neighborhood satisfaction among permanent, temporary and transit residents in this region. The main reason of this satisfaction is walkability, mixed-used and diversity, sense of safety, sense of security.

Keywords: walkability, neighbourhood satisfaction, neighbourhood centre, travel behaviour, safety, security.

1. INTRODUCTION

Walkabiliy is one of the core aspects of ‘Travel Behavior’ and ‘Principles of New Urbanism’. Urban professional such as urban planners, urban designer and transportation planners in travel behavior field of research are focusing on walking and walkability issues in urban design. Such issues cannot be resolved through intuition and experience rather on scientific research evidence which may convince policymakers to apply walkability principles in urban management.

There is an evaluation about the rate of implementation of the principles of new urbanism such as: Walkability; Connectivity; Mix-used and Diversity; Mixed Housing; Quality Architecture and Urban Design; Traditional Neighbourhood Structure; Increased Density; Green Transportation ;Sustainability; Quality of Life. The purpose of this study is to explore the main elements of neighbourhood centre towards achieving neighbourhood satisfaction and examine the rate of implementation of new urbanism’s principles.

Controversial and critical issue of walkability as considerable human behaviour approach in town design and planning is evident in the trend toward behaviour-oriented investigations. Decision behaviour analysis (Handy, 2004; Mokhtarian, 2005; Coa, 2006) and activity-based modelling (Krizek and Waddell, 2003) are the most particular areas of concern for walkability inquiry in urban design and urban planning.

Handy (1996a) states that research into urban form and travel behaviour should be modified to focus from changing behaviour to determining initiatives to provide choices for residents to walk. Current urban design philosophies tend to emphasize accessibility rather than mobility by building the environment to offer choices and alternatives (Levine, 1999). A combination of urban design, land use zoning, and transportation systems intended to promote non-motorized travel may create healthier and more liveable communities (Handy et al., 2002). Meanwhile, refined decision making approaches will be necessary to achieve this, including better measures of the physical and environmental indicators towards walkability, accurate and comprehensive data on non-motorized travel, and spatial matching of travel data. Shay et al. (2006) identified differences between the walk and drive modes in the number of internal daily trips within a neighbourhood, where walk trips decrease considerably with distances between residences and the commercial centres. The complexity of travel behaviour and its multiple measures make it difficult to express how the environmental factors affect trip generation and travel distances.

Campbell et al., (1976) and Francescato et al., (1987; 2002) described the level of "satisfaction" conceptually as the degree to require meeting of need, in contrast with some other kinds of attitudinal queries that a number of argue sketch more on emotional, normative, or cognitive beliefs. Since reported levels of satisfaction are intrinsically random scales, they are mainly helpful for comparing relative levels crossways sections of an example and for investigating what variables are connected with higher satisfaction ratings. The combination of reflection and perception of respondents' preference was studied by Lovejoy, K; Handy, S; Mokhtarian, P (2010) to achieve satisfaction levels.

Dependent variable such as measuring the levels of neighbourhood satisfaction include: *attractiveness; quiet; liveliness; big yards; safety; mixed-use; good infrastructure.* (Marans and Rodgers, 1975; Galster and Hesser, 1981; Weidemann and Anderson, 1985; Lu, 1999).

2. METHODOLOGY

2.1 NEIGHBOURHOOD AND RESPONDENT SELECTION

In this research the method used to obtain data is questionnaire survey. The survey contains mixed mode of qualitative and quantitative approaches on data gathering from the target population in an urban neighbourhood of Taman Sutera Utama,Skudai-Johor. The site study is the 400 meters (5min.walking distance) from A as focal point in a neighbourhood centre. Randomizing the election of respondents procedures help control error from sources which the research might not anticipate (Zeisel, 2006). Figure 1 shows the study area.

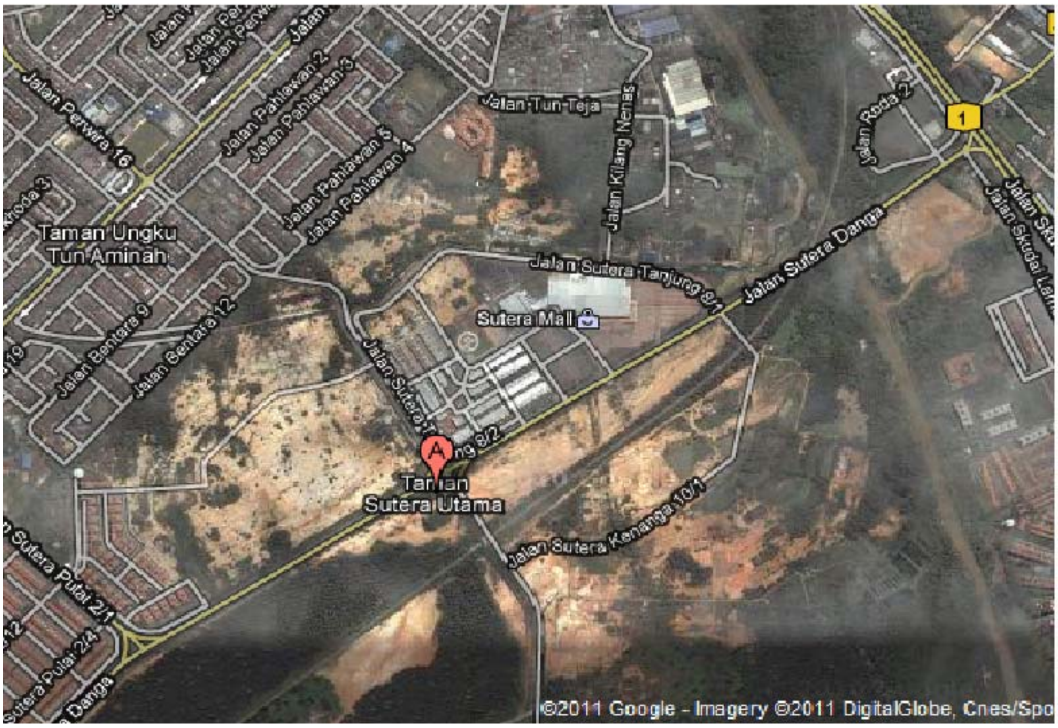

Figure 1. *Study Area*

2.2 STUDY COMPONENTS.

This study evaluated the type of differences in design characteristics (environmental analyses) and used these to interpret self-reported travel behaviour provided by residents. For example, distances between homes and everyday facilities such as convenience stores were related to self-reported style choices.

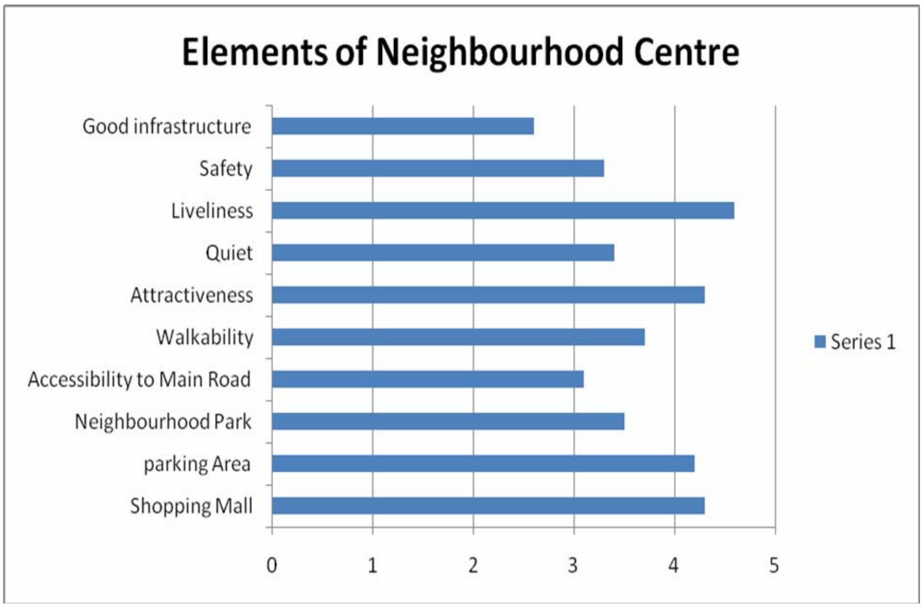

Figure 2. *Preferable Elements of Neighbourhood Centre by Residents and Transit*

Table 1. *Diverse Walking Behavior and Influencing Factors in Resident 'Walking Decision Making Towards Specified Shopping Centers.*

Name Shopping Centre	Sense of safety	Sense of security	Comfort	Convenience	Visual Interest
Sutera Mall	**75.28%**	**73.67%**	**52.37%**	**56.78%**	**80.87%**

Table 2. *Walking Behavior Factors Impact on Residents before and after Walking Decision Making*

Name of Shopping Centre		Sutera Mall	
		Before	After
a. Sense of safety	a.1.Traffic speed and pedestrian crossing	65.67%	46.75%
	a.2. crossing facilities	63.14%	40.12%
	a.3. Facility for sidewalks	42.8%	25.23%
b. Sense of Security	b.1.Security at day	50.76%	32.13%
	b.2. Security at night	54.89%	34.24%
c. Comfort	c.1.Physical comfort	46.76%	27.45%
	c.2.environment effects	57.34%	31.65%
d. Convenience	d.1.Functionaly of sidewalk with diverse activities	48.32%	10.27%
	d.2.Easy access and crossing	62.12%	10.76%
e. Visual Interest	e.1.Street-Scale and Enclosure	34.54%	29.45%
	e.2.Variety	72.14%	39.43%

3. CONCLUSIONS

Data gathered from the surveyed areas were analyzed respectively through the method of 'Analytical Hierarchy Process'. The model shows the five walking behaviour factors which are *Sense of safety, Sense of security, Comfort, Convenience and Visual Interest*. The data gathered from the surveys conducted within the residential areas around the shopping centres of Sutera Mall, shows that this shopping centre with the percentages of 87.08% is more preferable than others specially by residents of Taman Sutera Utama. In exploring the main elements of neighbourhood centre towards achieving neighbourhood satisfaction, some qualitative and quantitative methods were conducted. Figure 2 identifies the preferable elements of neighbourhood centre by residents and transit visitors while table 1 represents the diverse walking behaviour and influencing factors in resident 'walking decision making towards specific shopping centres and table 2 describes walking behaviour factors impact on residents before and after walking decision making.

Based on the results, the most notable and cited reasons of residents and transit visitors towards achieving neighbourhood satisfaction are, existing walkable main elements such as shopping mall, parking area, neighbourhood park, public transport, attractiveness, liveliness, sense of safety and security. Therefore, In reference to the main aim of this paper, raising the rate of implementation of the principles of new urbanism will improve the quality of life and neighbourhood satisfaction.

In summary, the scope of investigation and study on travel behaviour is naturally vast and the range of sensitive factors is large which are influenced by both urban form and personal characteristics. The existing literature of interaction between environment and travel behaviour is not entirely comprehensive, because of the complexity of both urban form and human travel behaviour subjects. New approaches or techniques to measure this relationship are needed.

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ASSESSMENTS OF GREEN BUILDING INDEX RATING FOR CONCRETE AND ITS EFFECTS ON GLOBAL WARMING

Alonge O. Richard[1] and Mahyuddin B. Ramli[2]

[1,2]Universiti Sains Malaysia, Penang, Malaysia.

Olarichy07@yahoo.com[1] , mahyudin@usm.my[2]

ABSTRACT: Carbon dioxide is considered worldwide as the major cause of global warming and the effects thereof on the surface of the Earth. Studies have shown that production of Portland cement alone accounted for approximately 5% of the total global Carbon-dioxide (CO2) emissions through various human activities. A further study shows that a tonne of concrete produced carbon dioxide of between 0.05 to 0.13 tonnes approximately ninety-five percent (95%) of the total estimated carbon-dioxide emissions derived from production of concrete. Gore (2003), considered buildings as one of the largest end consumer of energy. It was established that building sector accounts for 25 to 40% of the final energy demand. This paper study the elaborated efforts made so far in the production of green concrete and its emphasis on the use of GREEN BUILDING INDEX as a comprehensive rating system to evaluate the extent of green nature properties of green concrete, the environmental design and the performance. All these will be evaluated to establish their correlations with fundamental sustainability requirements.

Keywords: green concrete, global warming and green building index rating.

1. INTRODUCTION

The increasing emissions of anthropogenic greenhouse gases has being causing climate changes over some decays now and this fact has being established with measurement through researches and studies that there is global growth of CO2 concentration in the atmosphere.

Concrete is considered worldwide as the most important building materials and also the most common materials used in the construction of building or civil engineering structures. The reason behind this are not farfetched; if concrete is properly and adequately designed and produced, it possess some valuable, conducive and economic features through its properties chemically, physically and mechanically, hence its popularity in the world, its durability and adaptability is worldwide friendly. As a result of its environmental impact also, it remain one of the most costly materials of construction. Concrete is mouldable, affordable highly resistance to fire attack, easy to work with and readily available. It can be worked upon to meet up with any specifications or requirements, easily modifiable hence term engineering materials.

(Neville, 1996), revealed that an ordinary concrete typically contains about 12% cement and 80% aggregates by mass. Likewise,(Mehta,2001), claimed that the global construction industry uses approximately 1.6 billion tonnes of cement and 10 billion tonnes of sand, gravel and crushed rock every year. He further explained that the world's yearly cement production of 1.6 billion tones accounts for about 7% of the global loading of carbon dioxide CO2 into the atmosphere.

(Mehta,2001), says mining large quantities of raw materials for the production of cement such as limestone and clay and fuel such as coal often results in extensive deforestation or denudation and top soil loss. All these need of vast quantity of natural resources to produce an estimated billion of tons of concrete each year resulted in great impact on the immediate environment. Apart from the estimated 7% of the global loading of carbon dioxide CO2 into the air annually, the cement industry through the production of Portland cement is also considered to be energy intensive. In addition to these, quite a large quantity of water are required for the production of concrete and this is considered burdensome considering the availability of good and fresh water suitable for concrete production.

In addition, concrete structures have life span, a service lives of several decades up to a century or more. Those structures that failed after achieving their life span and could no longer fulfill their original purpose are demolished. The demolition and the disposal need of the debris or left over of concrete constitute another environmental problem. Construction as well as demolition debris constitute a considerable fraction of solid waste in developed countries even as concrete is considered as the largest single components of it all.(Mehta,2001), claimed that the annual worldwide output of concrete and masonry rubble has been estimated roughly as one billion tonnes. The continuous use of natural aggregate and the use of cement in the production of concrete conventionally likewise has negative effect on the environment. This effect on the habitat as stated by (Winfield and Taylor, 2005), is the extraction of aggregates from pits and quarries results in the destruction of the natural habitats of many organisms and in the disturbance of the pre-existing stream flow and water resources. There are great impact of the natural aggregate production on the quality of both groundwater and running surface water. The flow patterns of the water are disturb – in the course of natural aggregate production and likewise the ground water storage capacity deposits are considered as the reservoirs for underground water.

After quarrying, the natural aggregates have to be transported; the cost of transporting the already crushed aggregates from quarry to the place or location of use visa a vise the transportation of the ready mix concrete has great financial impact. Despite, all these short coming of concrete production impart on the environment, concrete is still considered as an inherent environmental friendly structural materials.

Many research work and studies have been done and still in the offing to see that concrete production achieve greenish features hence green concrete. Among the contemporary research studies is the use of some cementitiouse materials such as fly ash, ground granulated blast furnace slag and silica fumes as a substitute to Portland cement in the production of concrete. In some part of developed countries, many achievements have been reached with partial substitute of Portland cement with some cementitiouse materials in the ratio of either 70% of cementitiouse materials and 30% of Portland cement or 60% of cementitiouse and 40% of Portland cement. For instances, in Denmark according to (Ecoserve,2006b),the blending of Portland clinker with supplementary materials mainly takes place at the concrete plant whereas the trend in central and southern Europe goes towards blended cements where the blending take place at the cement plant.

However, the application of various types of supplementary materials depends strongly on the availability and traditions in the various countries. For instance in North America, the concept of environmentally friendly concrete is closely related with high volume fly ash concrete where about half of the cementitiouse materials consist of fly ash (Malhotra 2006, Obla et al. 2003).With this in mind, this paper seek to assess the green nature or features of this environmental friendly concrete using green building index rating and look at its effects on global warming.

2. GREEN BUILDING AND GREEN BUILDING INDEX

Green building is generally classified as the process of building and structure production that are environmentally responsible and efficient resourcefully throughout the building or structure life’s span beginning from the sitting to the deconstruction. It can also be refered to as high performance building. (Kibert, 2004), defined a sustainable or green building as a subset of sustainable construction, representing simply the structure. Years later, he went further to explain that sustainable construction most comprehensively addressed the ecological, social and economic issues of a building in the context of its community (Kibert, 2007). (Ahmad S,2010), defined Green Building Index as the comprehensive rating system for evaluating design and performance of buildings based on six (6) main criteria using Malaysia GBI;

- Energy efficiency which carry 35%
- Indoor environmental quality 21%
- Sustainable site planning and management 16%
- Water efficiency 10%
- Innovation 7%
- Materials and resources 11%

The actual reason for the construction of greener building is quite important and this has to do with the main fact that we need to live most lightly on the earth surface as much as possible. The impact of we as human being with our various activities has on the earth's ecosystem cannot be over emphasized and the major factor of concern in most of our activities and operations boils down on the use of energy production, dissipation, use and consumption of energy from various sources. Green building target on increasing the resource use energy efficiency, water and materials efficiency and at the same time reducing the impact of these various activities on human health, well being and the environment during the life time of such building and these can be achieved through better sitting , green nature driving design, construction, operation, maintenance and removal.

In the building and construction sector, 40% of the world's energy are consumed,12% of water are also consumed with 40% of the waste sent to landfill, all these contribute majorly to the global problem. The design and choice of materials has most significant ways of affecting our environment, likewise the quantity of energy embodied in the building materials themselves, their transportation and their assemblage duly call for concern in term of global warming. These energy consumption and dissipation has a direct correlation or effect on our environmental quality because of the environmental pollution through the emission of greenhouse gases and other types of emissions. However, the building and construction sector can as well be a major playmaker in the solution of world acclaimed global warming problem.

International researchers confirmed that green buildings consume less energy, water and generate less waste thereby create a healthy and productive environment. It has being claimed through the chairman, world green building, Tony Arnel, that research had shown that green building practices can reduces a building costs by as much as 9% building values by 7.5% and realized a total 6.6% increase in return on investment, so, green building don't just make a sound ecological and environmental sense, they make sound economic sense too.

In line with growing green building delivery system worldwide, quite a number of green building assessment tools have been invented by several international and national research organization, institute and scholars and most of these assessing tools or ratings have being tested and accepted although many are still being improved upon yearly or quarterly. The area of focus of most or majority of these rating tools are;

(a) Energy efficiency of all components of the building, i.e. all the elements, assess one after the other.

(b) Indoor environmental quality

(c) Sustainable site planning and management

(d) Water efficiency

(e) Innovation

(f) Materials and Resources.

And the main objectives of most, if not all of these green building ratings are;

- To evaluate the overall performance of the green building by considering all the elements one after the other
- To monitor and coordinate the process of green building production in other to achieve the basic fundamental principles of green building which are economic growth, environmental and ecological balance equity and social advancement.
- To increase and improve on the evolution of green building construction.

2.1 NOTABLE GREEN BUILDING INDEXES IN THE WORLD

There are several green building index rating tools developed across the whole continents of the world and many are still being developed .There are few of these rating tools that are considered as the mainstream building index tools for the general use in the production of green building, however, it was found that the generation of this rating tools has to be localized as it has much to do with locality, culture and regional system of building production.

Among these major green building tools are;

(1) LEEDS, which is interpreted to mean Leadership in Energy and Environmental Design. The United State Green Building Council (USGBC) first developed this system of rating in the year 1998. It was developed with a target of evaluating the environmental performance of a green building over the whole life span. Although this version has being improved upon, as we now have LEED (V.2.0) in the year 2000 and LEED (V.3.0) in year 2009. This current one has larger advancement than LEED (V.2.0), although LEED (V.2.0) is still more prominent in use in the prevailing market of green building. (http://www.usgbc.org.2011)

(2) CASBEE, this is developed in Japan by Japan Green Building Council (JAGBC) in the year 2001 and her subcommittee that includes the government official and the academicians. This green building tool adopts the value of building environmental efficiency in the evaluation used.

BEE = Building environmental quality and performance / Building environmental loadings. (http://www.ibec.or.jp/casbee, 2011)

(3) SBTOOL, this tool is designed with two focuses, which are to assess the environmental and sustainable performance of building. It started in 1996 by a group of more than a dozen team from different countries and later handed over to international initiatives for a sustainable built environment (iisbe) in the year 2002. Initially it was called GBTOOL and it was designed compel the users to reflect the different priorities, technologies, building traditions and cultural values of various regions of the world. It make use of a Microsoft excel workbook configured to fit into any construction condition of all regions of the world. (http:// www.iisbe.org. 2011)

(4) BREAM, this was developed in the United Kingdom in the year 1990 by the Building Research Establishment. The BRE is owned by the foundation for the Built Environment, a registered charity in the UK, and it provides a range of consultancy, testing and commission research services for all aspects of the built environment. (Bunz, Henze, et al. 2006) .This happen to be the pioneer building index tool in the whole world and this tool make use of environmental assessment method for buildings. A review of this tool was launched in 2008 and two major changes was introduced which include, a new two stage assessment process which are design stage and post construction, the second is the introduction of mandatory credits. (http://www.bream.org.2011)

(5) ESGB, In the year 2006, China ministry of housing and urban rural development came up with education standard for green building and the china government has made this to become a national standard named GB/T 50378- 2006, the ESGB consist of six divisions, each with 3 sub divisions.

(6) BCA- GM is another tools that was launched in Singapore in January 2005. This tool was supervised and endorsed by the National Environment Agency with main aim of the production of environment friendly buildings. BCA, green mark consist of assessment criteria for two main categories which are new building and existing building and the criteria is sub divided into two categories BCA – GM non residential building which is the latest version and BCA- GM for residential building. (http://www.bca.gov.sg/green mark 2011)

(7) MALAYSIA GBI, this building rating was generated in year 2008 and was launched in 21st of May 2009. It was developed for new buildings with two ratings one for residential and another one for non-residential. It was created to provide the building industry a common and verifiable mechanism to benchmark buildings within the Malaysian context. It was formed to set out a vision for sustainability within the built environment and provides guidance that will assist end users to deliver sustainable townships. It was developed specifically for the Malaysian tropical climate, environmental and developmental context and social needs and created to;

- Define green buildings by establishing a common language and standard of measurement

-Promote integrated, whole building designs that provides a better environment for all

-Recognize and reward environmental leadership

-Transform the built environment to reduce its negative environmental impact

- Ensure new buildings remain relevant in the future and existing buildings are refurbished and upgraded to improve the overall quality of the building stock. (http://www.green buildingindex.org/indexhtml.2011).

3. METHODOLOGY OF RATING CONCRETE USING GREEN BUILDING INDEX.

Concrete is considered as the most important building materials worldwide because of the readily available of its constituents. It consists of aggregates, which can be coarse or fine as the case may be, cements and water. We have different types of concrete, no fine aggregate concrete and lightweight aggregate concrete. Lightweight concrete is a major improvement through continuous research on the conventional concrete over the years, all to meet up with the global solution of reducing gas emission, which makes the globe to warm.

Looking at the greenish features of concrete in the light of some of the green building index highlighted above, this paper come to conclusion that concrete can thoroughly be green hence environmentally friendly and provide solution to global warming in a no small measures.

The table 1 below shows the comparism of the energy consumption and green house gas emission of traditional and green concrete as highlighted by Abbas, et. al. 2007.

Table 1. *Comprise of the energy consumption*

	Traditional concrete	Green Concrete
GHG Emission in tones	1.25tonnes from Cement/ tone	0.875 sq. from Cement
GHG Emission per km.	15,000per km from aggregate	7,500 cube from aggregate
Energy Consumption (MJ)	4,000MJ from Cement	2,800MJ from cement
Energy Consumption (MJ)	63MJ from aggregate	31.5MJ from aggregate

Concrete are measure in the following area of focus of the green building index;

(a) Energy efficiency – There are many categories of energy needs in the cause of concrete production such as in the production of Portland cement, a tonne of Portland cement requires about four giga- joules of energy according to research. The use of some newly innovated cementitiouse materials such as fly ash, ground slag, silicon fumes ,etc for partial or complete replacement of cement in the productions of concrete and some admixtures, surfactants , all these put together, drastically reduce the energy consumption in the production of concrete and the gas emission through cement hydration process.

(b) Another area of energy requirement in concrete is in the production of crushed aggregates. The Portland cement association (1993) came up with estimated energy requirements for sand and gravel productions to be 5.0 M- joules per tonne while crushed aggregates requires an energy in – put of 54M- joules per tonne. The use of natural sand, washed and unsaturated sea sand and natural gravel will definitely reduces the energy production in the cause of doing that.

(c) The haulage of aggregate materials to processing plants requires enormous energy, so also is the transporting process of aggregates to ready mixed concrete plants. (PCA,1993) revealed that 2.7M-joules per tonne- km for sand and gravel and 3.8M- joules per tonne- km for crushed aggregates are required to transport these materials to ready- mixed concrete plants.

In addition to all these energy is the main energy needed in the production of wet concrete itself as established by (George and Michael, 2001), the production of new concrete using recycling concrete aggregates requires the same amount of energy as producing concrete with natural aggregate and this energy can be approximately estimated as 1.3 to 2.4 GJ/m3. Therefore, for concrete to be green and fulfilled the requirement of building index rating, it must be able to reduce drastically, the energy consumption and dissipation in the course of sourcing and production to the placement process.

Using, exergy analysis of energy system as postulated by (chunhai et al. 2007), though not for concrete but for heat and cold system in building, this paper is of the opinion that the energy efficiency of concrete can be estimated, the energy transfer includes work and heat, when work is converted into heat, the quality of energy depreciates. As a result of this, the ratio of work, which can be delivered to its surroundings to total energy can be used as a measurement of the energy quality, which is defined as the Energy Quality Coefficient (EQC) in GOBAS, and the formulae is;

$\gamma = W/ Q$.

Where, Q is the total quantity of energy in (GJ) and W is the exergy of energy (GJ).

The heat generated in concrete during hydration process can be used to arrive at the work being converted to heat.

4. WATER RESOURCES CONSERVATION OF GREEN CONCRETE

This is another green building index parameter that concrete that want to be green must fulfill. (Winfield and Taylor 2005), declared that natural aggregates production impacts on the surface and groundwater quality and the natural habitat of many species is a serious concern. Many are the ways that the excavation of natural aggregates changes land and vegetation slope and tamper with the underground water reservoir as natural aggregate is considered a natural reservoir for underground water. For any concrete to be green then it must make use of lightweight aggregates or recycle aggregate so that the water resources can be conserved.

5. CONCERETE RECYCLING

Another parameter of green building index is the ability to be recycled. According to (George and Michael, 2001), concrete constitute 52% of construction and demolition waste, and approximately 73% of it is reused in low value applications as filler materials or as road sub grade. The natural aggregates can be substituted with recycle aggregates since it has being confirmed that continuous use of natural aggregates and cement to produce conventional concrete has negative effects on the environment. Research has suggested some many ways of overcoming the problems associated with the use of recycled concrete, such as large amount of fines found in recycle concrete, lower density than the original materials used because of cement mortar attached to the aggregates surface and various contaminants that can be found in recycled concrete. Building demolished materials are generally termed as construction and demolition waste and the main part of this are considered to be concrete, in the word of (Mehta,2001), the annual worldwide output of concrete and masonry rubble has being estimated roughly as one billion tones.

6. EFFECTS ON GLOBAL WARMING

The green building index rating tools if employed in the assessment of concrete produced in the building, civil construction industries throughout the world, then the concrete produced will be environment friendly and contribute in the reduction of global warming which is a global problem. The following are the ways green concrete can affect global warming;

a) The energy production will be reduced likewise the green house gas emission.

b) The use of cementitiouse materials as replacement partially or fully, in the production of concrete will go a long way to reduce gas emission into the atmosphere.

c) Natural aggregates and sand can be replaced by the use of recycle concrete aggregate and lightweight aggregate as researched in the current years, hence energy are reserved and little disturbances to the natural water reservoir.

7. CONCLUSIONS

The invention of green building index with common aims throughout the whole world is a significant indication that the world is seriously facing and tackling the global warming problem as associated with the various activities of humankind. Shelter, which is considered as one of the three main basic needs of life, should be taking as major point of impact to the environment at large. Therefore, concrete that is green in nature will go a long way to help reduce global warming as the greenish nature can be assessed with green building index parameter as postulate by this paper. Another major area that needs to be studied throughout the world is the parameter to show the effectiveness of the various green building index in the various countries of the world, the activities of the facilitators of the various index and the government policies to back up their work needs to be investigated.

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THE POTENTIALS OF URBAN WIND POWER IN MALAYSIA

Hirda Lailani Khalid[1] and Abdul Malek Abdul Rahman[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]hirda_khalid@yahoo.com[1] , [2]malik@usm.my[2]

ABSTRACT: Located near the Equator, the Malaysian climate can be classified as hot-humid Equatorial. The sun generally shines throughout the year with high humidity and copious rain. Winds however are light and variable with the average of 0.3 m/s which can be considered as calm. Wind flows in the urban areas are complex but some general patterns can be identified such as the variation of height and different types of terrain. The uses of electric fans without proper design strategies to encourage air movements in the buildings will only distribute hot air that trapped inside. Due to this problem, most Malaysians are resorted and preferably to use air-conditioning systems to achieve thermal comfort at the same time avoiding air pollutions. Unfortunately, the modern fully air-conditioned buildings making the cities temperature grows warmer and becoming urban heat islands. The evidence shows that the average global temperatures rose by modest but significant amount since the 20th Century. The exhaust air from air-conditioning condenser which is technically released heat to surroundings, potentially becoming the alternative source in wind power generation. With the average of 9.23 m/s it is more than sufficient to generate electricity using a vertical-axis wind turbine (VAWT). Renewable energy is a promising way to solve the problem of environmental pollution and global warming. There has been increased interest in using renewable energy technologies especially in the urban areas. Solar energy is an abundant source in Malaysia and people are making use of it by using solar photovoltaic (PV). It is unfortunate that the wind characteristics here are unpredictable, multidirectional and erratic except for two places which are on the sea sides (land and sea breeze) and hilly areas (prevailing wind). Thus, this paper explores to what extent can wind or air movement be contributively and be better utilized in whatever form possible as an alternative energy for Malaysia's initiatives to meet the Kyoto Protocol requirements.

Keywords: wind power, Malaysia, urban area, exhaust air

1. INTRODUCTION

Malaysia is located near the Equator (4° 0' 0" N / 102° 0' 0" E) where the weather here can be classified as hot-humid Equatorial (www.met.gov.my). It is well-known for its extreme weather which is hot and humid throughout the year with copious of rain. Light winds, year-round warm and pleasant weather with constant temperatures and high humidity make outdoor living is much desirable (Furturarc, 2010). In the old days before the electrical lighting and cooling system were introduced, people rely solely on natural ventilation for comfort purposes by having many openings in their houses such as doors and windows in the living rooms, a chimney in the kitchen, and slits near the roof to allow air circulation and also illumination (Salmah Ahmed, et al., 2006). Traditional houses especially in rural areas were built in such a way the orientations of the buildings meet the criteria that favor the natural ventilation to take the effect.

It has been gradually changed when space is now becoming limited and expensive especially in urban areas. Houses were built to accommodate as much as possible in provided land, making people depending on electrical appliances for cooling and lighting purposes. As a result, the demand on energy rises as the number of population grows, thus consume a hefty bill for electricity every month. Most people are resorted to use air-conditioning systems to cool the buildings, thus becoming urban heat island. The modern fully air-conditioned building is well-known as the late 20th Century phenomenon (Roaf, et al., 2005) which is over 75% of the energy consumption in high rise buildings is allocated for air-conditioning systems only. Malaysia has been experiencing a dramatic increase in number of air-conditioning used which expected to be higher in the future (Mahlia, T.M.I., 2001;

Masjuki, et al., 2001). If looking into the air-conditioning process, the exhaust air path is the route ventilation of air travels between the outside and the occupied space inside the building. Exhaust air will be releases through the condenser unit that located outside the building. The hypothesis is the exhaust air releases from the air-conditioning unit as waste energy can be useful to generate electricity using the vertical-axis wind turbine (VAWT). This is the on-going research project that will be explained further on Section 3.

According to Kyoto Protocol, the members should initiate programs to reduce global warming. The Kyoto target is a relatively small step which can be considered as the first step on a 'ladder of change' (Stankovic, S., 2009). Malaysia National Policy has been created and one of the strategies is by promoting the use of alternative energy instead of fossil fuel. Suria 1000 targeting the residential and commercial sector will establish the new BIPV market and will provide direct opportunities to the public and industry to be involved in renewable energy initiatives and environmental protection (www.mbipv.net.my/suria.htm, 2010). To achieve the Kyoto Protocol and our National Policy, we need to discover whatever possibilities we have in our region to optimize the use of renewable energy such as wind, biomass, hydro energy, wave energy, etc.

The potential for urban wind energy in windy area is large and there are many forms that can be exploited (Stankovic, S., et al., 2009). In tropical region like Malaysia where it is difficult to obtain predictable or large amounts of wind energy, smaller systems may still be used to run low power equipment such as lighting, parking meters, etc. The key elements that determine the viability of urban power are wind turbine technology, wind resources, cost, and regulation (Christopher, et al., 2004). Besides, in considering the urban wind regime, two main characteristics appear which are lower annual mean wind speeds and turbulent flow (Paulides, J.J.H., et al., 2009). Omni-directional wind or uniform wind (turbulence) is common in many urban areas which help to accelerate the wind through building designs (Stankovic, S., et al., 2009). Those factors are crucial in order to choose the right and efficient turbine that is worth as alternative energy. This is important to get sufficient power output from wind generation.

2. WIND CHARACTERISTICS IN URBAN AREA

Malaysian winds are unpredictable, multidirectional, and erratic (Abdul Malek, 2004) except for sea sides that creating land and sea breeze and highlands, creating prevailing winds; which are set for rural settings. The average of natural mean velocities in several towns in Malaysia is in the range of 1.0 m/s to 3.0 m/s (Kubota, T., & Supian Ahmad, 2006) which can considered as calm.

In the cities (urban areas), wind flows are complex but some general patterns can be identified, starting with the variation of velocity with height as the wind flows over different types of terrain (Dutton, 2005). Most of the buildings are high-rise, thus affecting the pattern of wind flow. The air on the lower levels is normally polluted by exhaust air from transportations and other human activities. To avoid this, most tropical high-rise buildings especially in developing countries replicate cold-climate models with sealed facades, reliant on air-conditioning (Futurearc, 2010). As the higher the altitude, the intensity of air pollution is decreasing and the wind flows are increasing as well (Figure 1). These winds will not only have a relatively high energy content but are also likely to be less turbulent (Stankovic, S., et al., 2009). These advantages are useful to design high-rise buildings with natural ventilated internal spaces and making the air-conditioning as an option.

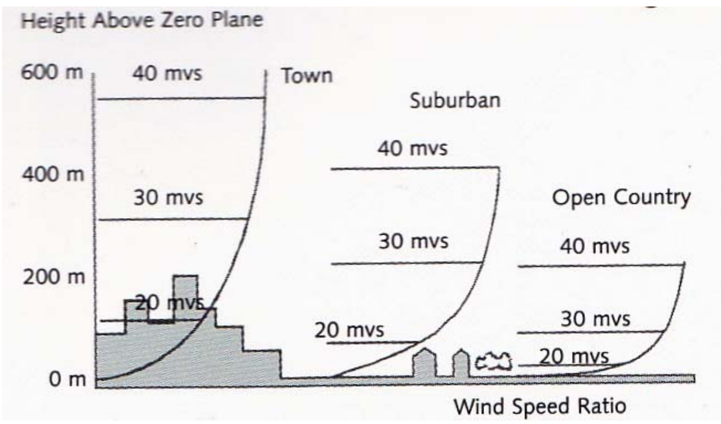


Figure 1. *Power law relation for varying wind speed with height (Source: Ecology of The Sky)*

The bioclimatic skyscrapers designed by Ken Yeang are taking these factors into the design considerations which can be seen through the building façades in most of his projects such as Editt Tower (Figure 2a), Menara Mesiniaga (Figure 2b) and Menara UMNO (Figure 2c) (Richards, I., 2001). Floor plans on the higher levels are different from those on the lower levels. Balcony, sky court and terrace garden (Figure 2b and 2c) are applied on upper levels not only functioned as resting areas but also to accommodate natural winds and provide greenery spaces to filter the natural wind. The other feature of bioclimatic skyscraper that considering wind factor is the wing-walls (Figure 2c) that functioned to redirect the wind to special balconies zone that serve as pockets with 'air-locks'. The positioning of wing-walls is indicated by the wind rose for the site.

The other example of tropical skyscraper that taking the advantage of natural wind on higher altitude is 'The MET', which is located in Bangkok. The climate in Bangkok is almost similar with Malaysia. The designs are accommodate cross-ventilated tropical houses in the sky with breezeways, full exposure to light and views, outdoor living areas, planters and high-rise gardens, open-air communal terraces, libraries and other facilities (Futurarch, 2010). These are the proofs that the tropical high-rise buildings have great potentials to achieve thermal comfort through passive design if we know the characteristics of nature itself.

(a) (b) (c)

Figure 2: *The highlighted areas show the design features that accommodate natural wind in Ken Yeang's bioclimatic skyscrapers designs (Source: Ecology of The Sky)*

The swirling turbulent character of low-level urban wind, due to complex interaction of the wind with buildings is detrimental to the amount of energy that can be extracted. The wake effect is a common phenomenon of winds in urban environments. It usually occurs when the wind is accelerating with high turbulence when hitting the edge of buildings. The disturbed regions of wake effect should be justified and should be avoided by placing the turbine as higher as possible (Figure 3). This is an important consideration when siting turbines in urban environment which commonly achieved by ensuring the blades of turbine are sufficiently elevated above the level that needs to be avoid. In the Western countries, the horizontal-axis wind turbine (HAWT) type is widely and commonly used especially in wind farms in rural areas because the direction prevailing winds are predictable. In urban environments, HAWT type is not suitable to be used due to the unpredictable pattern. However, there are cases where turbulence or the acceleration near buildings can be used to gain an advantage (Stankovic, S., et al., 2009). As the wind is at its highest speed at the edges (highlighted area), the optimum length need to be determined in order to reduce the unwanted turbulence. There are promising ways to generate electricity using natural wind in our low-wind speed conditions as the studies from Western countries on the characteristics of urban winds are studied thoroughly.

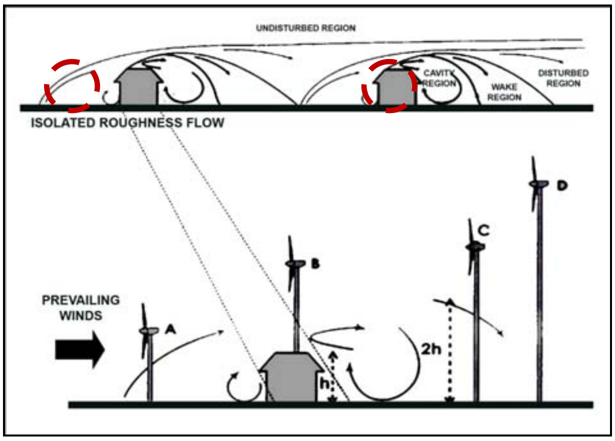


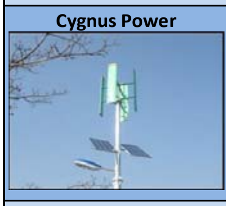
Figure 3. *The wake effect phenomenon and the possible sitings of wind turbines to avoid the disturbed region (Source: Urban Wind Energy)*

3. TURBINE TECHNOLOGIES FOR TROPICAL URBAN WIND POWER

There are two types of wind turbines that categorized by the way they look; which are horizontal and vertical. Horizontal-axis wind turbines is effective when the wind speed is higher enough and not suitable for the locations that have average wind speed is not enough and too low (Masaaki, H., et al., 2008). The vertical-axis wind turbine (VAWT) is the most suitable wind turbine to be used in Malaysia or urban areas where the winds are multidirectional and low-wind speed condition (Stitt, 1999; Masaaki, H., et al., 2008; Paulides, et al., 2009; Stankovic, S., et al., 2009; LeGault, M.R., 2009). VAWT is Omni-directional, which means it can harness wind from all direction. Changes in wind direction have fewer negative effects on this type of turbine because it does need to be positioned into the wind direction (Paulides, J.J.H., et al., 2009). With the minimum wind velocity of 5.0 m/s it is sufficient to generate electricity (Gauzin-Muller, 2002).

In Malaysia, there are several manufacturers that show potentials for urban wind purposes such as iWind, Cygnus Power and Loopwing. They have been designed to suit in low-wind speed conditions with the start-up wind speed as low as 0.8 m/s and it could start generate electricity (cut-in wind speed) at 1.8 m/s (www.iwindenergy.com.my). The technology from these manufacturers already has been tested in our region for stand-alone purposes and even in research field.

Table 1. *Several manufacturers of wind turbines in Malaysia that suited with low-wind speed condition*

MANUFACTURER	COUNTRY	Type (load)	Blade diameter	Start-up wind speed	Cut-in wind speed
iWind 	Taiwan	300 W	1.2 m	0.8 m/s	1.8 m/s
Cygnus Power 	Japan	200 W	1.0 m	1.0 m/s	1.3 m/s
Loopwing 	Japan	Tronc Series µ Series	1.5 m 2.85 m	2.0 m/s 2.0 - 3.0 m/s	4.0 m/s 4.0 m/s

3.1 *Small Wind Energy: Retrofitting and Building-mounted Wind Turbines*

Small-scale renewable energy technologies generate clean and renewable energy while reducing CO_2 emissions (Paulides, J.J.H., et al., 2009). The small-scale stand-alone power generation systems are an important source of electrical energy as they can be applied in locations where conventional generation is not practical (Ahmed Nabil, et al., 2009). As been discussed on the above section, the disturbed areas caused by the wake effect can be avoided by placing the turbines as higher as possible. Wind turbines installed at high levels are able to access winds with higher energy contents (Stankovic, S., et al., 2009). In housing areas, it can be done by determine the minimum height of disturbed area, thus small and stand-alone turbines can be building-mounted using high poles on the building's main structure.

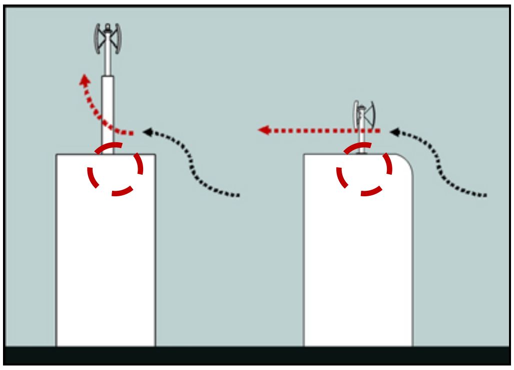


Figure 4.*(Left) The sharp edge of a typical building is normally creating turbulence and a disturbed region, thus wind turbine needs to be placed as higher as possible. (Right) With the smooth edge, winds will be less turbulence and wind turbine has the advantage to be placed much lower.*

3.2 Hybrid Solar Photovoltaic and Wind Turbine

Renewable energy source from wind turbine and solar photovoltaic (PV) is the most environmental-friendly type of energy to be used in tropical regions. Economic aspects of these renewable energy technologies are sufficiently promising to include them for rising power generation capability in developing countries (Nema, P., et al., 2009). In our region, solar is an abundant energy, making the hybrid solar photovoltaic and wind turbine as the most appropriate technology. Winds are less reliable in our climate condition but it is possible to be generated if it is being placing on the right locations such as sea sides, highlands and higher altitude.

Figure 4: *The examples of hybrid solar photovoltaic and wind turbine*

In cold-climate countries, wind speeds are usually low in the summer when the sun shines brightest and longest. The wind is strong in the winter when there is less sunlight available. Because the peak operating times for wind and PV occur at different times of the day and year, hybrid systems are more likely to produce power when it needed. For the times when neither the wind generator nor the PV modules are producing electricity (at night when the wind is not blowing), most stand-alone systems provide power through batteries or an engine-generator powered by fossil fuels like diesel. If the batteries run low, the engine-generator can be run at full power until the batteries are charged. It is applicable for Malaysian wind due to its unreliability factors. Today's technology has made wind turbine is easy and quick to install, besides it has came in modular system. Hybrid solar photovoltaic and wind turbine is cost-effective and suitable for the sites that under varying climatic conditions. It can be installed without measuring the irradiance of the photovoltaic or wind speed on site (Ahmed Nabil, et al., 2009).

3.3 Building-Integrated Wind Turbine (BIWT)

BIWT is defined when the wind turbines associated with buildings designed and shaped specially with wind energy in mind (Stankovic, S., et al., 2009). The buildings also act as concentrator of the wind (Mertens, S., 2002) in which it can speed up the wind velocity at the position of wind turbine or create a high turbulence level. Thus, the wind turbines will be located at the high wind speed zones in the buildings.

The integration of dynamic rotating machines will influence decisions on building designs such as orientation, massing, structure, insulation, etc. The matching of supply and demand within local areas and the connection of loads to the system need consideration at urban site planning scale because of the importance of the mix of building types and activities (Pitts, 2004). Alternative building forms, orientations and design features may be necessary to provide for or enhance the energy supply potential of the alternative technologies.

Wind turbines for the built environment that exploit higher wind speeds around buildings have to be designed for different types of flow and low noise emission (Mertens, S., 2002). The turbines that integrate smoothly to the building both structurally and visually will minimize the maintenance and noise especially in highly dense populated areas (Paulides, J.J.H., et al., 2009). For large-scale BIWT, it needs to consider the public safety aspect, the visual effect, noise, shadow flicker and blade reflected light, electromagnetic interference (EMI), biodiversity and birds, and property values (Stankovic, S., et al., 2009), if not these will distracting the existing surroundings.

The concentrator effect will only be present for small wind turbine dimensions compared to the building dimension since the wind turbine has to be located in high- wind speed zones, which only exist close to the buildings (Mertens, S., 2002). When a void or open floor is created, this would cause strong acceleration of wind. For example, the sky court design which is used to serve as transitional areas between inside and outside areas, it also can functioned as concentrator to receive direct wind flow of external wind and redirect it into indoor environment. Besides, small VAWTs can be retrofit in this area at the same time to harness the accelerate winds. A hole in the middle of building façade where the turbine will be placed is also called a ‘coin’ model (Beller, C., 2008).

The Castle House is an example of the coin model that acts as concentrator. It is a 43-story buildings rising to 147 meter above ground level with three 9-meter diameter wind turbine at the top. Another large-scale urban wind power is Bahrain World Trade Center. It has three units of horizontal-axis wind turbines that located in between two towers. Each turbine has 225 kW power make the total of 625 kW of power. The HAWT is the best type to be used for this building because they have the advantage of facing the Gulf Sea which has high annual wind speeds. Air Apartments, designed by Australian’s Rothe Lowman Architect is another proposal of integrated wind turbine design on top of towers. Wind turbines are adapted for local conditions will offset a percentage of energy loads of the buildings (Futurarch, 2010).

(a) (b) (c)

Figure 5. *(a) The Castle House, London (http://inhabitat.com/castle-house-eco-skyscraper/) (b) Bahrain World Trade Centre (http://www.bahrainwtc.com/) (c) Air Apartment, Australia (http://www.archdaily.com/33510/air-apartments-rothe-lowman/) that integrate the building designs with large-scale wind turbine.*

3.4 Harnessing Exhaust Air of Air-Conditioning System

This is the on-going research project entitled "Optimizing Wind Power for Energy Efficient Building Design in Tropical Hot-Humid Climate of Malaysia". The hypothesis is the exhaust air of air-conditioning can be the alternative source for wind power generation in Malaysia and other low-wind speed regions. It is predictable winds that already available from air-conditioning systems for most of buildings in hot-humid countries, thus wind turbine technology is ready to be retrofit into the existing buildings.

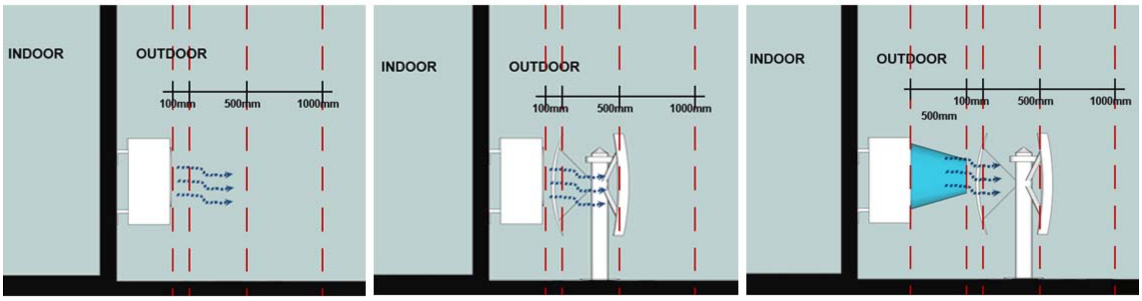
Based on the literature review discussed on the above section, it can be conclude that natural wind in Malaysia is unreliable, multidirectional and unpredictable making that as limitation factors of research in tropical countries. As for Western countries, wind power generations have large potential in replacing fossil fuels. However, they are also facing the same problems to harness wind power in urban areas due to the unpredictable flow pattern. The methodology of research that they are using can still be adapted for our conditions such as the juxtapositions of wind turbines on the existing buildings. The experimental study is divided into three phases, which are:

(a) Phase 1- Lab test: To test the efficiency of VAWT from manufacturer

(b) Phase 2- Exhaust air distribution: To investigate the characteristics of exhaust air distribution of condenser

(c) Phase 3- Field test: To test the efficiency of both VAWT and alternative source

After done the surveys on 30 condenser units around the USM campus, Topaire brand has the highest velocity with the average of 9.23 m/s. It is being used to test the characteristics of its air distribution without and with Venturi concentrator. The Venturi concentrator is expected to create the Venturi effect which will accelerate the exhaust air and give higher wind velocity. The final part of the thesis will be the recommendations on architecture design solution to retrofit the VAWT technology on the existing buildings in Malaysia especially in urban areas. To support all the empirical data, the result also will be shown in Computational Fluid Dynamics (CFD) Modeling.



(a) (b) (c)

Figure 6. *(a) The characteristics of exhaust air distribution will be investigated, (b) without concentrator and (c) with a venturi concentrator*

4. CONCLUSIONS

The research on wind power in low-wind speed conditions have not been discovered widely as there are many limitations on wind characteristics itself. In Western countries, wind power is a potential renewable energy alongside with solar energy, hydro power, biomass, tidal energy and geothermal. Thus, their studies on urban wind power in which the conditions and characteristics are almost similar to our climate conditions, could be used in this research. There are many possible ways to optimize wind power in Malaysia; therefore any possibilities should be explored as it is our initiatives to meet the Kyoto Protocol requirements.

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FACILITIES MANAGEMENT DECISION SUPPORT IN INTELLIGENT BUILDING TECHNOLOGIES: A STUDY IN KLANG VALLEY, MALAYSIA

Mohamad Ridzuan Yahya

Kulliyyah of Architecture and Environmental Design, International Islamic University Malaysia, 50728 Kuala Lumpur.

moham643@gmail.com

ABSTRACT: It is important to focus on developing facilities management (FM) decision support aspect in the intelligent building technologies (IBTs) of high rise buildings in order to make sure it is in good condition and positive performance all the time. In Malaysia scenario, various obstacles have to be faced by the FM for controlling all activities related to the high rise building and its system. The crucial issue is to provide advance technology in order to support adequate economical and technical aspect in FM. FM decision support involves obtaining maximum benefit from the investment made on the maintenance activities. The present research focused on a pilot study which interviewed all professionals involved in the project particularly based on the value aspects of building which are the quality, safety and service of the FM provided and implemented in five selected high rise buildings in Klang Valley, Malaysia. The paper concludes that even though measured a current practice of FM, the model of IBTs in Malaysia has not been adopted as rapidly and extensively as expected because of lack of information and knowledge support to the entire group of professionals involved at the design stage until future maintenance of a building. This paper provides a concise general idea on the new advances of IBTs and discusses ways of supplementing the decision making process by adopting two methods for economical and technical aspects of IBTs. It goes on to initiate an on-going study which aims to develop knowledge based systems to grant all-inclusive evaluations FM decision support to retrofit building maintenance when they are being conceptualized.

Keywords: **intelligent building, facilities management, decision support**

1. INTRODUCTION

IBT of high rise buildings in Malaysia are becoming one of the criteria in the FM and performance management. As new products and technologies are emerging, this problem seems to be more evident as the gap widens between available IBT and the actual number of high rise buildings incorporating IBT concepts. Recently significant advances have been made in the design, engineering and construction of IBT, which not only accommodate major advances in high rise building but also provide better physical and environmental settings for the occupants. However, the development and implementation of sustainable FM in that particular building is still lacking. Lee and Scott (2008) identify building policy and strategy, strategic management, facility management and performance management as the four main aspects influencing the sustainable of IBT implementation. Nowadays building owners are more concerned about building maintenance performance as it reflects their buildings image as well as competitive advantages in terms of a marketing strategy to attract more people to buy and rent.

There are many potential reasons for this gap. One of them is the lack of information and understanding of IBT among building owners and FM personnel. FM, with characteristic least cost mindset, often considers IBT expensive to install as well as maintain. They lack true thoughtful of the IBT. Very often they are not well knowledgeable of the lifecycle costing of the project. Hence, these decision makers often fail to consider the effectiveness and flexibility that IBT can ensure to their tenants and occupants, which will raise rent potential. In addition, engineers develop new technology to mechanical and electrical system as well as electronic system such as building automation system (BAS) and other advance technology in their components in order to save energy and easier to control these particular systems. In contrast, contractors refuse to take on these concepts in their products fearing that it will make their job more difficult and increase project risk and costs. Again, lack of knowledge and appreciation of IBT plays an important part.

The IBT is considered to be fundamental part of ensuring high-quality service for the high rise buildings such as Petronas Twin Tower, TM Tower, Maybank Square, Maybank Tower and Kawalram Tower. According to Seeley (1976); Lee (1987); Spedding (1987); Barrett (1995); Chanter and Swallow (1996); Horner *et al* (1997) the future implementation in facilities management, including IBT and its maintenance programming and scheduling are developed.

2. THEORITICAL FRAMEWORK

This paper focused on high rise building managed by the facilities management (FM). Theoretically, in managing IBT of high rise building, both parties, i.e. FM and the owners have to attain a consensus in all management-related matters. This is to ensure an effective management for the building. This study found that there are many potential reasons and gap decision maker in FM regarding IBT among high rise buildings. In discussing this matter, the paper starts briefly on several key issues that led to a management gap. It is then followed by the methodology in identifying the management gap and the important findings from this study.

As a IBT can be seen as a system, the systems approach can be used in the FM development process. The systems approach integrates the analytic and the synthetic method, encompassing both holism and reductionism (Heylighen, 1998). This approach employs the systems thinking introduced by Simon (1960). When solving a complex problem following the systems thinking, one is to take out the parts which can be well defined and solve them separately. For IBT problems characterized by complexity (such as those concerned with FM and maintenance management re-development) using systems concepts offers a way of rationalizing aspects of existing practice and of suggesting directions for improvement (Stewart et.al., 2001).

While systems theories and models can certainly be improved, De Greene (1993) placed great emphasis on the deficiencies in the structure of the IBT implementation and in policymakers. He provided the results of many years of working with complex systems in the contexts of systems analysis, systems design, and policy-making and decision-making.

While aiming at the application in the housing sector, Burian (1989) who discussed a framework for IBT and development. The framework presented can be used in the IBT development process or post facto in FM. The usefulness of the framework is that it provides a means to think about IBT and its development.

According to the systems approach as understood from the literature, six (6) steps of IBT development process are defined in this paper, namely:

i. Preparation and informal consultations
ii. Establishment of facilities management agenda
iii. Creation of facilities management personnel commitment and owners as well as end user opinion
iv. Decision making
v. Evaluation
vi. IBT Implementation and FM development

3. RESEARCH OBJECTIVE

To rectify this problem, an aim of study is to develop a knowledge based system to provide information and decision support to FM, engineers, contractors, potential tenants and occupants, on the adoption and application of IBT. It involves several major stages of development, such as scope definition of IBT applications, categorization of IBT and extraction and formation of decision-making processes of design teams and FM. Unique mechanisms for simulation and knowledge representation developed for the system allow the simulation of decision making models while maintaining system flexibility. The incorporation of life cycle costing analysis into the system adds another dimension and creditability to overall process of the identification, selection, evaluation and feedback for the application of IBT.

Having identified some of the possible reasons for the resistance to IBT applications and based on these findings, this paper gives a brief review on the recent IBT development and its relevance in tenant and occupant requirement. It discusses the specific considerations in the decision making process of IBT applications in terms of economical and technical evaluations.

The present research focused on sustainable IBT of existing FM in high rise buildings in Klang Valley, Malaysia. Its goal was to stimulate and support sustainable and consumer oriented transformation of IBT in high rise buildings. The main results of this research are management tools designed for FM organization and decision makers (building owner) in order to integrate sustainability and building end user participation in IBT implementation processes. It was concluded that an integrated approach for decision making based on a strategic plan could contribute to better outcomes. Based upon questionnaires interviewed to the 100 respondents, a comprehensive analysis of data was compiled. The main objective was to identify the problems of sustainable IBT of FM in high-rise buildings. Not only were problems from a technical perspective discussed, but also FM performance indicators were taken into account.

4. INVESTIGATION METHOD

This study attempts to provide a technical analysis on the IBT of FM in high rise buildings through unstructured interview. The study surveys the availability of relevant information and documentation, and the performance of building maintenance in FM. The interview is based on the following questions:

i. How buildings were managed?

ii. Whether or not IBT is provided in practicing FM operational processes?

iii. Whether or not the FM personnel have appropriate skill and knowledge?

iv. Whether the record management is systematic?

v. Whether the maintenance job is conducted in house or outsourced?

vi. Whether the FM complies with building requirement?

vii. Whether the IBT operation is efficient?

viii. Whether the IBT fulfill a satisfaction level and comfort?

Five high rise buildings in Malaysia were randomly selected. The buildings were Petronas Twin Tower, TM Tower, Maybank Square, Maybank Tower and Kawalram Tower located in Klang Valley, Malaysia. Building Owner, FM personnel, engineers, maintenance professional, contractors, potential tenants and occupants were interviewed. A series of test runs will be combined with verification and evaluation of the system by engineer, building management personnel and contractors.

5. RESULT AND DISCUSSION

It was found IBT implementation was available in certain system only such as air-conditioning, lighting, alarm, close circuit television (cctv), building access, lift in the buildings but the development of strategic management, FM and performance management were available in them. However, planned maintenance and unplanned maintenance are not in a high priority list in most of the IBT in FM organizations. It was also found annual budget for unplanned maintenance and long-term planning for planned maintenance of IBT, were practiced at only three buildings. Finally, it was found all buildings did not conduct building services audit of IBT.

5.1 BUILDING MANAGEMENT AND FM OPERATION

The operation of building and FM is becoming much more business driven and managers will increasingly see occupants of buildings as customers or clients who must be kept happy rather than tenants who just happen to occupy the buildings. FM able to utilize both human systems for FM and computer systems for building such as building automation system (BAS) in order to provide a better service. In turn, this demands three levels of solutions to ensure a successful intelligent building in place. First, an effective building shell is needed to absorb information technology expansion and allow organizations' growth and change. Second, building and BAS and space management systems are required to facilitate effective internal and external communication, support environmental and individual comfort control and monitor change, access and usage. Third, there is a need for integrated technologies and services that allow the integration of disparate organizations, systems, data and personnel to focus on the common goal of increased business effectiveness.

5.2. SPECIFIC CONSIDERATIONS IN THE IMPLEMENTATION OF IBT

The application of new IBT was met with problems associated with the decision making process, in the conception stage of the project development. Key issues include the lack of information and established assessment criteria.

It is evident from the findings that the knowledge and quality of maintenance of FM implemented for IBT system is not consistent. There is a need to develop a FM decision support to be enforced practically to ensure comfort and safety to public. Maintenance of IBT seems to be implemented 'half cook' basis. In other word, planned and unplanned maintenance is not given a priority. This can be inferred from the finding of this study that annual budget for unplanned maintenance and long-term planning for planned maintenance are not implemented in some buildings. They more concentrated on corrective maintenance rather than preventive maintenance. This argument is supported by other findings that building audit, response time, down time, request for repairs and quality workmanship; the benchmarks for operational processes improvement were also not implemented. A sound planned maintenance relies on comprehensive data collection through building audit.

6. RECOMMENDATION

A systems approach for IBT can be used to integrate decision support and to structure the process involving many stakeholders. The implementation of this approach is not simple since it will take much effort to break down closed thinking patterns and working cultures as well as to introduce new roles in the collaboration. In order to do so, we firstly need to understand the real concept of IBT for sustainable FM. The model for IBT development process can be used to define the scope of its development. For example, at the building owner level, IBT development is aimed at elaborating strategies which reflect the building image for sustainable FM in high rise building. IBT development for FM of buildings is in direct relation with implementation for product development, for instance sustainable building maintenance quality in the high rise building. In setting-up strategies for building maintenance, one should take into account the current high rise building, such as the quality of design and engineering as well as the consequences of the applied techniques and materials of IBT for the future implementation of the building.

It is recommended to follow the above discussions for further in-depth investigation by looking into the following objectives:

- define and identify the basic terms in the IBT operation processes;
- develop the system of IBT;
- understand the objective and justification of planned maintenance and unplanned maintenance for IBT;
- identify the list of the priority for planned maintenance and unplanned maintenance operation in the IBT system;
- identify the categories and problems of maintenance strategy challenges;
- identify the categories of impacts on building maintenance operation processes due to FM and strategic management;
- improvement of maintenance operation processes through building performance management and building audit;
- rationalize and improve the gaps between top management at strategic level and maintenance personnel at operational level.

To provide evaluation and feedback on the performance of IBT with respect to conventional building projects, two methods of evaluation can be used together with an analysis of design and construction options.

6.1 ECONOMICAL ASPECTS

The Life Cycle Costing (LCC) analysis is a method used to assess design alternatives considering all the significant costs of ownership. The ultimate objective of LCC is the determination of optimum design decisions following the evaluation of all viable design alternatives. The LCC process can be applied to a total building or to any building components. It generally includes the following steps:

- Choose design alternatives for cost studies.
- Determine elements for each option to be studied.
- Establish capital cost and cost-in-use for each alternative.
- Convert cash flow for each option to a common time basis for rational comparison using discounting techniques.
- Select final design solution with consideration of function, quality etc.

Single building systems apply those LCC components related to capital cost, maintenance cost, operating cost and demolition cost. In addition to cost components, another critical aspect of LCC is the determination of evaluation period over which the building or component is expected to operate. For the purpose of evaluation, the following operation periods are suggested:

- The shell, skin, and structure are designed to last for at least 50 years.
- The services, the primary mechanical and electrical systems have a lifespan of usually no more than 15 years.
- Ceilings, partitions, furniture and finishes, which constitute the fitting-out to accommodate a particular division or tenant, are often cleared away within five to seven years.

6.2. TECHNICAL ASPECTS

Technically, it was decided that the proposed system development must incorporate three key features:

- Ability to conduct consultation processes of IBT decision making during project conception and design.
- Ability to perform basic life cycle cost information and to provide comparisons.
- Ability to assess the level of building intelligence based on user options and indicates areas of concerns and possible improvement.
- Ability to present alternatives to users, provide evaluations and make recommendations of decisions.

With the novice nature and vast body of knowledge to be accessed in these types of projects, it has been decided that a prototype system should be developed in a controlled domain of IBT applications. At the same time, this area of application must also be practical so that the research results can be readily verified and used by the industry.

Therefore, to cope with environmental, technological and organizational changes, building management have devoted much effort to those building services that have shorter lifespan. New innovations and development in energy saving and automatic lighting and air conditioning controls, cctv, alarm are to name just a few. However, the major asset of an IBT lies within its ability to learn and adapt. Hence, the development of new technologies in association with building management and operation is given high priority.

7. CONCLUSIONS

IBTs are set to play a major part in the future high rise building development. This has been demonstrated by the changing business requirement and the subsequent adoption of intelligent buildings by major corporate clients as building users/tenants. However, the application of IBT is lagging behind their invention and development. There has been inadequate support to the dissemination of information and particularly the decision support at the conceptual design stage of building development. The objective of the paper is to raise awareness of the importance of maintaining a comprehensive understanding of IBT implementations. The on-going system development introduced here attempts to fill in the gap between increasing demand and the lack of tools in tackling the complex issues of selecting the most appropriate design alternatives, when clients and consultants work with other construction professionals for IBT applications.

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T151

A STUDY OF THE HEALTH AND SAFETY CIRCUMSTANCES OF SHOP APARTMENT AND APARTMENT BUILDINGS IN KLANG VALLEY, MALAYSIA

Mohamad Ridzuan Yahya

Kulliyyah of Architecture and Environmental Design, International Islamic University Malaysia, 50728 Kuala Lumpur.

moham643@gmail.com

ABSTRACT: Residential stratified property is now becoming a high-density built environment poses challenges of living style among the urban space in Malaysia which is currently facing serious problems with the health and safety of particular building stock. To examine the significance of the high-density problem, this study aims to survey the health and safety performance of shop-apartment and apartment buildings in a compactly inhabited urban, using a basic assessment method. An assessment method based on a hierarchy of building performance indicators concerning the quality of five attributes: (i) architectural design, (ii) building services design, (iii) the surrounding environment, (iv) operation and maintenance, and (v) facilities management approaches was developed. Nine shop apartment buildings and eight apartment buildings were randomly selected in Klang Valley, Malaysia and assessed through site inspections, desk searches, and interviewed facilities management personnel, management corporation personnel, home-owners as well as tenants. A performance an analysis was conducted to examine and compare the overall health and safety performance of the buildings. The results had shown that there were significant variations in health and safety conditions across buildings, although they are situated within a single locality. Most of the problems were attributed to differences in building management systems rather than building design. Enhancing strategic management approaches (e.g. a comprehensive planned maintenance) appears to be the most critical factor that underperformers should consider in order to improve the occupants' quality of life.

Keywords: health, safety, building performance indicator, strategic management

1. INTRODUCTION

Currently, shop-apartment and apartment stratified buildings are becoming accepted in Malaysia especially among metropolitan citizens. One of the reasons they prefer to dwell in the particular buildings is the facilities provided in the building and its surrounding. The FM is the responsible party to manage all the facilities in the building. Shop-apartment stratified building is a unique building and it differs from landed building, such as factories and terrace houses. Its uniqueness presents itself during the management era after the buildings have been occupied, where facilities management became an issue (Linariza and Ashok, 2003). To handle the issues pertaining to the management and maintenance activities, STA was implemented in 1985, to accommodate the insufficiency of the National Land Code that was ineffective in handling the issues that arose in stratified building types (Tiun, 2006). According to STA, the FM is accountable for all the management and maintenance aspects of the building and common facilities in the building (Teo, 1993; Jamila, 1994). Unfortunately most of the stratified buildings found were not effectively managed. Owners complained through the mass media and the issues were always about health and safety as well as disputes between FM and home-owners. Referring to Liias (1998), Jamila (1994), Malaysia Government (1999), Sapian (2003), Tiun (2006) and Eddy (2004a) on the management side, the most challenging issues faced by them in managing the stratified buildings were health and safety aspect which related to FM activities. These issues led to the management gap in managing facilities of shop-apartment stratified building. The management gap occurred when the expected services by the owners cannot be delivered by the FM. The owners were paying on a monthly basis and expected the facility management to be effective. Under STA, there is a legal provision for the FM to engage the services of a Management Agent (MA) as the party responsible for running the facility management activities. In practice, most FM functions in that way because they do not have the expertise to run and maintain the high rise building. If the FM fails to function well as stipulated in STA, the owners of the particular building have the right to summon the said FM. Therefore in order to avoid this case, the FM normally engages a Management Agent in order to transfer their liability towards the building. Even so, the management gap continued to exist as referred to the on-going issues reported in mass media (Tiun, 2006). Owners continued to complain about the health and safety in the stratified residential buildings as well as the responsibility of FM in ensuring effective FM.

2. RESEARCH OBJECTIVE

Since health and safety are among the most problematic issues in a high-density environment, this paper aims to survey the health and safety performance of shop apartment and apartment buildings in Malaysia using a simplified assessment scheme. Health and safety were chosen as research focus because they are the most fundamental aspects that a dwelling should fulfill, and yet they usually cannot be easily observed and evaluated by occupants and the public. Moreover, health and safety problems have a stronger spillover effect in a high-density setting than a low-density one a building with poor health and safety conditions not only adversely affects its own occupants, but also jeopardizes those living and working in the neighborhood. The research, therefore, contributes to the revelation of hidden building information to the community, which, in turn, helps build a more sustainable residential area.

3. RESEARCH METHOD

Step 1	The background and objectives of the survey are first presented to the respondents.
Step 2	The respondents are given brief instructions on how to complete the questionnaire, followed by an explanation of the key terms used.
Step 3	The respondents fill in the questionnaire.
Step 4	The questionnaire results are keyed into computer.
Step 5	The ranking, weighting, and internal consistency ratios are calculated by using the *Expert Choice 2000*.
Step 6	The analyzed results (i.e., the weighting and ranking of each attribute) are reported to respondents.
Step 7	If the internal consistency ratio at any level is not smaller than 0.1, the computer package will locate the possible sources of inconsistency. The respondent is allowed to revise his/her responses.
Step 8	The revision is keyed into a laptop computer. Instantaneous feedback on the internal consistency ratio is shown to each respondent.
Step 9	If the internal consistency ratio is still not lower than 0.1, the respondent can follow Steps 7 and 8 again for further revision. The process will continue until the respondent rejects making any further changes.

Figure 1. *A flowchart showing the procedures of the BHHI and BSCI*

Utilizing Ho et al. (2004) simple assessment framework, nine shop apartment stratified buildings and eight apartment stratified buildings have been surveyed within a short period of time in Klang Valley area (refer to Figure 1). Research also extended Ho et al. (2004) health assessment framework to building safety. To make the survey results more comprehensible to the public, the research has translated technical performance details into indices (e.g. a health index and a safety index) for building classification. With these indices, the public and building owners can easily know the health and safety performance of buildings.

The research method was organized which include site inspections, desk searches and interviewed facilities management personnel, management corporation personnel, home-owners as well as tenants.

Two assessment schemes such as Building Health and Hygiene Index (BHHI) for health and Building Safety and Conditions Index (BSCI) for safety were developed which based on the theoretical assessment framework introduced by Ho et al. (2004). There are 25 building factors for the BHHI and 19 building factors for the BSCI.

The weightings of the building factors were assessed by two expert panels by using analytic hierarchy process (AHP) (Saaty, 1982).

1. 15 experts were selected in building health who gave their perceptions of the relative importance of the building factors in the BHHI scheme.
2. 13 experts were selected in building safety who provided the weightings of the building factors in the BSCI scheme.

The respondents' weightings of the different factors were extracted from a pair wise comparison of the relative importance of all pairs of factors at the same level as the hierarchy using the AHP computer package Expert Choice 2000.

To compute the rating of each building factor in the assessment scheme, one would normally use a continuous scale ranging from the best practice (rating = 1) to the worst practice (rating = 0).

3.1. ASSESSMENT PRINCIPLES

The purpose of assessment is to quickly scan the health and safety conditions of shop apartment stratified buildings and apartment stratified buildings at the city level for building classification. The principles of the survey design based on the assessment model proposed by Ho et al. (2004), which is simple, and yet theoretically sound. This assessment model is in line with the ideology of the assessment model developed by Kim et al. (2005), which was designed for existing buildings with various degrees of quality.The principles of survey design are as follows.

1. The items to be surveyed should be flexible enough to embrace most settings of shop apartment and apartment buildings (e.g. buildings of different designs).
2. The items to be surveyed should be easily obtainable. Site inspections were confined to common areas and the external environment. These common areas are usually the most problematic in terms of management and maintenance due to the co-ownership nature of these areas (Chen and Webster, 2005).
3. Only measurable and verifiable items would be assessed, while subjective items should be avoided as much as possible. If subjective judgment (e.g. hygienic condition and visual structural condition) was unavoidable, assessor would be given clear guidelines, with judgments referenced to sample photos of different grades or scores to ensure consistency.

3.2. DEFINING HEALTH AND SAFETY

Ho et al. (2004) pointed out some characteristics that a healthy building should have:

1. A healthy building should not be too densely populated;
2. Its window design and layout should facilitate natural ventilation and diffusion of daylight;

3. It should be isolated from noise and air pollution sources;
4. Its water supply and waste systems should be appropriately installed, maintained, and managed; and
5. Its environmental conditions should be clean and hygienic.

Ho et al. (2004) was defined a safe building as one that minimizes the risk of physical injury and the death of occupants, such as evacuating them effectively should emergencies arise. Hence, a safe building should have the following characteristics:

1. a structurally sound construction design and condition;
2. properly installed and maintained electrical and gas supply systems;
3. a design that facilitates the evacuation of occupants in case of emergency; and
4. a location that is less prone to flooding or landslides.

When building factors are qualitative in nature, a different method is used to deal with intermediate cases. A dichotomous building factor (e.g. presence or absence of a certain design), by definition, has no intermediate case, and its rating is either zero or one. For multinomial building factor, we will describe clearly what each rating means, as illustrated in Table 1.

Table 1. *Rating scale for drainage conditions*

Grade	Rating	Description
Satisfactory	1	Good conditions without observable defects
Above average	0.75	Surface of the pipe or brackets slightly rusted
Acceptable	0.5	Pipe partly rusted
Deficient	0.25	About half of the length of pipe rusted; vegetation growth around the pipe
Poor	0	Choking and unsanitary conditions; whole length of the pipe seriously rusted; dripping observed, or pipe broken

3.3 DESK SEARCH

Most of the information required for building design was acquired through desk searches which include:

(a) taking measurements from building layout plans;

- to get information on a building design, such as its window-to-floor area ratio, the size of its residential units, and the widths of its staircases.

(b) searching for information on the web;

- to find information like the air pollution index and other information such as population density.

(c) analyzing street maps for items under external environment;

- to give information for external environment factors, such as the distance of a building from the nearest fire station and green area.

3.4 SITE INSPECTION

Site inspection is important because the information from other sources does not often reflect the real situation of a building. Hoxley (2002) stressed that anything missed during the initial inspection of the exterior could be re-inspected relatively easily, but it may not be so easy to re-inspect the interior when any problem is raised during the subsequent external inspection. All parameters to be measured or inspected on site were confined to:

(a) common areas (e.g. podiums, lobbies, lifts, staircases, and corridors) where permission for access is given by owners or management agents,

(b) the surrounding external environment.

3.5 STRUCTURED INTERVIEW

With the use of a preset questionnaire, interviews with facilities management personnel, management corporation personnel, home owners as well as tenants were conducted to collect this information. If necessary, the owner or management personnel was requested to provide documentary records (e.g. tenant survey records and monthly financial statements) for verification.

4. RESULT AND DISCUSSION

Table 2. *Building factors assessed under the BHHI and BSCI, and their relative weightings*

	Level 1		**Level 2**		**Level 3**	
		Weight (%)	**Category**	**Weight (%)**	**Building factor**	**Weight (%)**
BHHI						
	Design	**53.6**	**Architecture**	**18.5**	**Size**	**2.5**
					Plan shape	**3.5**
					Master Bedroom	**2.0**
					Windows	**5.7**
					Noise reduction	**3.4**
					Open space	**1.4**
			Building Services	**19.3**	**Water supply**	**5.6**
					Drainage	**6.8**
					Refuse disposal	**4.7**
					Lift	**2.2**
			External Environment	**15.8**	**Density**	**1.9**
					Adjacent use	**1.7**
					Air quality	**5.2**
					Aural quality	**2.6**
					Visual obstruction	**1.6**
					Thermal comfort	**2.8**
	Management	**46.4**	**Operations & maintenance**	**27.1**	**Cleaning**	**5.1**
					Pest control	**3.1**
					Refuse handling	**4.6**
					Drainage	**4.6**

					condition	
					Unauthorized alteration	**4.0**
					Water quality	**5.7**
			Management approaches	**19.3**	**Owners' duties**	**7.9**
					Documentation	**6.8**
					Emergency preparedness	**4.6**
BSCI						
	Design	**47.0**	**Architecture**	**22.1**	**Height and disposition**	**3.8**
					Means of escape	**9.3**
					Means of access	**6.3**
					Amenities	**2.7**
			Building services	**16.6**	**Fire service installations**	**8.3**
					Electrical installations	**4.3**
					Fuel supply	**4.0**
			External environment	**8.2**	**Proximity to special hazards**	**6.4**
					Proximity to fire station	**1.8**
	Management	**53.0**	**Operations & maintenance**	**33.5**	**Structural condition**	**8.6**
					Building services condition	**5.3**
					Exit routes condition	**8.4**
					Fire compartment	**4.3**
					Illegal appendages	**6.9**
			Management approaches	**19.5**	**Owners' duties**	**4.3**
					Documentation	**3.5**
					Emergency preparedness	**7.8**
					Financial arrangement	**3.9**

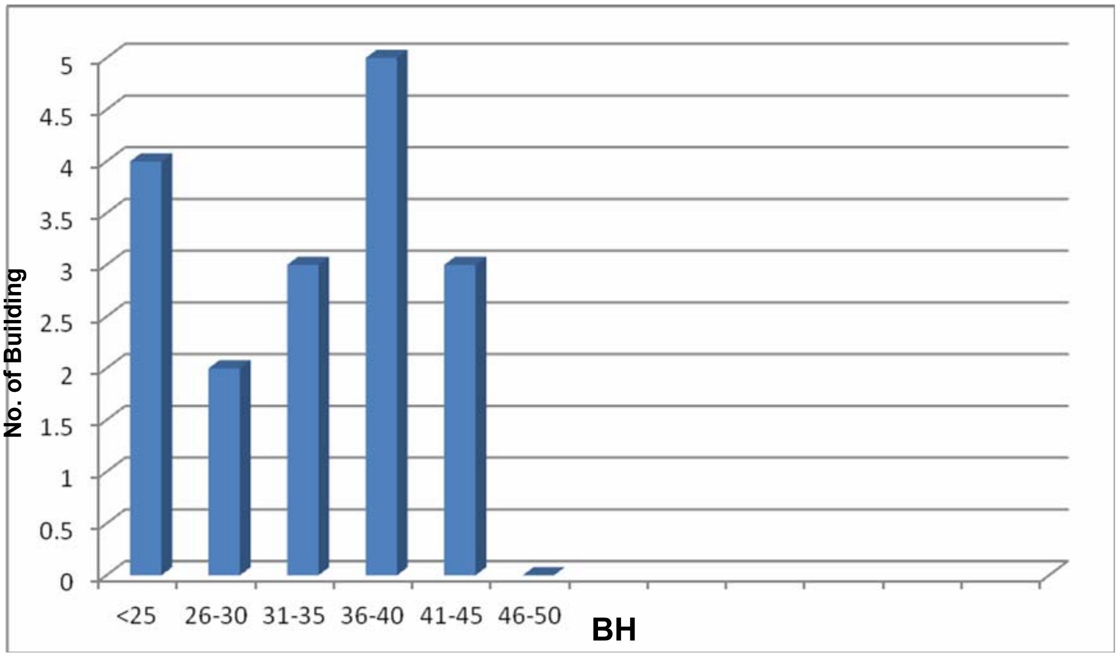

Figure 2. *Building Health and Hygiene Indices (BHHI) of the sampled buildings*

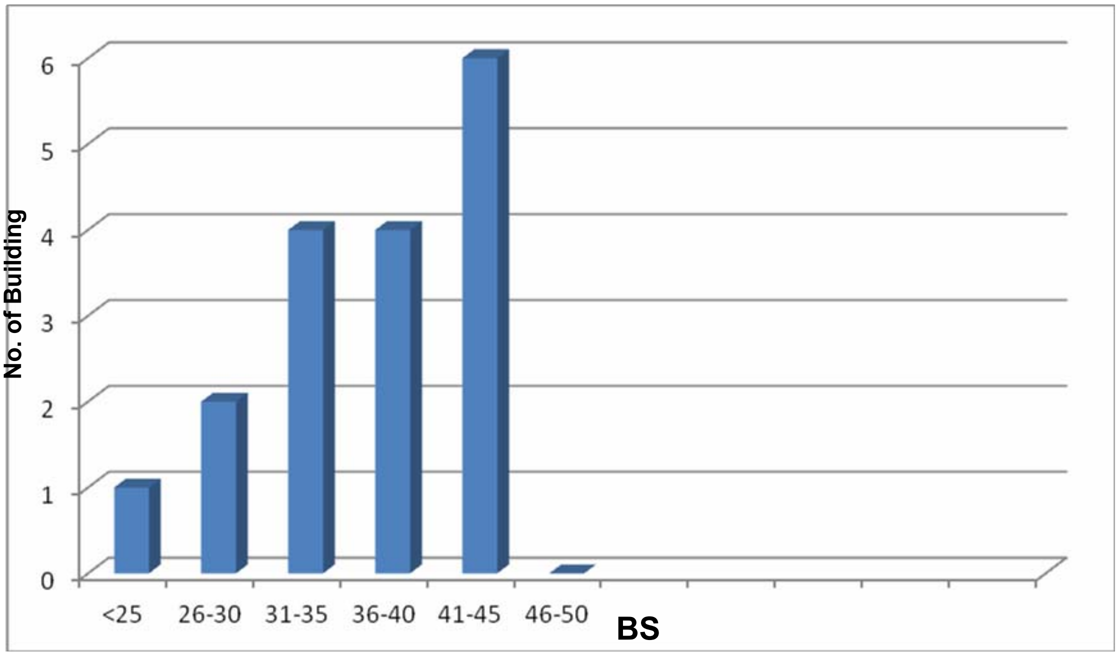

Figure 3. *Building Safety and Conditions Indices (BSCI) of the sampled buildings*

4.1 Overall results

The raw data collected was converted into a set of performance indicators that represent the health and safety conditions of each building factor in Table 2. The analysis of the assessment results will contribute to the key factors that influence the variations in the health and safety performance of the shop apartment and apartment buildings.

$$\mathbf{BHHI}_k = \sum_{i=1}^{n} W_{H,i}\, F_{H,ik}. \qquad \text{Equation 1(a)}$$

$$\mathbf{BSCI}_k = \sum_{i=1}^{ns} W_{S,i} F_{S,ik.}$$ Equation 1(b)

The distributions of BHHI and BSCI after the application of Equation (1a) and Equation (1b) to each building are presented in Figure 2 and Figure 3, respectively. The median BHHI and BSCI scores were 42% and 58%, respectively. Specifically, the BHHI ranged from 26% to 65%, whereas the BSCI ranged from 33% to 68%. Since the indices are building-specific, the health and safety performance of every building, it can be compared to the others. The stakeholders can use these results to know whether a building outperforms or underperforms. Homebuyers as well as FM also can use the data to ascertain the performance of building before they make their decisions.

4.2 Performance attribution

As shown in Figure. 2 and Figure 3, buildings differ, at most, by 35% for the BHHI and 43% for the BSCI. A variance disintegration analysis was conducted to tell the relative significance of the first level factors (design and management) in affecting the dispersion of the BHHI and BSCI.

By definition, the BHHI and BSCI are the weighted sums of the design index (DI) and the management index (MI), respectively. In other words:

$$\mathbf{BHHI}_k = w_{H,D}\,\mathbf{DI}_{H,k} + w_{H,M}\mathbf{MI}_{H,k.}$$ Equation 2(a)

$$\mathbf{BSCI}_k = w_{S,D}\,\mathbf{DI}_{S,k} + w_{S,M}\mathbf{MI}_{S,k.}$$ Equation 2(b)

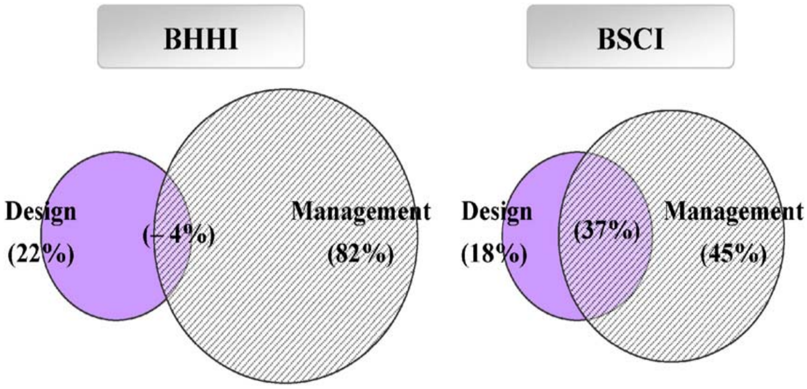

Figure 4. *Contributions of design and management factors to variations in the BHHI and BSCI*

Figure 4 summarized the results in Venn diagrams. 82% of variations in BHHI are merely attributable to management factors, suggesting that management factors dominate design factors in differentiate healthy buildings from the relatively less healthy ones. Only 4% of variations due to their co-movements were very low and insignificant. In BSCI, pure design factors contributed 18% to the total variation, while pure management factors contributed 45%. Similar to its health counterpart, pure management factors are more influential than pure design factors in affecting the variations in safety performance. In other words, most of the variations in building health and safety conditions were attributed to be difference in building management rather than building design. So, FM should improve health and safety performance by enhancing management. However, as opposed to the BHHI's results, the co-movement of design and management factors occupies quite a significant share, being responsible for 37% of the variations in the BSCI. A probable rationale for such a strong co-movement is that design and management factors are determined by some ordinary factors, especially building age.

5. CONCLUDING REMARKS

This paper developed two simple and cost-effective assessment schemes to evaluate the health and safety performance of stratified residential buildings in Malaysia. There were significant variations in health and safety circumstances across buildings, although they are placed in the same area. Neglecting and tolerating poor building health and safety conditions could now make a high-density society pay a higher price in the future. Most of the variations in building health and safety conditions were attributed to differences in building management systems rather than building designs.

Enhancing strategic management approaches (e.g. better delineation of home-owners' rights and duties) appears to be the first priority to building performance in order to improve their buildings and living conditions.

To encourage more sustainable buildings at the community level, the assessment results can be summarized into two simple and user-friendly performance indicators for public consumption, namely the BHHI and the BSCI. By publicizing these performance indices, the public would be better informed of the health and safety risks of different buildings so that building owners, FM as well tenants can make more responsible decisions in the future. It is envisaged that further research can be conducted to investigate the relationship between building performance and extraneous factors, such as building age, management structure, and scale of development.

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MAINTENANCE MANAGEMENT SYSTEM FOR INDUSTRIALIZED BUILDING SYSTEM

Ng Ban Kiong[1] and Zainal Abidin bin Akasah[2]

[1, 2]University Tun Hussein Onn Malaysia

[1]wanqiang0505@hotmail.com , [2]zainal59@uthm.edu.my

ABSTRACT: With the aim to reduce the number of the unskilled foreign labor, government had encouraged our local contractor to use the Industrialized Building System (IBS). They are few type of IBS system in Malaysia and this included precast concrete frame, panel and box systems, steel formwork systems, steel frame systems, prefabricated timber systems and block work systems. But at the same time, when a building was built by the IBS system, the building also needed a system of technique maintenance of the building to prolong the building life. Hence, this paper will focus on the maintenance management system use for the building built by IBS. The objective of this paper is to identify building maintenance problems in IBS, to explain the development of maintenance management system for IBS. As such, the building need to be identified before the research start. The defect and the problem face by the owner of the IBS building will be collect and note down in a check list. This will be done by questionnaire and distributed to all Malaysia's government IBS's building management office. Interview session and site visit also will be done with the contractor and the building owner to understand the problem that facing by the IBS building that has been chosen. To prolong the building life cycle and prevent the wasting of the money, a systematic of maintenance management system (technique) used for Industrialized Building System need to be developed

Keywords: industrialized building system (ibs), maintenance management system, life cycle

1. BACKGROUND OF STUDY

In Malaysia, construction sector play an important role in generic country economy. Over the year in Malaysia, 3% to 5 % of the national Gross Domestic Product(GDP) for the pass 20 years was contributed by the construction sector. Apparently this statistic has obvious shows that the construction was one of the important major sector that stimulate our country's economy. A boom construction activities can show that country's economy was in brisk condition while the sluggish construction activities show that the nation's economy was under depression condition. Thus, the construction sector also can serve as an important barometer indicating our country economy condition.

Industrialized Building System(IBS) generally can be known as all building components which are mass produced either in factory or at site factory. All the mass component of the building will be design accordingly to specifications with standardize shapes and dimensions.

Eventually, when all the building component has been successfully produced, all the component will be transported to the construction site to be rearrange with certain standard to form a building(Lim, 2006).Construction industry in Malaysia generally comprise many process and this included many parties and different stages of work. Involvement of various parties from different sectors can effectively ensure the high efficiency of the construction that will be carried out. Only with the high quality of managerial and organizational performance with the effective co-ordination through good teamwork from different parties will create an efficiency and success of construction development and activities. Thus, every team member play an effectives roles in their work to ensure the development of the construction industry will be achieved.(.Tay,2006)

The concept of using Industrialized Building System (IBS) in Malaysia getting started after the ministry of housing and local government of Malaysia visiting several European country. This became the significant starting point for the IBS in Malaysia although IBS was not so popular at that era. Our

Malaysia construction field achieve another new mile stone when the pioneer of the project Pekeliling Flat at Kuala Lumpur has been successfully built within 27 months which utilized the panel pre-cast concrete wall and plank slabs in the project.

Nowadays, there are lots of local IBS manufacture and yet were mushrooming. Most of the IBS system used in Malaysia are large panel systems, steel frame, precast frame and formwork system. All this system has been largely used for private residential project in Malaysia which included projects in Shah Alam, Wangsa Maju and Pandan, Dua Residency, KL, Taman Mount Austin and Tongkang Pecah, Johor (CIDB, 2006).

In this 21st century, IBS is not new to the construction industry. This method had effectively can cost saving and improve the quality through the reduce of the labor intensity and construction standardization. Besides, it minimized the wastage, less site material, more cleaner and neater environment, quality controlled, and reduce the total construction costs. Successful implementation IBS in the world are Sesikui Home(Japan), Living Solution(United Kingdom),Open House(Sweden) and Wenswonen (Netherlands).

IBS should approach to the maintenance management. Maintenance management is a system that orderly and systematically that approach to the planning, organizing, monitoring and evaluating maintenance activities and their costs. Thus, a good maintenance management system could prevent health and safety problems and environmental damage, yield longer asset life with fewer breakdown and result in lower operating costs and a higher quality of life.

2. STATEMENT OF THE PROBLEM

In the 1960's, IBS was often misinterpreted with negative meaning. Normally, IBS building associated with pre-fabricated mass construction method, low quality buildings, leakages, abandoned projects, unpleasant architectural appearances and other drawbacks. Public have bad impression about the precast concrete due to the poor architectural design for the old pre-fabricated buildings. The example of two early pre-fabricated flats were constructed was Pekeliling Flats in Kuala Lumpur and Taman Tun Sardon,Gelugor,Penang. The very basic design for the Taman Tun Sardon by British Research Estalishment,UK was create lots of problem. The lack of design such as the need for wet toilets and bathrooms was leads to problems of leakage Futher more, many low cost housings are not maintained properly and this give the negative vision and poor image to the IBS building. (Rahman and Omar, 2006).
However, lack of knowledge in structural analysis and design of pre-fabricated components also contribute to the problem of the implementation IBS system. The most common problem are the connection between the beam to column and column to base. The lack of knowledge of design could cause the poor connection at site work. The poor connection may leads the issued of comfort and safety. When the steelwork structures are design as the conventional reinforced concrete structural system, this concept result exposed steel beams and columns. Unfortunately, this can comes out of many problem such as leakage. The rain water was easily seep into the building joint between the wall and steel beam. On the other hand, the dampness leads to corrosion to the lighting system and the beam (Rahman and Omar, 2006).

3. IBS DEFINATION

Actually there are various definition of the Industrialized Building System(IBS) in Malaysia. This indicated that no agreed definition of the IBS since there have few type of definition from different author (Mohamad Rofzdi, 2009). IBS can be consider as the building built by using pre-fabricated components. The component was systematically manufactured either by using machine or other forms of mechanical equipment. The component was manufacturing off site and delivered to construction sites for assembly and erection (Rahman and Omar, 2006).

Chung and Kadir (2007) has defined that the IBS is a mass production of building components either in factory or at site. It depends on the specification of the standard shape and dimension to transport to the site to be re-arranged according to certain standard to form a building. Construction Industry Development (2003) defined IBS as a construction technique included components which are manufactured in a controlled environment either on or off site, transported, positioned and set up into a structure with minimal additional site works.

Warswaki,1999 defined industrialized as an investment in equipment, facilities and technology with the aim to maximizing the output and minimizing the labour resources and improving the quality. For the building system, Warswaki defined as a set of interconnected element that joint together to enable the designated performance of a building.

Esa and Nuruddin,1998 stated that IBS is a kind of manufacturing process in order to minimize resources wastage and enhance value for end users through continuous beginning from utilizing craftsman for every aspects of construction to a system that make use of manufacturing production.

3.1 HISTORY IBS IN MALAYSIA

Since 1960's, IBS was starting used in Malaysia after the ministry of housing and local government of Malaysia inspired the idea from the visiting at several European countries. The pioneer project of building using IBS was Pekeliling Flat at Kuala Lumpur and the Rifle Range Flat at Penang. This two project significantly had cause the Malaysia construction field achieved another new milestone in the era of 1960's when IBS which also known as modern method construction had been recognized in Malaysia. Since then, many construction after the year using precast wall panel system. This method became popular when most of the low cost high rise residential building using IBS following the years after 1960's. But unfortunately some of the technique that had been utilized in Malaysia from over sea did not suitably use in Malaysia's weather climate and social practices. In the early of 80's, structural steel component getting using in high rise building in Kuala Lumpur (CREAM AND CIDB, 2010).

The 36-storey Dayabumi complex was successfully completed in 1984 by a Japan company name Takenaka Corporation. In the period of 1981 to 1993, Perbadanan Kemajuan Negeri Selangor (PKNS) adopted precast concrete technology from Praton Haus International based on Germany to build low cost house and high cost bungalow for the new townships in Selangor (Zuhairi Abd Hamid, 2007). The evolution of the construction in Malaysia had encourage many local company involved in IBS construction. To improve the IBS technique, some local company had collaborate with the foreign company from Japan, Australia and US to develop a quality technology in IBS. There are various of project utilized IBS in this modern era and this including Bukit Jalil Sport Complex, Kuala Lumpur Convention Centre, Lightweight Railway Train, KL tower, KLIA, and Petronas Twin Tower (Zuhairi Abd Hamid, 2007).

3.2 CLASSIFICATON OF IBS

Generally, there are four categories in construction method and this including conventional method, cast in situ, composite method and fully prefabricated. Basically cast in situ, composite method and fully prefabricated was classified as non conventional method. While for the structural aspects, IBS can be divided into five major types:

a) Precast Concrete Framing,Panel and Box System
   -this including PC columns, beams, slabs, 3D-components(balconies, staircase, toilet, lift chambers, box girders, etc), (MIIE, 2009)
b) Steel Formwork System
   -Tunnels forms, beams and colums moulding forms, permanent steel formworks(metal decks), etc (MIIE, 2009).
c) Steel Frame Systems
   -Steel beams and columns, portal frames, roof trusses, etc (MIIE, 2009)
d) Prefabricated Timber Frame Systems
   -Timber frames, pre-fab timber, roof trusses, etc (MIIE, 2009).
e) Blockwork Systems
   -Interlocking concrete masonry unit(CMU), lightweight concrete blocks, etc (MIIE, 2009)

3.3 PERCENTAGE OF FOREIGN LABOR BASED ON STATES

States in Malaysia	Average %
Selangor	97
Wilayah Persekutuan Kuala Lumpur	95
Melaka	86
Pahang	76
Johor	73
Negeri Sembilan	62
Pulau Pinang	52
Sabah	48
Perak	48
Kedah	36
Terengganu	24
Kelantan	17
Perlis	13
Sarawak	6
Average per state	52

Source: Malaysia International IBS Exhibition 2009

3.4 REGISTRATION OF IBS MANUFACTURER AND PRODUCTS IN "ORANGE BOOK"

NO	TYPES OF IBS	MANUFACTURER	PRODUCT
1	Precast Concrete System	39	249
2	Formwork System	23	29
3	Steel Frame System	27	45
4	Timber Frame System	21	21
5	Blockwork System	9	11
	Total	119	355

Source: Malaysia International IBS Exhibition 2009

3.5 REGISTERED IBS CONTRACTOR(ACTIVE) IN MALAYSIA BY CIDB GRADE (2007)

GRED	NUMBERS
G7	334
G6	52
G5	83
G4	42
G3	191
G2	76
G1	71
TOTAL	**849**

Source: Construction Industry Development Board(CIDB) Malaysia

4.0 DEFINATION MAINTENANCE

Maintenance can be defined as the combination of all the technical and administrative action which intended to retain or restore it to a state it can perform its required function according to BS8210. BS3811:1964 had defined maintenance as a combination of any actions that carried out to restore, retain and acceptable condition or standard. This combination of any action had included combination of many parties that involved in maintenance work which were contractor, worker, management and other parties. Retain and restore will well defined as there are two process which work are carried out in anticipation of failure and work carried out after failure. While word acceptable condition stand for the person which receiving benefit from the acceptability to the person paying. Each person which

paid for the work has it own acceptable condition based on the building, building life budget and workers.
From the Oxford Advance Learners's English Dictionary defined maintenance as the action of maintaining something or the state of being maintained. While Majdi(2002) has described maintenance as methods and techniques used to restore a specified level of service and to prolong life by slowing its deterioration rate.(Mohamed, 2010)
While according to Wordworth(2001), maintenance is the action referred to the initiation, organization, and implementation of series of works. There are two processes of works that envisage, retaining and restoring. Retaining is more to the work carried out in anticipating of failure and restoring is the work carried out after the failure. For the further explanation, the concept of maintenance acceptable standard is referred to the person who is paying the work to the person receiving benefit or to some outside body with the responsibility for enforcing minimum standards.(Hashim,2007)

4.1 IMPORTANT OF MAINTENANCE

Why maintenance was so important? Basically building and structure will last longer with proper and continuous maintenance. If a structure or building had a poor maintenance, this will resulted the need for reparation, renovation or restoration. Hence, this will increase the cost at the end of life cycle of the building or structure. The value of the maintenance is discussed from the aspect of:

i) Time
If compared the time needed for reparation and renovation, maintenance was totally less time then the reparation and renovation but can produce a better quality result. On the other hand, work qualities for maintenance are also relatively lesser compare to reparation and renovation.

ii) Cost
Cost required for maintenance are lesser then cost required for repair or renovation. When the maintenance work is carry on, the specific structure can still be running and this saving cost from the economic perspective.

iii) Structure value and performance
Structure will have high value and good performance during its service life if maintenance works are done according to schedule and plan. The unproper maintenance will cause the structure will not be able to provide services as its maximum performance all the time.(Yacob,2006)

4.2 FACTOR OF GOOD AND EFFECTIVE MAINTENANCE MANAGEMENT

Maintenance management consists of managing, planning and also controlling. In spite of that. There are four supporting factors that need to be considered in making the maintenance management more effective and efficient when it is executed.

i) The Organization structure and general responsibilities of maintenance management.
ii) The maintenance policies and standard for maintenance.
iii) The maintenance management planning and scheduling
iv) The maintenance management for budgeting and cost controlling.(Yacob,2006)

5.0 METHODOLOGY

5.1 INTRODUCTION

First of all, this research will start with identifying research problem which cover significance, objective and scope of study followed by exploratory research of the literature. All the information is gather from two kind of sources which firstly is journals, books, internet and reports while secondly is through preliminary interview to consultants, contractors, architectural firms, IBS manufacturer, CIDB and CREAM and site visit . Questionnaire will develop after all the information that related with the research will obtain from the potential respondents. All the questionnaire which was successfully develop will distribute to the potential respondents with either by post, hand or by email

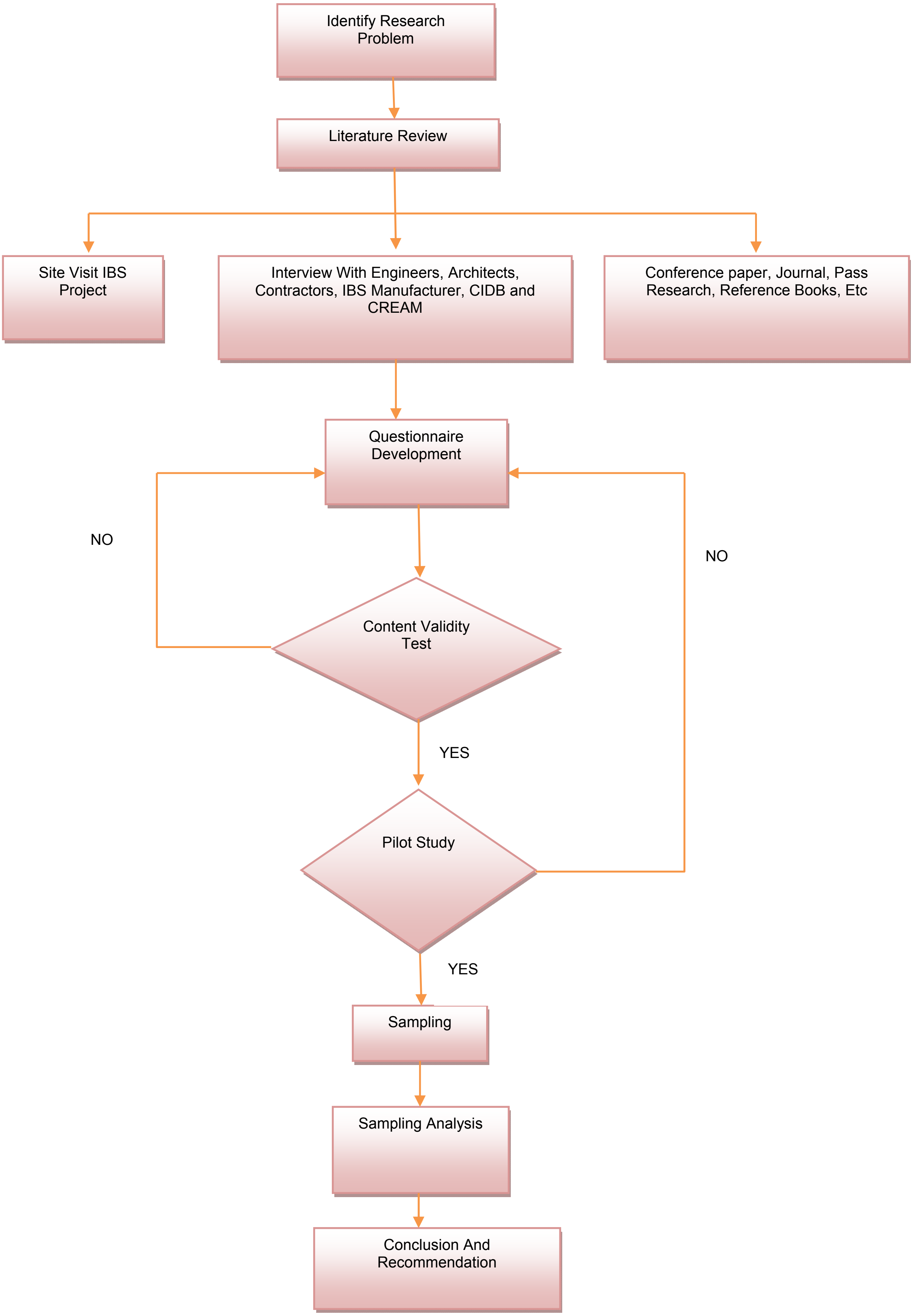
5.2 PROCESS OF METHODOLOGY

Process methodology need to arrange systematically to achieve the aim of the research study. This process started from the topic selection followed by identifying issues, objectives, scopes, data, data collection, data analysis, conclusion and recommendation and finally the thesis write up.

To achieve the objective of the research, the following steps were carried out such as:

1. A preliminary research on the maintenance management system for industrialized building system through the study of the literature.
2. Site visit, interview and discussion session with the contractors, engineers, architects, IBS manufacturer, CIDB and CREAM. This will help a lot to identify the datas and to understand the building maintenance.
3. Formulated questionnaires after the preliminary interviews and literature review was obtained.
4. The pilot study need to be conducted before distributed to the respondent. This pilot study are very important to:
   - Test the adequacy of the questions for each question.
   - Test the efficiency of the sampling.
5. After the pilot study has been achieved, the questionnaire was distributed by post, email or by hand to the potential respondent.
6. The questionnaire will be collected after the feedback from the respondent but the uncompleted questionnaire will exclude.
7. The questionnaire will be examined weighted overall factors that affect IBS building maintenance.

5.3 METHODOLOGY FLOW CHART



6. EXPECTED OUTCOMES

IBS maintenance management system will be develop and to be used by the contractors and the administrators of the building. This maintenance management system is important to prolong the building's life cycle and avoid the wasting of the money.

7. CONCLUSIONS

This paper significantly revealed the important of develop the maintenance management for IBS building in Malaysia. All the review that has been highlighted for example history, issues and problem of IBS in Malaysia hope will be an important information and reference to develop the maintenance management system for IBS building.

ACKNOWLEDGEMENT

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IMPLEMENTING LIFE CYCLE COSTING IN MALAYSIA CONSTRUCTION INDUSTRY: A REVIEW

Nor Azizah Mohammed Rum[1] dan Zainal Abidin Akasah[2]

[1,2]Universiti Tun Hussein Onn Malaysia

[1]jijah22@yahoo.com , [2]zainal59@uthm.edu.my

ABSTRACT: Globally, the construction industry is one sector that contributes to economic growth in Malaysia. The construction industry is developing rapidly and become one of the backbones of the country compared with other developing countries in the world. As developing countries are now moving towards sustainable development is in line with the 9th Malaysia Plan, development should be built to meet current and future needs for achieving sustainability of economic development, social and environmental responsibility for the prosperity without compromising the needs of future generations. Therefore, to ensure the government's objective was achieved construction industry must implement the Life Cycle Costing (LCC) method to construct the structures and infrastructure projects that it will not cause losses to the industry itself and government. Each stage of construction project begins with planning, design, construction, operation, maintenance and demolishes the project and it cost should be considered to ensure the overall project costs as are known. Through this LCC, factors and maintenance costs for a building products accounted at the design stage to obtain a more accurate projection of the cost. This paper will describe the LCC as a vital element in all sectors of the construction industry because this method can drive quality improvement by taking into account the cost of a comprehensive project. The aim of this paper is to understand the LCC for the construction industry in Malaysia expecially in maintenance phase. Secondly is to identify the process of LCC in buildings construction and last but not least is recommending that appropriate measures to implement the LCC in the construction industry in Malaysia. The main sources of the literature research will be taken through articles, journals, references books in the resources centre and internet search. Hopefully the implementation of LCC in the construction industry in Malaysia could be accessed and used for research reference by future researcher and as reference materials for the construction management industry in Malaysia.

Keywords: construction industry, life cycle costing, implementation, maintenance

1. INTRODUCTION

Construction industry is one sector that contributes to economic growth (CIMP, 2007). This industry is regarded as a stimulus to the development of other industries in introduced Malaysia as a developing country in the developed world. As the economy improved the construction industry will follow the same increase and at the same time demand will increase in economic construction. It is closely linked to the development of other sectors such as manufacturing, services, and etc. However, production of the construction sector is different from other sectors, it is not homogenous and each of it is considered unique and has its own value. The output of construction or design that is required must been meet the external influences which include sociology, politics, culture and economy (Aminah & Ismail, 2007).

According to the Economic Report 2010/2011, Ministry of Finance Malaysia (2010), development of construction industry was largely supported by the increase in civil engineering and non-residential property immediately following the implementation of development projects under the Ninth Malaysia Plan (9MP) and stimulus packages. The industry is estimated to grow 4.9 percent in 2010. Consistent with this report indirectly any infrastructure construction projects and commercial building ever built with the progress and success achieved by the country.

As a developing country, the construction industry in Malaysia is the most important and consistent where it involves a very detailed management on various matters such as management of construction materials, machinery, technology, labour, implementation and etc, so well that it can realize its aspirations as well as fulfilling people’s basic needs. This industry is important because it is like as a tool of economic management for government as the Gross Domestic Product First Quarter of 2010 (GDP) highlight that the growth momentum to continue in the construction sector this quarter with record growth of 8.7 percent which includes Sub-Civil and Non-Residential which is a major contributor to this strong economic growth.

However the more sophisticated approach is needed for the design and construction of building facilities that through a combination of economic theory and computer technology where it is seen in terms of cost to design and build (Dhillon, 1989, Kirk & Dell'Isola, 1995; Hyo 2008). Nevertheless, owners can expand their perspective to include the cost of the operation, maintenance, repair, replacement, and disposal (Sirin, 2007; Venkataraman & Pinto, 2008). Therefore, to ensure the government's objective was achieved construction industry must implement the Life Cycle Costing (LCC) method to construct the structures and infrastructure projects that it will not cause losses to the industry itself and government. LCC is the process of identifying and documenting the initial cost and total cost of the future on the development and operation of buildings during the lifetime of the building (Sirin, 2007, Bahr & Lennerts, 2008; Ade Asmi, 2009). Through this LCC, factors and maintenance costs for a building products accounted at the design stage to obtain a more accurate projection of the cost.

2. LIFE CYCLE COSTING

2.1 LIFE CYCLE COST DEFINITION

There are different terms used in the literature today like, *cost in use*, *life cycle costs* (LCC), *whole life costing* (WLC) and *whole life appraisal* (WLA). Where defined that the terminology has changed over the years from *cost in use* to *life cycle costing* and further to *whole life costing* (Flanagan & Jewel, 2005). They defined the new term *whole life appraisal* which is the systematic consideration of all relevant costs, revenues and performance associated with the acquisition and ownership of an asset over its physical / economic / functional / service / design life. It minimises total expenditure through proper appraisal of coats that will be incurred through the life facility.

LCC is a valuable technique that is used for predicting and assessing the cost performance of constructed assets. LCC is one form of analysis for determining whether a project meets the client's performance requirements (BS ISO: 15686-5, 2008). The Norwegian Standard defined LCC as including both original costs and cost incurred throughout the whole functional lifetime including demolition (NS 3454, 2000).

However, LCC also a process to determine the sum of all the costs associated with an asset or part there of, including acquisition, installation, operation, maintenance, refurbishment and disposal costs. It is pivotal to the asset management process as an input to the evaluation of alternatives via Economic Appraisal, Financial Appraisal, Value Management, Risk Management and Demand Management. LCC adds all the costs of alternatives over their life period and enables an evaluation on a common basis for the period of interest. This enables decisions on acquisition, maintenance, refurbishment or disposal to be made in the light of full cost implications (TAM, 2004).

According to Olanrewaju, Khamidi, Idrus and Shobowale (2010), the large sums of money are required to keep the facility in operations and in good conditions. Generally, in some cases running cost could be as high as 40% of the capital cost or even more. Anecdotal evidences have suggested that only about 10% of the total cost is actually required to complete the project. What this mean is nearly, 90% of the total cost of project is claimed by maintenance and running costs. The effects of the maintenance and running costs on the construction are so evident that any attempt to overlook them would be at the detriment of the client and in fact, to the professional competence of the designs’ and construction teams. Figure 1 show the LCC planning at different stages during a building lifespan.

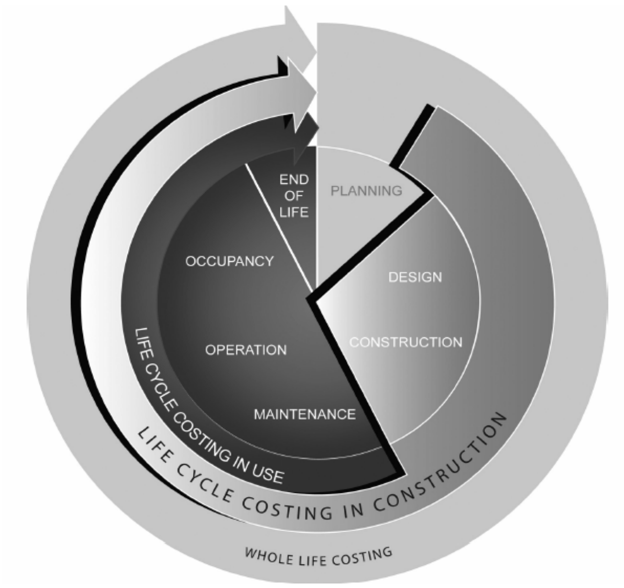

Figure 1. *Life cycle cost planning at different stages (BS ISO: 15686-5, 2008)*

2.2 LCC MODEL

According to Total Asset Management (2004), LCC model is an accounting structure containing terms and factors which enable estimation of an asset's component costs. A number of commercially available models can be used for LCC analysis. However, in some cases it may be appropriate to develop a model for a specific application. In either case, the LCC model should:

a. Represent the characteristics of the asset being analysed including its intended use environment, maintenance concept, operating and maintenance support scenarios and any constraints or limitations;
b. Be comprehensive enough to include and highlight thefactors relevant to the asset LCC;
c. Be easily understood to allow timely decision-making, future updates and modification; and
d. Provide for the evaluation of specific LCC elements independently of other elements.

Before selecting a model, the purpose of the analysis and the information it requires should be identified. The model should also be reviewed with respect to the applicability of all cost factors, empirical relationships, constants, elements and variables. Refer figure 2 below to know the process of LCC fundamentally involves:

a. Assessing costs arising from an asset over its life cycle; and
b. Evaluating alternatives that have an impact on this cost of ownership.

An asset can be any item that has a value to an organisation over time. Items such as buildings, physical plant and equipment and computer software are normally regarded as assets. Consideration of all these costs is important, even if they are funded from different sources within an organisation (Better Practice Guide, 2001).

One may describe LCC models as predictive in nature and characterize a stochastic process involving many parameters such as interest rates and reliability. Nevertheless, irrespective of the types of models used for LCC analysis they all must be visible, transparent and effective in representing systems, equipment, subsystems or devices (Dhillon, 2009).

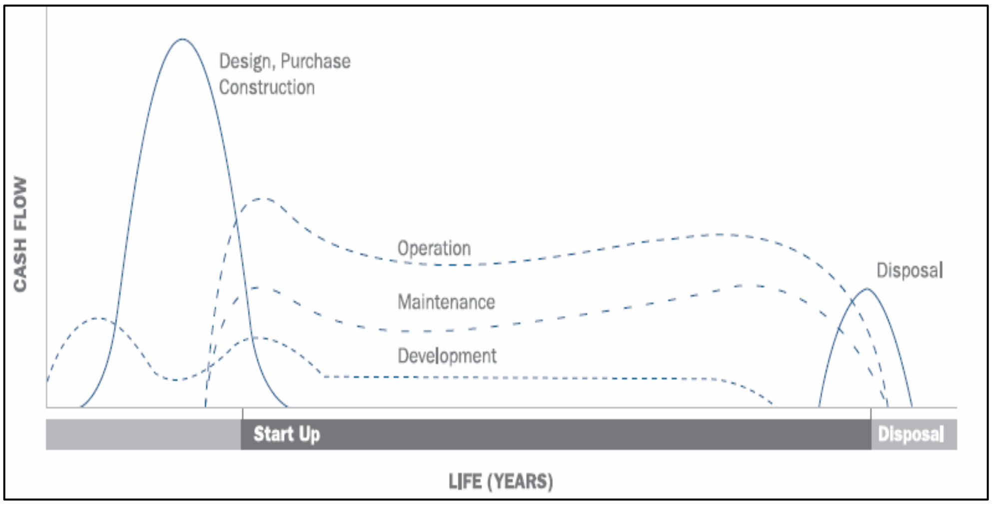

Figure 2. *Life cycle cost concept (Better Prctice Guide, 2001)*

2.3 LCC METHOD

Cost estimating is a popular activity as various kinds of costs have to be estimated each day. The estimated cost has to be as close as possible to its actual value; otherwise it may lead to a consequence of a severe degree. Most specially, the success or failure of an organization may depend on the quality of cost estimates however over the years, people working in cost estimating area have to come up with several different cost estimation methods, models and procedures (Dhillon, 2009).

The literature shows that the most suitable approach for LCC in the construction industry is the net present value (NPV) method. Existing mathematical LCC models, which are based on NPV, have various advantages and disadvantages, as they differ in the breakdown costs elements. The model from the American Society for Testing Materials as shown in Eq.(1) for example, distinguishes between energy and other running cost, which is useful in adopting different discount rates for different cost items.

$$NPV = C + R - S + A + M + E \quad \ldots\ldots\ldots\ldots\ldots..(1)$$

C = investment costs

R = replacement costs

S = the resale value at the end of study period

A = annually recurring operating, maintenance and repair costs (except energy costs)

M = non-annually recurring operating, maintenance and repair cost (except energy costs)

E = energy costs

According to Arja, Souce & Souyri (2009), LCC can be calculated at any phase of the building life in assessing the overall cost of the buildings which are under construction or already in service. There are basically few steps to go in LCC (Barringer, 2003), as shown below:

1. Identify what has to be analyzed and the time period for the project life study along with the appropriate financial criteria.
2. Focus on the technical features by way of the economic consequences to look for alternative solutions.

3. Develop the cost details by year considering memory joggers for cost structures.
4. Select the appropriate cost model, simple discrete, simple with some variability for repairs and replacements, complex with random variations, etc.
5. Acquire the cost details.
6. Assemble the yearly cost profiles.
7. For key issues prepare breakeven charts to simplify the details into time and money.
8. Sort the big cost items into a Pareto distribution to consider futher study.
9. Test alternatives for high cost items such as what happens if maintenance cost is up to 10% than planned, etc.
10. Study uncertainty/risk of errors or/alternatives for high cost items as a sanity check and provide feedback to the LCC studies in iterative fashion.
11. Select the preferred course of action and pelan to defend the decisions with graphics.

2.4 INTEGRATED LIFE CYCLE DESIGN

Integrated life cycle design is still at a phase of rapid development, and this is the start of the final formulation of a new integrated design process and methodology, which in future will serve as general design culture. The step will be in integration of the design, management and maintenance planning of buildings and civil infrastructures into comprehensive life time engineering (Asko, 2002). The main aspects included in the model are displayed in figure 3. The model might be able to address several facets of uncertainties, going beyond a traditional LCC model.

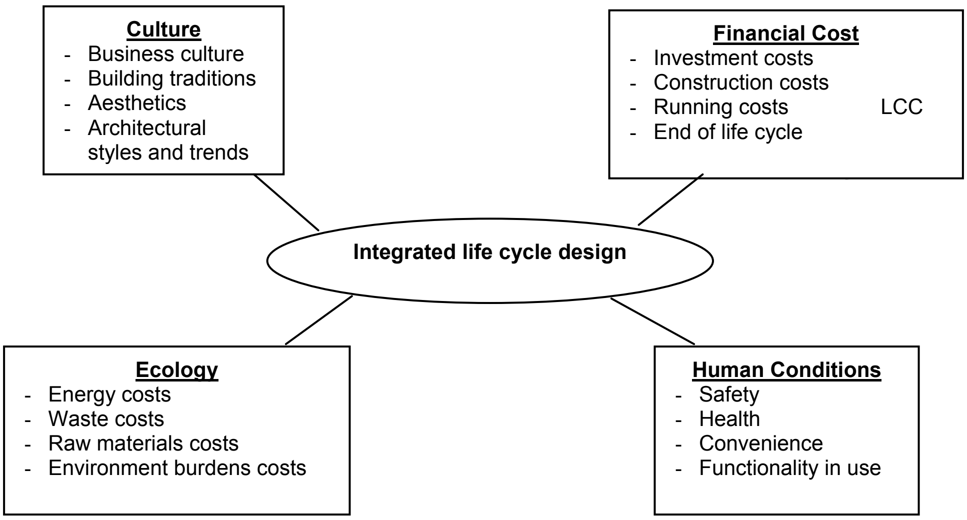

Figure 3. *Integrated life cycle design (Erika, Jutta & Lars, 2007)*

3. OPERATIONS AND MAINTENANCE

Maintenance means any activity or combination of actions that cover the technical aspects and management function undertaken to maintain, preserve or recover a thing to the original condition. It is also a combination of any action taken to repair or relocate the things or tools appropriate to the situation acceptable (Wood, 2009). However, it also the work other than daily and routine cleaning, necessary to maintain the performance of the building fabric and its services (BS: 8210,1986).

Maintenance also is the one of the most difficult decisions facing facility/asset owners is the timing of different types of maintenance work in order to keep facilities up to a proper and acceptable state of repair. Several policies are available, ranging from a short-term temporary repair to the undertaking of a full-scale renewal. The choice of action depends on a number of factors and as a result significant reductions in maintenance costs are likely to result from a life cycle approach:

a. The cost of different types of repair.

b. The rate of deterioration.

c. The relationship between the physical life of the repair and required physical, functional and economic life of the facility.

d. The disruption and disturbance to the building occupants and time required for the repair.

Maintenance repair consists of three stages: inspection, diagnosis and constructional or remedial action. The maintenance can be classified under the following subheading (Flanagan & Jewel, 2005).

Maintenance of the Main Structure: The main structure is exposed primarily to the natural elements and maintenance work will probably involve inspection and routine planned maintenance. *Maintenance of the Finishing/Fixture/Fittings:* The finishing suffers from wear and tear by the occupants and will require periodic renewal.

Maintenance of the External Works: The external works will require extensive maintenance with grass cutting, replacement of shrubs and tress and paving.

Maintenance of the Plumbing, Mechanical and Electrical Services: Each of the services elements will have its own maintenance requirements. While planned and preventive works will be undertaken, frequent corrective maintenance dependent upon the age will be needed.

Modernisation and Adaptation: This will often take place on a planned basis at a certain point in the facility life.

Redecorations: Internal and external redecorations will be necessary on a planned basis.

4. RESEARCH AIMS

The aim of this paper is to understand the LCC for the construction industry in Malaysia expecially in maintenance phase. Secondly is to identify the process of LCC in buildings construction and last but not least is recommending that appropriate measures to implement the LCC in the construction industry in Malaysia. Those objectives will show the important of the implementation of LCC in construction industry especially during in operation and maintenance phase.

5. SCOPE OF STUDY

The scope of this study is limited to the government building in Malaysia such as Institution building like several building in Universiti Tun Hussein Onn Malaysia (UTHM), Johor. The scope is chosen based on the higher cost for energy consumption in institutions of the industrial sector. Institutions are also a place for convenience in order to produce excellent and quality students.

6. RESEARCH METHODOLOGY

Among the approaches used is through searching the source of printed materials and electronic media to facilitate the study of literature carried out while the questionnaires, interviews and brief discussion with the supervisor, certain person and respondents also facilitate the empirical study. To obtain accurate information and detailed the methods of this study will be implemented gradually based on figure 4 below:

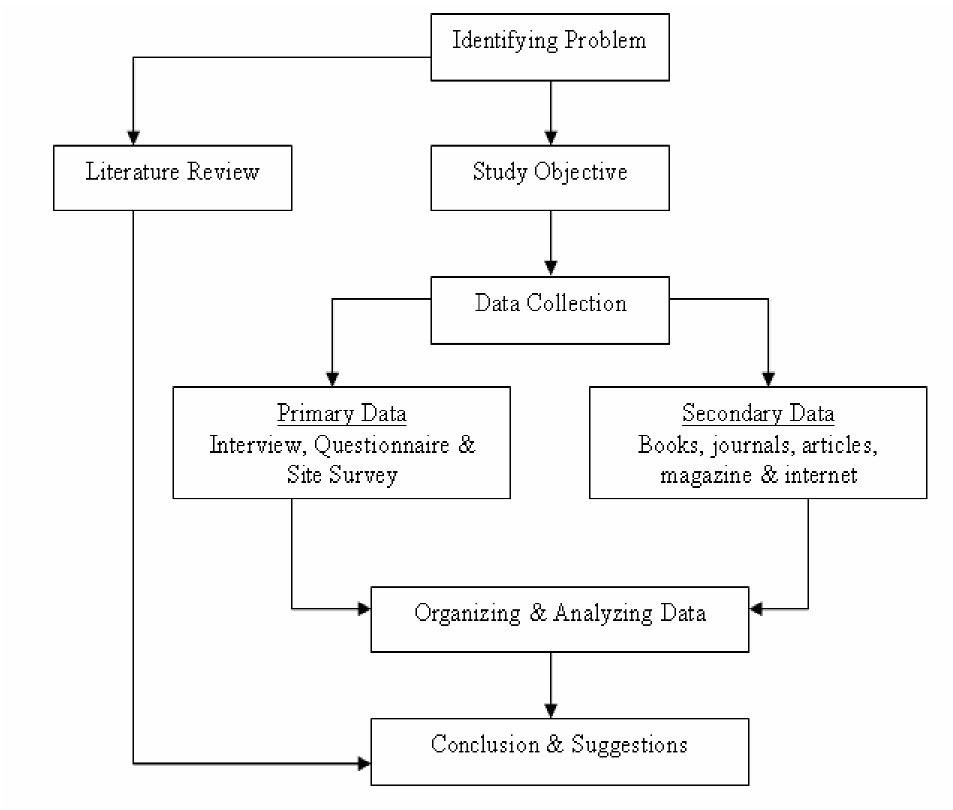

Figure 4. *Research Methodology Flow Chart*

6.1 RESEARCH POPULATION/SAMPLE

In this study, a questionnaire will be asked to certain people such as the occupant of the building, contractors, developer and building consultant to obtain statistically useful information about a given topic. It is the only feasible way to reach a number of reviewers large enough to allow statistically analysis of the results.

6.2 PROPOSED DATA COLLECTION METHOD

The interviews will be conducted with the questionnaire and also site survey to obtain detailed investigation information and data in maintenance problem and also to implement the life cycle cost approach to control the costing of the building during their life time.

6.3 Data analysis

This study data for questionnaire will be analysed by using Statistical Package for Social Sciences (SPSS) and Mircosoft Excel to calculate the costing with information from interview and site survey will be discuss for more detailed result.

7. EXPECTED OUTCOMES

Hopefully at the end of this research will answer the objectives of this study which are to understand the LCC for the construction industry in Malaysia expecially in maintenance phase. Secondly is to identify the process of LCC in buildings construction and last but not least is recommending that appropriate measures to implement the LCC in the construction industry in Malaysia. Those objectives will show the important of the implementation of LCC in construction industry especially during in operation and maintenance phase. Those objectives will integrate life cycle costing model in construction industry.

According to Arja et-al (2009), for futher suggestion is that the LCC analysis should be developed and extent in application for different type of bulding to obtain more reliable results and different LCC formule to contribute to other functions of building services such as building design and costing, in decision making and planning process.

8. CONCLUSIONS

As a conclusion, consideration of economic concept of time value of money involves variables such as cost, time and also discount rate (Cryder & Lally, 2008) in oder to compare the amout of money that will be spent over an extended number of years.

This research also will provide benefits and advantages not only to the building management but also favourable to the government bodies involved in the construction sector and to overcome certain problems in maintenance phase through the life cycle cost approach in government building. It also can create an effective and safe environment that is comfortable to the occupier. The building work will increase productivity indirectly, increase the value of investments and reduce maintenance costs. From the study that has been conducted, perhaps it will be parallel with the objectives that achieved through the methods which have been stated earlier.

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KEY FACTORS IN DEVELOPING MAINTENANCE CULTURE OF PUBLIC ASSET MANAGEMENT

Suwaibatul Islamiah Abdullah Sani[1] dan Abdul Hakim Mohammed[2]

[1,2]Jabatan Pengurusan Harta Tanah, Fakulti Geoinformasi & Harta Tanah (FGHT), Universiti Teknologi Malaysia (UTM), 81310 Johor Bahru, Johor, Malaysia

[1]sweetbatul85@yahoo.com , [2]hakim@fksg.utm

ABSTRACT: Malaysia is grow developing country and its visions in becoming a developed country with a first class infrastructural network. Towards this vision, asset facilities are developed especially buildings constructions and infrastructure to fulfill the need of society and organization. To ensure the functioning asset, minimum cost of repairing and providing a safe environment for user maintenance as activity those are undertaken to maintain and preserve asset. However, the asset especially public building and infrastructure are not maintained properly because the lack of maintenance culture. Therefore developing maintenance culture is hardly needed to increase the awareness about maintenance activity in managing public asset. Maintenance culture is an alternative measure that helps to increase the quality of maintenance and reducing the cost of repairing also ensuring the function asset and good safety environment. This paper will discuss and emphasize about nine factors in developing maintenance culture:- (a) Leadership, (b) Communication, (c) Reward and Recognition, (d) Organization Structure, (e) Teamwork (f) Training and education, (g) Motivation (h) Involvement and Empowerment (i) Policy of Maintenance.

Keywords: public asset, maintenance, maintenance culture

1. INTRODUCTION

Malaysia is a developing country that will achieve first class infrastructural network in 2020. Towards this vision, the public asset (physical asset) especially building and infrastructure are constructed to provide quality services as facilitating to fulfil the need of society and government organization. However numerous comments and arguments from various parties especially from the public, private and government sector about the condition of public asset did not achieve its function. The neglecting of maintenance public asset especially building damages is not repaired and replacement properly by parties involve of asset management. Part of this poor maintenance shows the lack of awareness in maintaining public asset by the government agencies. Maintenance activity or work process which is an important aspect in supporting business function in an organizational is to preserve for function of physical asset, reduce cost of operation and providing a safe environment for internal and external customer. In cultivating awareness concerning maintenance culture, the government or parties must held promotion and self-motivation in managing public asset. The new cultures about maintenance are needed to transform the traditional cultural and behavioural in the mind-set of all participants to improve quality maintenance. In developing maintenance culture the key factor which influences maintenance culture should be identified as a successfully quality maintenance performance**.**

2. MAINTENANCE IN PUBLIC ASSET MANAGEMENT

In February 2001, during the opening of the Kuala Lumpur 21 Convention and Exposition, the Deputy Prime Minister of Malaysia in referencing to the state of FM in Malaysia emphasized that: Unless Malaysians change their mentality to become more aware of the need to provide good services and improve the upkeep of buildings, we will forever be a Third World country with First World Infrastructure (Prime Minister of Malaysia, 2001). From this statement it describing, the vision of government to be first world infrastructure by providing the good services but cultural in maintaining public asset especially government building were not keep inside in each individual of all parties asset management. People do not have awareness the importance of maintenance work until maintenance work is not given a high priority in the organization.

Every year government spends a lot of money replacing and repairing the breakdown of public asset that are caused by poor maintenance. According to the Chief Secretary of Government of Malaysia, despite the government commitment with the allocation of RM1.079 million (US$306,012.47) for upgrading renovation, there is evidence of shoddy work and buildings falling into despair (Syahrul and Emma, 2009). Beside that The Ministry of Housing and Local Government received between 2400 and 4500 maintenance complaints each year over the last five years (Chuan, 2008) in (Idrus *et. al*, 2009). According Head of Maintenance Sector, Public Works Department Jabatan Kerja Raya (JKR), said that the standards of maintenance in government buildings are not up to the expectations (Annies, 2007). This is part of issue are discussed about maintenance problem in public asset management. Many factor are contributed to the poor of maintenance public asset but the critical factor is the lack of management commitment by maintenance group to dexterity and responsive maintenance work (Derek Miles *et. al*, 1987; Elizebeth, 1993 dan Adenuga *et. al*, 2007, Jalal 2003). The other word this problem caused by human fault that is lack of culture in maintaining public asset by government agencies which regarding about lower quality of work, lack of ethics in maintenance effective, attitudes to understanding maintenance work, experience and unskilled worker and manpower, less supervision from leadership, postpones and delayed repairing and replacing asset, management not provide unclear policy and standard to guide staff, less information about maintenance and lack of commitment to maintenance plan (Emma and Syahrul, 2009; Jalal 2003; Elizebeth, 1993; Hammad, 1995; Elizebeth, 1993; Adenuga *et. al*, 2007; Mark *et. al*, 2006; Angelo, 2003; Smith 1995; Onohaebi, 2010; Rondeau *et. al*, 2006 dan Leather *et. al*, 1998).

Maintenance culture in Malaysia needs a lot of efforts by the government to bring awareness in performing maintenance work properly for all parties that managing public asset. The government has outlined plans strategies to be implemented by all government agencies in the public asset management practices to create maintenance culture (DPAK Manual, 2009). In addition, the government allocated financial resources to create maintenance culture from the year 2005 until 2008 in maintaining public asset (National Budget 2005-2008).

3. MAINTENANCE CULTURE

Many authors are not mention about maintenance culture but the concept of culture are similarity from other field especially in quality and safety culture. Culture is defined as those practices common to a group people (Saidin *et. al*, 2008), it described the learned and shared values and norms that influence members behavior and interaction with each others in organization group (Mark *et. al*, 2006). Otherwise culture is a key influence behaviour that is what gets done and people need to do the right things, the structure and strategy of the work and the goals won't be acheived (Smith, 2006). Many researcher from other field especially safety and quality field believes that term quality and safety culture as a sub-facet of organisational culture that effect members attitudes and behaviours in relation to organisation.

Thus the maintenance culture a is part of organisational culture that influences each member in organisation. The word maintenance is a noun derived from the verb "to maintain". Maintenance means the process of keeping something in good condition (Telang, 2010). The concept of Maintenance culture is the internal environment between management and staff in managing maintenance effectively through the sharing of ideas, beliefs and values for each member in organization (Mark *et. al*, 2006). Maintenance culture is a way of thinking and behaving that can be drawn based on the actions taken by each individual in maintaining, preserving and protecting a system, equipment and structures. Involved with the maintenance of cultural beliefs, values, norms, practices and attitudes related to maintenance work. In other words, maintenance tasks into daily practice and embedded within each individual organization to appreciate the importance of maintenance (Suwaibatul, 2010).

4. DEVELOPMENT MAINTENANCE CULTURE

Cultivating the awareness in important of asset and facility management base on culture development in maintenance (Hakim, 2007). Developing maintenance culture is being recognised as an important aspect to increase quality maintenance work. Although culture is unique to each organisational (Yusof, 2006), it is need cultural change. Olusanjo (2006) stated that, creating maintenance culture should be established based on a cultural change in mindset and attitude people in the organization to enhance the knowledge and expertise performance of maintenance activities. This is because the cultural change needed to break indigenous moulds of poor perception, old patterns of now inappropriate behaviour and previous but now outdated, beliefs and values (Paschal, 1992). Smith (2006) stressed that, the work environment or culture is achieved by fostering learning, change, open communication, continual improvement and problem solving. Commitment from management is vital in creating maintenance culture for all employees in organization (Mark *et. al*, 2006).

However, the maintenance culture has not been comprehensively studied by researcher from maintenance field. According to Oedewald and Reiman (2003) addressed that maintenance work has not been studied much from the cultural perspective and former approaches have seldom taken into account the demands of the maintenance work in the entire organization. The term of culture in maintenance are important to enhance the level good of maintenance perfomance.

5. FACTORS IN DEVELOPING MAINTENANCE CULTURE

Discussion about the factors that influences the development of culture are more discussing by researchers from the view of quality and safety culture and several researchers of other field. Thus the key factors will be identified base on literature riview from quality and safety culture field to develop maintenance culture. From literature riview in safety culture eigth important factors that contribute for the cultural development. This include leadership, communication, team work, training and education, motivation, recognition and reward, policy and involvement (Turner,1991; ACSNI, 1993; Health and Safety Commission, 1993; Peterson, 1993; Ostrom *et. al*, 1993; Saidin, 2009; Eckhardt, 1996; Sayer, 1994; Wiegmann *et. al*, 2002; Farrington *et. al*, 2005; Cabrera *et. al,* 2006; Parker *et. al*, 2006; Donald dan Center, 1994). While from literature review in quality culture had ten factors influencing the development of culture. These include leadership, communication, teamwork, training and education, motivation, recognition and reward, policy, involvement, empowerment and organization structure. Deming, 1986; Person, 1992; Juran dan Gyna, 1993; Bubshait, 2000; Goetch and Davis, 2000; Rita, 2003; Carlo *et. al*, 2006; Yusof 2006; Tungkunan *et. al*, 2006; Viljoen & Wavaren 2008). As a result nine factors are indentified as a major influence in the development of maintenance culture, it consists of leadership, communication, teamwork, training and education, motivation, recognition and reward, policy, involvement and empowerment and organization structure.

5.1 LEADERSHIP

Leadership is a process of influencing the behavior group in organisation to encourage them performs the quality work towards achieving the goals. Koonntz and O'Donnell (1972) stated that leadership as skills to influence employees completed their tasks with great enthusiasm and confidence. According Saidin (2008) leadership is an essential part of the process of management and it also an integral part of the social structure and culture of organization. People in organization have beliefs about the leaders and their leader’s strengths and weakness can influence their work culture. Credible leadership would create the vision for organization, spread and foster the vision so that everybody buy the idea, actively participate during implementation and organize lifelong learning for the organization (Hakim, 2007).

The role of leadership can be show by commitment from top management to create culture in maintenance. According Argyris (1999) in Govindaraju and Daily (2004) commitment is about generating human energy and activating the human mind. The management commitment is vital and need to be demonstrated and visible to build work environment among employee based on conducting promotion of culture and provide information, reward system, training for improve employee skill, communicated about vision that won’t be achieved with employee and motivated employee with encouraging the good perform in maintenance work. Nakajima (1989) stated that top management’s primary responsibility is to establish a favourable environment where the work environment can support autonomous activities.

5.2 COMMUNICATION

Communication is the transfer information from a person to other member in organization by sending and receiving information or idea in form of oral or written (Carlo et al, 2006). Saidin, (2009) stated that communication as a tool in transferring ideas, statements, thoughts, feelings and values do that information are understanding by everyone in organization. Effective communication is important in the employee empowerment process. The use of teams is successful means for cross-functional communications in organization. Multi-tiered management structure inhibited communication (Yusof, 2006). Zohar, (1980) in Parker (2005) stressed that important effective communication that frequent and open contact between managers and worker was related good performance. According to Rita, (2003) communication is an instrument of positive interaction between top management and employee to perform a quality work. Communication can be achieved through a trust to employee, shared ideas and information within group member, accurate communicate with accurately, interaction to make friends, freedom of communication, open communication and communication between groups.

5.3 REWARD AND RECOGNITION

The recognition and reward would only be effective if it is meaningful and given an acknowledgment to the work produced (Saidin, 2008). It is what is received by an employee as reward for what their done in their work (Luthans, 2006). Gyna, 2001 in Viljoen and Waveren, (2008) are describing the recognition is defined as some sort of public acknowledgement for superior performance on quality activities and rewards are benefits such as salary increases, bonuses and promotions due to an individual's performance on improving the quality aspects of his or her job.

Rewards and recognition is the best when it is an inseparable habit of management and not awkward and delayed special nuisance. Recognition is a powerful motivator, because its reward behaviour immediately following that behaviour and there is a high degree of probability that behaviour will be repeated. The basic idea behind rewards is to give employees incentives for working harder and smarter (Viljoen and Waveren, 2008).

5.4 ORGANIZATION STRUCTURE

The organizational structure is a system of rules and power relations that control how people in organizations work together and interaction each other. It is also associated with the use of structural resources to achieve organizational goals. The main objective organization structure is formulated as tool in coordinating human action and to motivate people in the achievement of organizational goals (Hizam and Zahir, 2002). Rita (2003) stressed that the organizational structure is important because it is a framework for carrying out the responsibilities and activities within the company. Organization is a division of function and mechanism of action about the task must be carry out for each people, without a good organization structure the activities will not run very well.

5.5 TEAMWORK

Team work comprised a group of people work together to achieve a goal. Specially, Salas *et. al (*1992) defining that team work one may conceive a team to be a distinguishable set of two or more people who interact dynamically, interdependently, and adaptively toward a common and valued mission, who have each been assigned specific roles or functions to perform, and who have a limited life-span membership'. People in organization are involved in conveying their opinions on works that they do and strategies to solve problems (Saidin, 2008).

Teamwork occurs when two or more persons commit themselves to series of systematic actions based on brainstorming, discussing and exchange the information within another about task responsibility. Teamwork is universally accepted as the vehicle for change and the organizational mechanism for involving people in work improvement (Yusoff et al, 2006) and the role of team also influence change the individual behavioral (Rita, 2003). According Jawahar (1997) best activities in creating teamwork base on self managing work groups, coordination, team development, management reports, making decision in group, identifying problem, functional group, quality improvement and setting the goal must be achieved. Kirkman et al, (1998) stated that self managing work teams are needed for groups of employees in performing and managing their assigned work together. These teams operate with high levels of autonomy and responsibility to achieve specific goals defined by business plans.

5.6 TRAINING AND EDUCATION

Training is the development of attitudes, knowledge and specialized skills that required of an employee to perform job properly (Mahmood Nazar, 2005). Through training to develop human resources that enhance employee self-awareness, improve skills and knowledge of workers in certain areas and motivate employees to do the job better. Training should not be limited to transfer of technical skills and knowledge needed of optimal task performance. It should also cover generic matters like the business imperative peculiar to organization, problem-solving techniques, team dynamics and facilitation skills (Tsang, 2002). Thiagarajan and Zairi (1997) further addressed that education and training is the single most important factor once the necessary commitment has been assured and had become a long-term strategy in the planning schedule to obtain aspirations and skills. While education is important when explicit knowledge as knowledge that can be articulated in a formal education is one of the knowledge that acquired through education. Its forms the key in development of human nature with personal beliefs, perspectives and value system (Nonaka and Tekeuchi, 1995). According Saidin, (2008), new employees must be trained before there begin their work. This is purpose to provide the performed with good, to understand, to recognise the new work. The management are responsible for ensuring that the both new and experienced employees received the training.

5.7 MOTIVATION

Motivation is considered as processes of forming the personality of a person which can be encouraged to act in achieve certain goals (Ishak, 2006). Incentives that given for employees will increase confidence and encourage them carry out the quality of work. The successful to motivate employees depends on management role to full the employee need (Ali, 1995). Effective leaders must show high motivation and commitment to personal appearance in influencing attitude of employee. The motivation can be created such as reward, recognition and support by commitment management (Angelo, 2003).

5.8 Involvement and Empowerment

Empowerment is the process of delegating decision - making authority to lower levels within the organisation. Employees are encouraged to take the initiative and broaden their scope. Empowerment leads to improvements in ownership and responsibility and acts as evidence of managements trust in their employees' abilities (Viljoen and Waveren, 2008). While the employee involvement is a participative process to increase commitment the employee any action and affair. The connection in involvement between employees and management can develop attitudes, behaviors and work culture (Dennis, 1997 in Saidin, 2009). Involving in teams of managers and workers "may reflect a growing awareness of the synergetic effects of working together. Involvement and empowerment is connection each other. Employee involvement (also termed empowerment) is where employees are encouraged to participate and take responsibility for decisions and management of their day to day tasks. Stenvenson (1996).

5.9 Policy

Policies can be described as actions that describe standard, rule and beliefs that must be followed by an individual or organization to achieve the goals (Dennis, 1997). The policy is a directive, persuasion, explanation and determination of responsibilities for the management and to ensure responsibilities are carried out efficiently review. The presence of the police is essential to create an effective management and it can explain the standards and actions that should be followed by everyone in the organization (Saidin, 2009). Statement of policy system should be simple and explain the core values and beliefs of a firm with a clear and express about the common goals and objectives that want be achieved (Ismail, 2006). Crosby (1979) in Yusoff, (2006) stated that policy as a standard for practice that sets priorities of what to do and not to do, without a formal policy people will develop their own individual and differing standards of practice.

6. CONCLUSIONS

Developing Maintenance culture is needed in organization. This is pupose to raise awareness important of maintenance in keep inside for each member that managing public asset. The effectiveness of maintenance depend base on the human factor to increase knowledge and experience of workers in solving problems. Thus internal environment between management and staff must exist with sharing of ideas, beliefs, and values for each other. The development maintenance culture can make change of cultural work from traditional maintenance work based on reactive activity to productive activity. Nine important culture element from literature review that contribute to successful developing maintenance culture, which include leadership, communication, teamwork, training and education, motivation, recognition and reward, policy, involvement and empowerment and organization structure.

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PERFORMANCE OF SAND-CEMENT BLOCK WITH KENAF (*HIBISCUS CANNABUNUS L.*) FIBER

Aimi Munirah Jalilluddin[1], Kartini Kamaruddin[2]

[1, 2] Faculty of Civil Engineering, Universiti Teknologi Mara, Malaysia

[1]aimi_munirah86@yahoo.com[1], [2]ce_kartini@yahoo.com[2]

ABSTRACT: There is no doubt that issues relating to sustainability and green technology have been the main key of concern within the industry in the 21st century. Moreover, there is encouragement from the 10th Malaysian Plan (RMK-10) to employ sustainability and 'green' building design and technology in designing buildings or structures in Malaysia. Hence, a need in developing sustainable materials for the construction industry is on the demands. Interests on natural fiber can be seen progressively growing in Malaysia since it possesses ecological and economical value compared to synthetic fibers. Investigations have been actively conducted to incorporate these fibers (i.e. coir fiber and palm fiber) into cementitious materials and resulted promising results to be adopted into building materials. Meanwhile, masonry units have been seen to be used widely in the construction field and utilising agricultural products into these products may be capable of widening the applications of the natural fibers. Therefore, the present paper investigates on the compressive strength properties of sand-cement masonry units (i.e. blocks) by incorporating kenaf (*Hibiscus cannabinus L.*) fiber with various percentages (0% - 2% by cement weight) with a cement to sand batching system of 1:2.5. A total number of 60 blocks with the dimension of 360mm x 180mm x 60mm were casted and air-cured in room temperature of 29°C. In this study, fixed water to cement (w/c) ratio of 0.5 and the length of fiber by 25mm long was specified throughout the study. The samples were evaluated based on the compressive strength taken at 7 and 28 days. It can be seen that the incorporation of 0.5% kenaf fiber was the optimum value in achieving the highest compressive strength. Furthermore, higher percentages of kenaf fiber were also found to reduce the self-weight of the block samples hence a lighter building material can be produced.

Keywords: compressive strength, masonry units, natural fiber, kenaf, sustainability

1. INTRODUCTION

Masonry unit is a common material that has been widely used within the construction industry to build structures of housing (i.e. low and medium rise construction) and non-housing (i.e. industrial, commercial and educational buildings) sectors (Hendry, 2001). Its application varies to the requirements needed for a particular element which includes performing as external or internal walls or can also be functional for load bearing and non-load bearing walls. Employing masonry units has several advantages including economical construction without heavy and expensive plant, low cost, low maintenance, good durability and having desirable aesthetical value to the structure (Hendry, 2001; Sousa et al., 2004). Meanwhile, it has been a current trend in today's construction technology to employ 'green' and sustainability design techniques for proposed buildings where it is also being urged in the 10th Malaysian Plan (RMK-10). In corresponding to the matter, numerous studies have progressively investigated on the potential value to adopt and utilize environmental-friendly materials into practical products for the usage of the industry. Moreover, natural fibers (i.e. coconut husk, sisal, palm, bamboo, sugarcane, wood and vegetable fiber) have been gaining significant interests to be explored as alternative raw materials to synthetic or inorganic fibers due to the benefits gained including being cheaper, lighter, renewable, non-abrasive and superior in terms of health and safety issues (Wambua et. al., 2003; Joshi et. al., 2004; Mahat et al., 2010; Ramli and Dawood, 2010). It has been observed from previous studies that incorporating fibers into cementitious materials serves the principle of providing reinforcement within the matrix which increases the tensile strength, increasing the fracture toughness, increase fracture roughness, modifying properties of elastic modulus and reducing the density (Kolop et al., 2008). Furthermore, it can be further described that the natural fibers was found to be comparable to the properties of synthetic fibers (Wambua et al., 2003). Meanwhile, incorporation of the natural fibers into cement pastes, mortar and concrete still remains a central focus that can be investigated to utilize agricultural products as much as possible by widening the applications of such materials (Ramli and Dawood, 2010). Hence, this study focuses to evaluate on the compressive strength development of sand-cement block by incorporating with various percentages of kenaf (*Hibiscus cannabinus L.*) fiber (0% - 2% by cement weight) at 7 and 28 days. Moreover, the density of the samples was also determined to observe the correlation of kenaf fiber inclusion into the sand-cement blocks.

2. EXPERIMENTAL PROGRAMME

This paper focuses to evaluate the compressive strength performance of sand-cement block incorporated with different proportions of kenaf fiber. The block samples were prepared by employing a mortar mixture of cement to sand (C:S) batching system of 1:2.5 and was further divided into six (6) groups indicating the various percentages of kenaf fiber (0% - 2% by cement weight) used in the study. The density of the blocks was also considered for assessment to observe the influence of kenaf fiber to the weight gained by the sand-cement block samples.

2.1 MATERIALS

The cement used in the mortar mixtures was Ordinary Portland Cement (OPC) of Type I, a product manufactured by Tasek Corporation Berhad. The chemical compositions of OPC used in this study are shown in Table 1. Meanwhile, the fine aggregate used was well-graded sand classified through the sieve analysis in accordance to BS EN 933-1: 1997 where the maximum size of fine aggregate particles used was 5mm. The sand was sieved in order to separate the deleterious and larger particles in presence of bulk form that could affect the strength. The type of natural fiber used was bast-typed kenaf fiber supplied by Everise Crimson (M) Sdn. Bhd. which was combed initially to loose the 'contact' between each fiber. The fibers were then measured to 25mm in length and cut into short fibers for mixing into the mortar paste.

Table 1. *Chemical Compositions of Ordinary Portland Cement*

Material	Chemical composition (%)							
	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	K_2O	LOI
Portland cement	15.05	2.56	4.00	1.27	72.17	0.08	0.41	1.33

2.2 Mix Proportions

The mix proportion for block samples casted was shown in Table 2 where allocation of six (6) groups were determined by the different percentages of kenaf fiber incorporated into the mix (0% - 2% by cement weight). Meanwhile, fixed water to cement (w/c) ratio of 0.5 and fixed length of fiber of 25mm were also specified for the study. The kenaf fiber was added into the mortar paste by the addition principle where the fiber inclusion into the mixes does not modify the cement and sand content.

Table 2. *Mix Design Proportions*

C:S ratio	C	S	W	Kenaf (kg/m^3)				
				0%	0.5%	1%	1.5%	2%
1:2.5	513	1282	256	-	2.57	5.13	7.70	10.26

Note: C-*cement,* S-*sand,* W-*water*

2.3 FABRICATION OF SAMPLES

The study was carried out by preparing a total of 72 sand-cement block samples with the dimensions of 360mm x 180mm x 600mm. The method employed during the mixing stage of the mortar paste was a technique carried out by Kolop et al. (2008) where the constituents (i.e. sand and cement and fibers) were mixed together for several minutes to ensure that the materials were evenly dispersed before adding the water. Nevertheless, the approach also takes into account by allowing the fiber to be saturated in water for 30 minutes and assuming that the fibers are in a saturated surface-dry (SSD) condition. Therefore, fibers will be prevented to absorb water from the mix, thus adjustment to the water to cement (w/c) ratio is negligible. The mortar paste was then casted into the steel moulds and vibrated. After 24 hours, the moulds were demoulded and air-cured in room temperature of 29°C until the day of testing.

2.4 TEST METHODS

a. Compressive Strength

The compressive strength was conducted in accordance to BS EN 772-1: 2000. A number of six (6) block samples with dimensions 360mm x 180mm x 600mm were used for the strength performance assessment at the Concrete Laboratory of Civil Engineering Faculty, UiTM Shah Alam. The test conducted was performed by using the Compression Auto Test machine which has the capacity of 3000 kN and the rate of load employed was 2.75 kN/m. The block samples were evaluated on the compressive strength at 7 and 28 days of air drying.

3. RESULTS AND ANALYSIS

3.1 COMPRESSIVE STRENGTH

Figure 1 shows the average value obtained from six (6) samples tested on the strength performance of the sand-cement blocks incorporated with different percentages of kenaf fiber at 7 and 28 days. It can be observed that the highest compressive strength at 28 days gained by the sand-cement blocks was achieved by incorporating 0.5% kenaf fiber with the value of 22.21N/mm^2. In comparison of 0.5% sample to the control sample (0% kenaf fiber), there is a slight increase of strength development of about 3.3%.

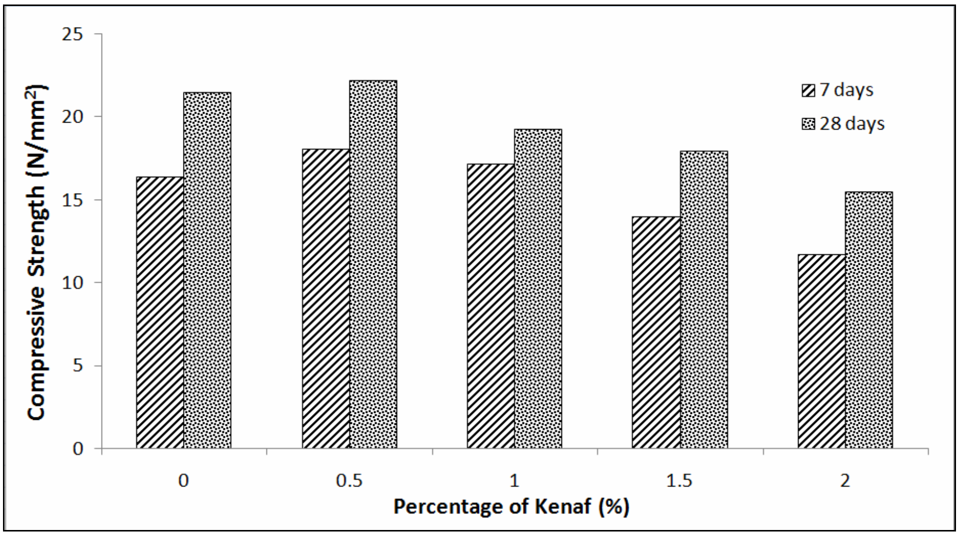

***Figure 1**. Compressive Strength of Sand-Cement Blocks Incorporated with Kenaf Fiber at 7 and 28 days*

Meanwhile, it can also be stated that higher incorporation of fiber content shows a higher reduction on the compressive strength of sand-cement blocks where fiber content exceeding 0.5% of kenaf fiber shows a rapid drop between 14% - 30% trend on the strength development. This study appears to be parallel to the findings of Kolop *et al*. (2008) where the compressive strength reduces with higher content of oil palm empty fruit bunches (EFB) fiber. In fact, this study is also in line with Ismail (2007) where remarked on every 0.5% addition of fiber resulted to strength reduction of approximately 10% at 28 days. Furthermore, the strength improvement from day 7 to day 28 illustrates a rapid increase between 11% – 24% can be seen in the strength development. The following Figure 2 shows the relationship between the compressive strength and the different percentages of fiber inclusion where it shows clearly that a slight increase occurs from 0% to 0.5% fiber incorporation, rapid drop from 0.5% to 1.0% and a gradual decrease in the strength development with fiber addition exceeding 1.0%.

Hence, it can be stated that the optimum compressive strength is practically obtained with lower percentages of fiber where uniform distribution in the mortar matrix is mainly achieved (Kriker et. al, 2005).

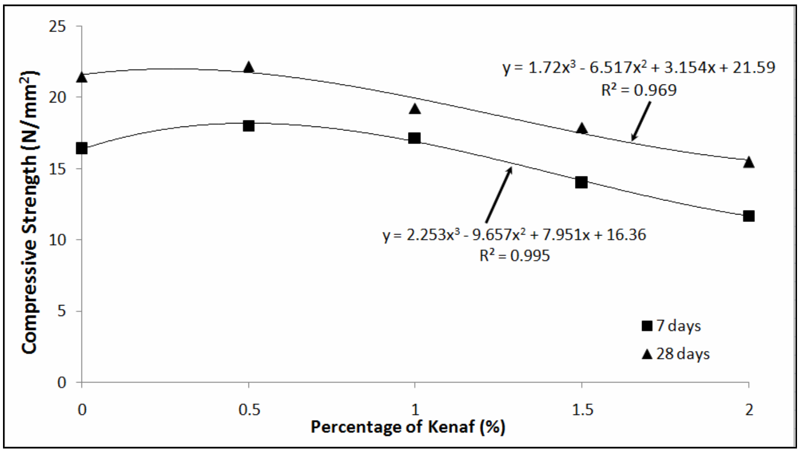

Figure 2. *Relation between Compressive Strength and Fiber Content at 7 and 28 days*

It can be stated that the reduction in the compressive strength is attributed by the presence of additional voids which exist in the mix due to presence of fiber which may lead to reducing the bonding between the cement matrix and disintegration (Djoudi and Khenfer, 2005; Ramli and Dawood, 2010). In this state, porosity will also be developed which gives direct impact to the strength development resulting to the reduction of strength (Djoudi and Khenfer, 2005). In fact, the occurrence of voids within the cement matrix may also be contributed by insufficient compaction due to high fiber content (Ismail, 2007). Furthermore, presence of fiber in the matrix is vulnerable to axial load hence it does not play a significant factor in the development of strength performance (Kolop et al, 2008). In addition, the strength gained by sand-cement block of 0.5% fiber inclusion can be considered to fall within the sand-cement brick category of 20N/mm^2. Meanwhile, sand-cement blocks incorporated with fiber exceeding 0.5% is comparable to sand-cement brick category of 15N/mm^2 - 20N/mm^2 in reference to BS 1180: 1972. Therefore, it can be functional to be applied as an infill of non-load bearing blocks within the walls of a structure or as external walls of cavity walls, solid construction and free standing walls. In fact, the results obtained for the sand-cement blocks is also comparable to clay bricks of class 2 and 3 (load-bearing brick) whilst class 2A and 2B (facing or common brick) or 3A and 3B (load-bearing or facing brick) when compared to calcium silicate bricks as stipulated in BS 3921: 1985 and BS 187: 1978 respectively (Kamaruddin, 2000). Hence, sand-cement blocks incorporated with kenaf fiber can be seen to have the potential to replace or act as a substitute to sand-cement bricks, clay bricks and calcium silicate bricks for building materials.

3.2 DENSITY

Figure 3 shows the effect of different percentages of kenaf fiber to the density of sand-cement blocks at 7 and 28 days. It can described that the inclusion of kenaf fiber in the cement matrix resulted lowering the density of the sand-cement block samples in comparison with the control samples. The reduction in density with lower fiber content of 0.5% - 1.5% kenaf fiber only gave a slight effect between 0.5% - 2% compared to higher inclusion of kenaf fiber by 2% which shows a higher gap of 7% density reduction to the control samples. Hence, it can be stated that the density of sand-cement blocks decreases as the content of fibers increases.

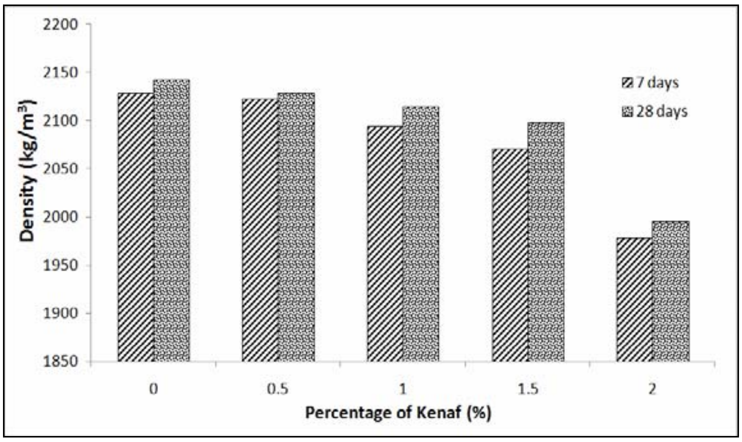

Figure 3. *Density of Sand-Cement Blocks Incorporated with Kenaf Fiber 28 days*

The reduction in the density can be considered as the result of the addition of kenaf fiber into the sand-cement blocks where the fiber content reduces the amount of sand-cement matrix within the volume per unit blocks hence dominating the space of the matrix by filling up the voids with fiber. Therefore, the more fiber is included into the mix subsequently replaces the constituents (i.e. sand and cement) with an additional factor of an external element that consumes the original space of the cementitious matrix. In fact, the natural characteristic of the fiber itself as a lightweight material also contributes in reducing the density of sand-cement blocks. On the other hand, the inclusion of fiber within the matrix may also result to insufficient compaction that might have been occurred and achieving the maximum density is possible (Ismail, 2007). Moreover, the lightweight factor of the sand-cement blocks can be expected to reduce the load from the structure to the foundation element of a structure. In fact, heavy density of materials or products gives disadvantages when it is applied as a building material for construction. The increase in density hence results to difficulties of working with heavy loads during transportation and handling. Moreover, lower density may give significant benefits in terms of load-bearing elements where the total weight of the building will be lower that corresponds to reduction of the size of foundations hence permits low load-bearing capacity to the ground (Neville, 1995).

Meanwhile, Figure 4 shows the relationship between the density and the different percentages of fiber content. It can be observed that gradual reduction in the density occurs when incorporating kenaf fiber of 0.5% to 1.5% into the cement matrix whilst, a rapid drop of density took place of fiber inclusion of more than 1.5%. The best fit-line to show the correlation between both parameters was by the polynomial of power three where the magnitude correlation coefficient, (R) of the curve shown is 0.99% and 0.993% for 7 and 28 days respectively.

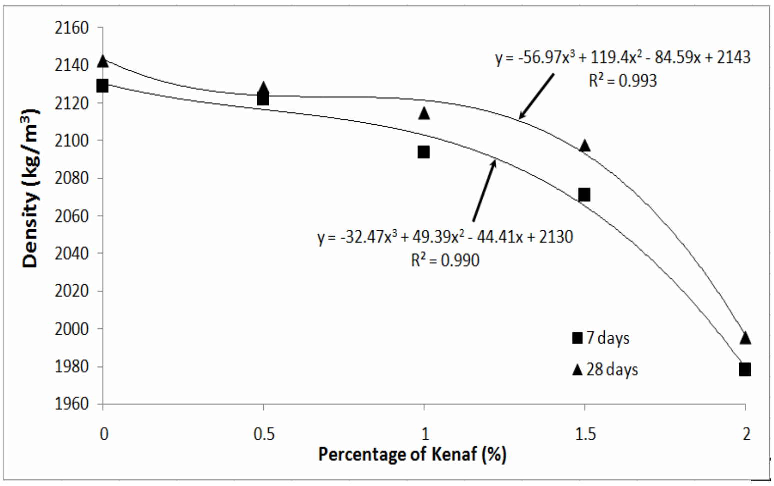

Figure 4. *Relation between Density and Fiber Content at 7 and 28 days*

Figure 5 shows the regression analysis correlating the density to the compressive strength of sand-cement block incorporated with kenaf fiber. The best fit-line to show the correlation between the two (2) parameters was by the polynomial of power three where the magnitude correlation coefficient, (R^2) of the curve shown is 0.99%. It indicates that the correlation between both elements is good.

It can be described from Figure 5 that the highest compressive strength gained by the sand-cement block incorporated with kenaf fiber was discovered when the density of the block was not in the maximum state. This condition can be explained by the presence of fiber within the cementitous material which may have a significant influence to the cement matrix whereby it may improved the bonding between the matrix and other elements (i.e. compressive strength and density) of the cement matrix thus the results obtained from this study is out of the norm.

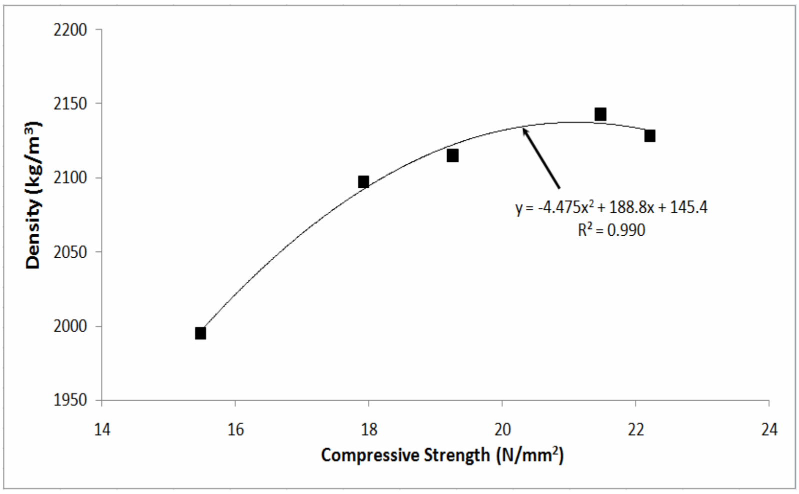

Figure 5. *Relation between Density and Compressive Strength at 28 days*

4. CONCLUSIONS

The following conclusion can be made from the study of sand-cement block incorporated with various proportions of kenaf fiber where:

1. Incorporation of 0.5% kenaf fiber in sand-cement blocks is considered the optimum value of kenaf fiber inclusion.
2. Incorporation of more than 0.5% kenaf fiber does not show any significance to the development of compressive strength.
3. Incorporation of high content of kenaf fiber resulted in the reduction of sand-cement block density.

ACKNOWLEDGEMENT

The authors would like to thank the Faculty of Civil Engineering, Universiti Teknologi MARA (UiTM) Shah Alam for supporting this study and Everise Crimson (M) Sdn. Bhd. for supplying the materials for the investigation.

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DATA CENTER OPERATION OPTIMIZATION USING COMPUTATIONAL FLUID DYNAMICS (CFD).

Rawnee Ho[1] and Dr.M.R Ismail[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]rawnee@yahoo.com , [2]rodzi@usm.my

ABSTRACT: In the past the Data Centers design for ventilation and air conditioning have been determined by the requirements of Computer kW per unit. The knowledge about the air flow patter/situation inside the Data Centers was unable to carry out because a precise experimental determination of the essential flow parameters is only possible with expensive measurement equipment. Computational fluid dynamics (CFD) can play important roles in Data Centers design/study by providing complementary information about the Data Center buildings environmental performance Computational fluid dynamics (CFD) simulation and modeling is currently the most details Data Centers ventilation and comfort analysis method widely available to study the air flow pattern. Integration of CFD will enable us to get an accurate prediction of the Data Centers building air flow pattern. Many of the calculation and assumptions is eliminated with the help of complementary model available on the CFD simulation study. A case study will be carry out to study the air flow pattern and describe the outcome of CFDs finding. This will be an opportunity on energy saving for Data Centers operation optimization. This paper illustrates a typical dynamic approach on using CFD software to study the 3D air flow pattern of the Data Centers.

Keywords: definition, computational fluid dynamic (cfd), data centers (dc), tileflow, airflow, raised floor & air flow model

1. INTRODUCTION

Computational fluid dynamics program provide complementary information about the performance of buildings. Innovative Research Inc. programs, such as Tileflow™ a powerful three dimensional software tool for simulating cooling performance of data centers. It uses the state-of-the-art computational fluid dynamics (CFD) techniques, and is applicable to both raised-floor and non-raised-floor data centers.

TileFlow has been specifically designed to give the user the ultimate ease-of-use and speed in setting up the data center model and examining the results. In addition, TileFlow has an unmatched speed in performing the calculations. We believe--and this has been confirmed by many experts in the field--that no other CFD software product in the HVAC area or even other fields has the ease-of-use and speed of TileFlow. Further, TileFlow presents the simulation results in a variety of colorful and attractive ways, making it an excellent communication tool (in addition to a reliable analysis tool). TileFlow is being used worldwide by data center managers, equipment manufacturers, architectural and engineering firms, and consulting companies.

CFD is a very powerful tool for simulation of natural ventilation driven by winds effect, stack effect, or both. CFD also can easily determine the temperature distribution and convective heat transfer coefficient with the help of building energy simulation programs. With the help of building energy simulation, it will able to provide critical information such as air-conditioning loads and surface temperature. This information is important for the success of this study, without the building energy simulation programs, the CFD programs will only able to compute results based on estimated boundary.

2. DATA CENTERS AIRFLOW MODELING

Data centers are facilities that house computer servers, data storage systems, and telecommunications equipment. To ensure that these computer systems operate reliably, they must be adequately cooled: The airflow requirement of each server rack must be met, in order to maintain the rack inlet air temperatures within acceptable range. The airflow distribution and rack inlet temperatures are controlled by complex fluid dynamics processes and depend on a large number of parameters, which often interact in a counter-intuitive manner. These parameters include the layouts of perforated tiles, CRAC units, supply and exhaust ducts, supplemental cooling units, and server racks; the open area of perforated tiles; the heat loads and airflow demands of the racks; and, obstructions under and above the raised floor.

Currently the data center floor layouts are designed using empirical guidelines based on limited measurements. These guidelines do not consider the complex fluid dynamics processes that control the airflow and temperature distribution. Consequently, the layouts do not produce the expected flow rates and rack inlet temperatures, and must be modified. However, because modifications in one region of the floor influence airflow and temperatures throughout the floor, considerable trial and error is involved in identifying adjustments that will yield the desired changes in the rack inlet temperatures. This design practice is time consuming and expensive, and often the resulting arrangement is not optimum. Computational Fluid Dynamics (CFD) modelling or airflow modelling offers a more scientific and comprehensive design approach.

Computational simulation can be used for a quick setup of any proposed layout, any desired placement of CRAC units and perforated tiles, and any imagined failure scenario. The “computational” trial-and-error process is preferable for two reasons. First, performing a simulation is much faster and more economical than building an actual layout. Second, the computed results provide not only the flow rate distribution through perforated tiles and rack inlet temperatures but also the underlying velocity, pressure, and temperature fields and thus explain the physics behind the results. This understanding is useful in guiding the computational trial-and-error process in the optimum direction. Figure 1: Temperature Distribution and flowpattern on a vertical plane (at a Data Centers Command Room). TileFlow constructs a computer model of the data center and uses the technique of Computational Fluid Dynamics (CFD) to calculate the airflow pattern and pressure/temperature distributions. TileFlow is a reliable, fast, and cost-effective tool for:

- Designing efficient data centers
- Evaluating options for positioning new equipment
- Examining "what if" scenarios
- Streamlining installation and commissioning
- Preventing heat related outages of computer equipment
- Making cost-effective investments in cooling-related hardware

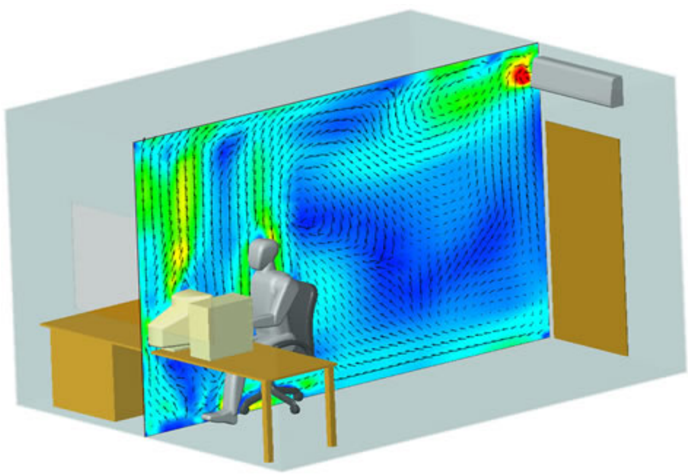

Figure 1. *Temperature Distribution and flowpattern on a vertical plane (at a Data Center Command room)*

3. THE COOLING CHALLENGE IN DATA CENTERS

The Uptime Instituted. Data centers are facilities that house computer servers, data storage systems, and telecommunications equipment. To ensure that these computer systems operate reliably, they must be adequately cooled: The airflow requirement of each server rack must be met, in order to maintain the rack inlet air temperatures within acceptable range.

The distribution of cooling air and the rack inlet temperatures (cooling performance) are controlled by complex fluid dynamics processes and depend on a large number of parameters, which often interact in a counter-intuitive manner. In a raised-floor data center, these parameters include the layout of perforated tiles, CRAC units, and server racks; the open area of perforated tiles; the heat loads of the racks; and, obstructions and blockages under and above the raised floor. Figure 2 show the temperature fog (3-Dimensional temperature distribution above the raised floor).

Often the design practices for data center cooling are based on empirical guidelines and prior experiences. These practices do not consider the complex fluid dynamic processes that control the airflow and temperature distribution. Consequently, they lead to inefficient and ineffective data centers. The key to designing an optimally cooled data center is a good understanding of the airflow patterns and temperature distribution in the data center. This understanding is best achieved through computer modelling of the fluid flow and thermal processes, that is, by computer simulation of the cooling performance of the data center.

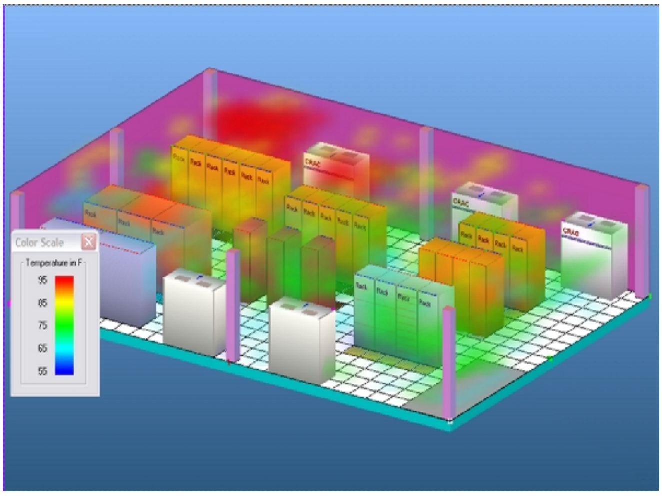

***Figure 2**. Temperature fog (3-Dimensional temperature distribution above the raised floor)*

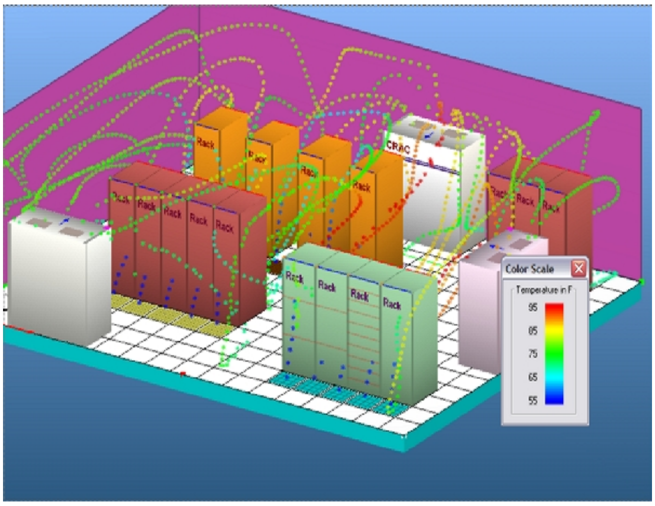

***Figure 3.** Animation of air movement above the raised floor (color rep the air temperature)*

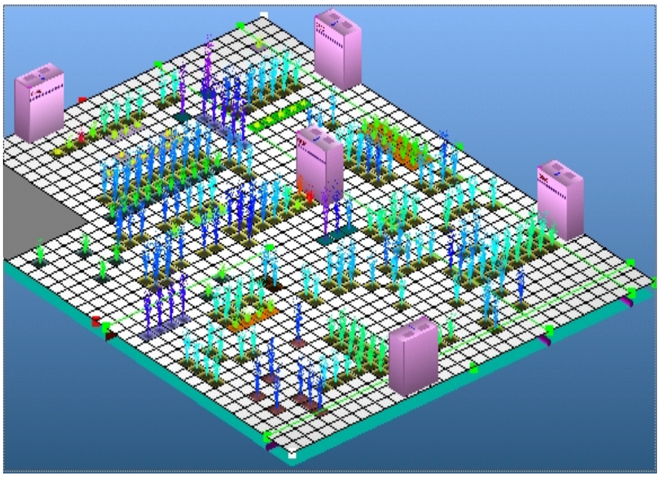

Figure 4. *Display of cooling plumes through the perforated tiles*

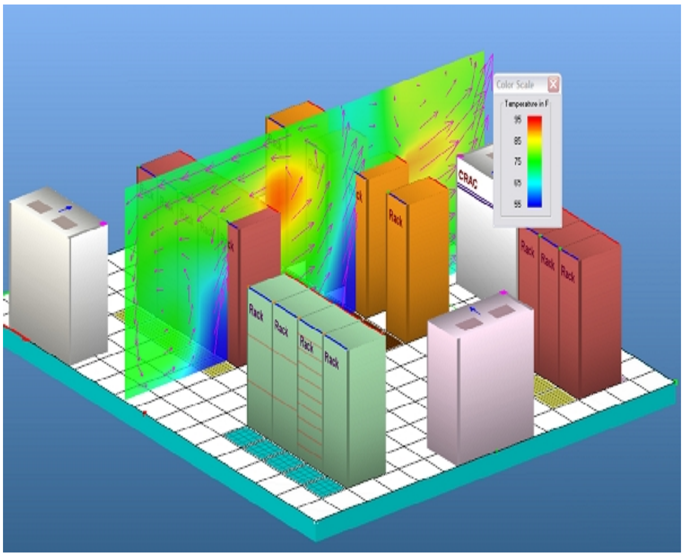

Figure 5. *Temperature distribution and flowpattern on a vertical plane*

4. CONVENTIONAL VS CFD IN DESIGNING DATA CENTERS

The current approach of achieving the desired cooling performance in a data center through trial and error is expensive and time consuming. **TileFlow** offers a comprehensive and cost-effective alternative involving computer simulation of the cooling performance of a data center. TileFlow constructs a computer model of the data center and uses the technique of Computational Fluid Dynamics (CFD) to calculate:

- Airflow pattern and pressure distribution under the raised floor.
- Distribution of airflow rates through perforated tiles and other openings on the raised floor.
- Airflow pattern and temperature distribution above the raised floor (computer room).

TileFlow can be used to model both raised-floor and non-raised floor data centers. It accounts for the effects of all important factors that influence the airflow patterns and temperature distribution, including

- Plenum and ceiling heights
- Location and flow rate of CRAC units
- Arrangement and open area of perforated tiles
- Cable openings on the floor
- Layout and heat load of server racks
- Under-floor obstructions
- Above-floor partitions and blockages
- Presence of overhead and rack-top cooling units

- Horizontal (in-row) coolers
- Overhead supply and exhaust vents
- Return ceiling plenum

9. CONVENTIONAL VS CFD IN DESIGNING DATA CENTERS

In 2007, a raised floor data center in the Intel plant in Penang, Malaysia, was about to be upgraded with new cooling system. This presented an ideal opportunity for using the Tileflow software to model the conceptual design of this room with proper cooling system. The raised floor area was approx. 8.5m x 10.4m, with 2 Computer Room Air-conditioning (CRAC) units placed into the room(as shown in the figure no.). This area was selected and it was sufficient to demonstrate the basis concepts. The raised floor was 18inch and the floor tiles were 610mmm (2ft) on a side. As shown in Fig no.6 a well developed modeling of PG12 Data Centers.

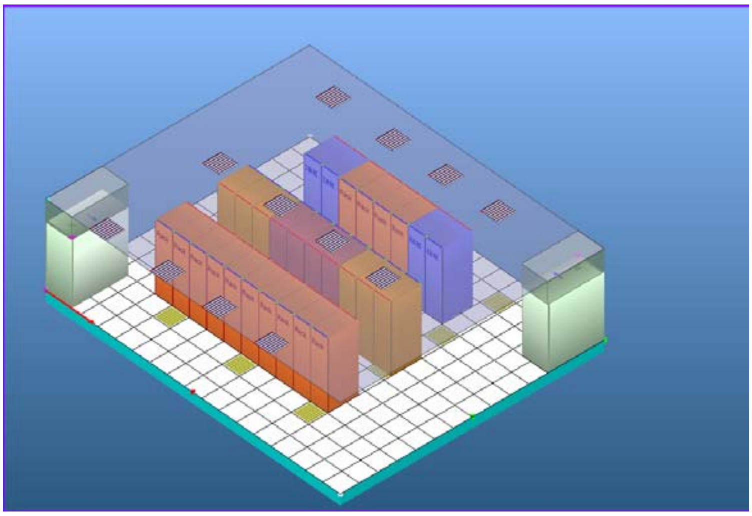

Figure 6. *A well developed modelling of Intel PG12 Data Center with 2 CRAC*

The positions of the CRAC units are shown in the top view included in Fig. no.6. The CRAC units were positioned such that the airstream exhausting from the two units were directed toward the pressurize air plenum, that is, the airstreams will discharge in a region between perforated tiles at the equipment racks. Figure 7; are displaying the experimental cooling plumes through the perforated tiles.

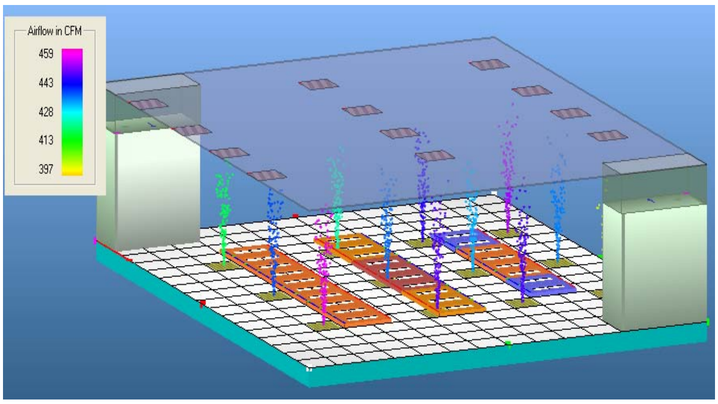

Figure 7. *Experimental setup display of cooling plumes through the perforated tiles*

The CRAC units were DB aire model DB20IC. These unit were approximately 20-ton units and provide airflow rates of 5000 cfm/min. After detailed examination of the CRAC units it was found that the outlet openings of the units were partially blocked by solid floor tiles. As a result, the actual flow rate delivered by the units was taken into account when performance the CFD calculations.

The rated opening area of the perforated tiles is 25%. Actual measurements, however, showed that the open area is only 19.5%.The flow resistance of the perforated tiles was measured on a flow bench and given. All nine perforated tiles arrangements considered in this study. Each tile arrangements were chosen such that it could potentially support arrays of rack aligned with the perforated tiles. The cold air from the perforated tiles washes the fronts of the racks, is drawn into the racks to cool the electronics, and is finally discharged from the rear of the racks to return to the inlets of the CRAC units above the ceiling.The airflow rates were measured with a calibrated Alnor balometer. This tool is capable of measuring flow rates from individual perforated tiles. The instrument was calibrated on a flow bench, and the collected data was suitably adjusted to account for the measurement errors. The total error in measurements, including instrument errors, is estimated to be within 5%.

10. CONVENTIONAL VS CFD IN DESIGNING DATA CENTERS

This configuration involves a 3 x 3 array of perforated tiles placed between two CRAC units. The flow rates of the tiles in the first two columns next to the CRACs unit are positive; that is, there is a forward flow is caused by the positive plenum pressure (pressure more than the ambient pressure above the raised floor) resulting from the supply air from the CRAC. The further tiles will received more air flow than compare to the ones nearer to the CRAC units. The slight dip for tiles in middle is caused by the presence of obstructions. Backflow of air is prevented with the help of the cardboard piece placed at the inlet of the unit to prevent this from happing. Figure 8: Experimental result display of cooling plumes through the perforated tiles.

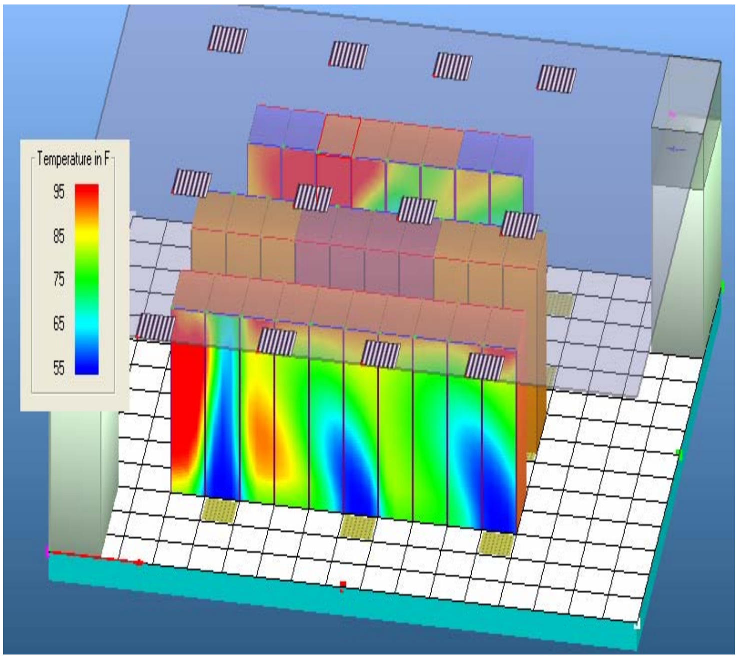

Figure 8. *Experimental result display of cooling plumes through the perforated tiles*

The results presented in the preceding sections clearly show that the airflow distribution, even for a simple configuration, is a complex function of the layout of the perforated tiles and the positions of the CRAC units. In actual Data centre, it will depend on the additional parameters related to the dimensions of the plenum, the open area of the perforated tiles, and the flow resistance/blockage of the obstructions like pipes, and cables in the plenum. A clear understanding of the relationship between the airflow distribution and the governing parameters is essential for achieving the desired airflow distribution. The most convenient approach for establishing this relationship is through mathematical (computational) modeling.

For a computational model to be useful as practical tool for designing new data centers and upgrading and retrofitting existing data centers, it must meet 2 criteria: (a) it should be east to use (it should not require specialized knowledge of CFD), and (b) it should produce results in short turnaround times. Off course, it should be produce results that agree well with the experimental data. At first glance, it appears that the prediction of the airflow distribution would require calculations of velocity and pressure distributions in the plenum and in the computer room above the raised floor. This approach would involve 3-D CFD model of the entire space.

The airflow rate through a perforated tile depends on the local pressure drop across the tile (pressure in the plenum minus pressure above the raised floor). The pressure variations above the raised floor are small compared to the pressure drop across the perforated tiles and can thus be ignored. That is, the pressure above the raised floor can be assumed uniform. As a result, the airflow distribution is governed primarily by the pressure distribution in the plenum and can, therefore, be obtained.

11. CONCLUSIONS

Air flow distribution is presented for a number of raised floor data center configurations. The key conclusions from this study are the airflow distribution is a strong function of the number of CRAC units in operation. An improved airflow distribution is achieved when both CRAC units discharge air in the same direction. The airflow distribution is more non-uniform when the CRAC units are oriented such that they discharge in opposing directions and their airstreams collide. The results from the CFD programs are in good agreement with the experimental data.

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ROOM AIR DISTRIBUTION OPTIMIZATION

Rawnee Ho[1] and Dr.M.R Ismail[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

rawnee@yahoo.com[1]; rodzi@usm.my[2]

ABSTRACT: Room air distribution and cooling load estimation for room/building are based on standard design approach (cooling estimation). The knowledge about the airflow distribution on a room/building is based on pattern inside the room/building need to be considered. Most room/building airflow distribution is not properly design, where the ventilation mixing plays a significance role toward thermal comfort. During Design stage, studying the airflow distribution and cooling load will enable us design a very efficient system. The case study has been carried at PBP Conference Room in University Science Malaysia, Penang. The outcome of the case studies shows the airflow distribution on a room/building play important key role room air distribution optimization study. The case studies based on recommended building heat gained (btu/hr) per sqft and understanding the airflow distribution pattern has proven to be useful for energy saving.

Keywords: definition, cassette type air-conditioner, air change rates per hours (ach), british thermal unit per hours (btu/hr), refrigeration tonne (rt), cubic feet per minute (cfm), airflow & room air distribution modelling.

1. INTRODUCTION

Characterizing how airflow is introduced to, flows through, and is measured from spaces is known as room air distribution. Uneven or short circuit airflows in a room/building will causing significant problem to cooling system. It will also affect the thermal comfort due to poor air distribution & etc. This call for consideration when designing the room air distribution prior to estimate the cooling load required. In proper airflows design will cause a problem on the room air mixing (cool and re-heat air).

2. VENTILATION MIXING SYSTEM

Mixing systems generally supply air such that the supply air mixes with the room air so that the mixed air is at the room design temperature and humidity. In cooling mode, the cool supply air, typically around 55°F (13°C) (saturated) at design conditions, exits an outlet at high velocity. The high velocity supply air stream causes turbulence causing the room air to mix with the supply air. Because the entire room is near-fully mixed, temperature variations are small while the contaminant concentration is fairly uniform throughout the entire room. On this case study, Cassette air conditioner turbo fan will create the high velocity supply air stream. Most often, the air outlets and inlets are placed in the ceiling. Supply diffusers in the ceiling are fed by fan coil units in the ceiling void or by air handling units in a remote plant room. The fan coil or air handling unit take in return air from the ceiling void and mix this with fresh air and cool, or heat it, as required to achieve the room design conditions. This arrangement is known as 'conventional room air distribution'.

The target of mixing ventilation applications is to diffuse the supply air draughlessly into the space so that the thermal conditions and eventual containment concentrations are uniform either in the entire space or in specific zone of the space (e.g. the occupied zone). The air flow rates typically defined using heat or humidity balance calculation, Alternatively, the air flow rate is determined on the basis of the dilution of emitted air contaminants in the space via air exchange. Naturally, the outdoor air ventilation specified in the building codes are respected. Ventilation efficiency is ensured by effective air diffusion through the occupied zone and avoid undesired stagnant zones and flow of the supply air into the exhaust.

3. DISPLACEMENT VENTILATION SYSTEM

Displacement ventilation systems supply air directly to the occupied zone. The air is supplied at low velocities to cause minimal induction and mixing. This system is used for ventilation and cooling of large high spaces, such as auditoria and atria, where energy can be saved if only the occupied zone is treated rather than trying to control the conditions in the entire space. The displacement outlets are usually located at or near the floor with the air supply designed so the air flows smoothly across the floor. Where there is a heat source (such as people, lighting, computers, electrical equipment, etc.) the air will rise, pulling the cool supply air up with it and moving contaminants and heat from the occupied zone to the return or exhaust grilles above. By doing so, the air quality in the occupied zone is generally superior to that achieved with mixing room air distribution. If air mixing is encouraged at the floor level, this type of floor-to-ceiling room air distribution is known as *underfloor air distribution*; if mixing is discouraged, it is *displacement*.

Displacement room airflow presents an opportunity to improve both the thermal comfort and indoor air quality (IAQ) of the occupied space. It also takes advantage of the difference in air density between an upper contaminated zone and a lower clean zone. Cool air is supplied at low velocity into the lower zone. Convection from heat sources creates vertical air motion into the upper zone where high level return inlets extract the air. In most cases these convection heat sources are also the contamination sources (e.g., people, equipment, or processes), thereby carrying the contaminants up to the upper zone, away from the occupants. Since the conditioned air is supplied directly into the occupied space, supply air temperatures must be higher than mixing systems (usually above 63 °F or 17 °C) to avoid cold draughts at the floor. By introducing the air at supply air temperatures close to the room temperature and low outlet velocity a high level of thermal comfort can be provided with displacement ventilation.

4. CASSETTE AIR-CONDITIONER VS CEILING DIFFUSER

The means of delivering the cool air from the air-conditioner unit is via air diffuser or air supply grille. The air diffuser or grille will uniformly distribute airflow from the ducted system. Whereas the cassette types of airflow, it is delivered with the help of auto air swing (30^{o}-60^{o}) angle. Both method of airflow distribution has it’s limitation, air diffuser and cassette types only can covered room or area within 10 feet radius. Cassette types airflow distribution pattern is more of active than passive (on diffuser). Cassette types auto swing louver will help the air distribution mixing effectively, than compare to air diffuser.

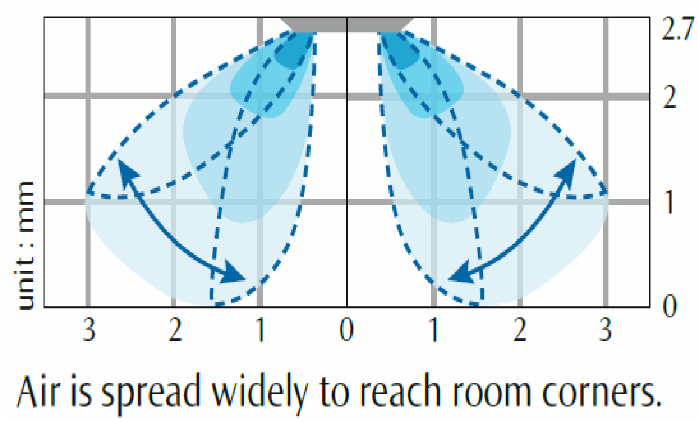

Figure 1. *Cassette types air-conditioner air flow distribution pattern in a room*

(Halton, 2009). Poor air distribution yields poor comfort, poor air quality, slow response system for the cool air to reach the designated area in the room/building. Diffuser selection needs to use mathematical modelling and most diffuser supplier will recommend the owner respectively.

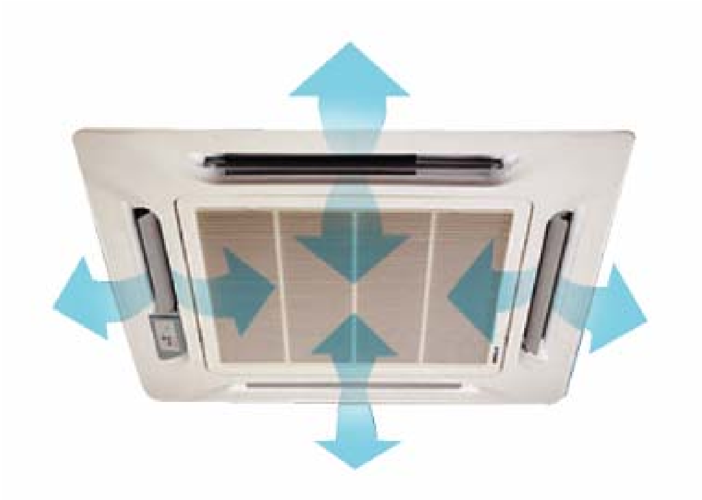

Figure 2. *Air flow supply direction of the cassette types air-conditioner*

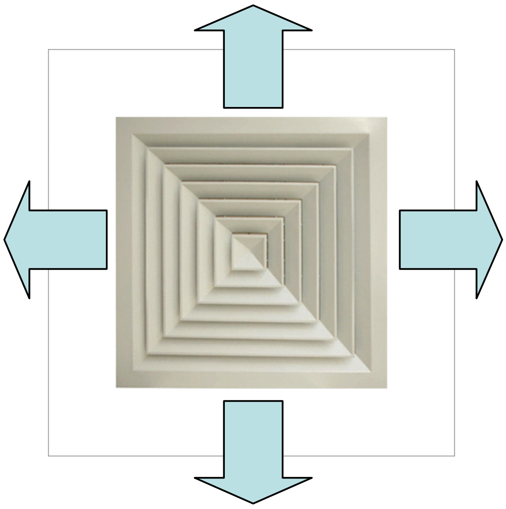

Figure 3. *Air flow supply direction of the ducted ceiling diffuser*

5. CASE STUDIES

The PBP Conference Hall (Dewan Persidangan PBP) at University Science Penang, Malaysia, was holding a post graduate seminar and the cooling system have been reported to be too cool. The room is full of students and in middle of conference room, it was not so cooled. This presented an ideal opportunity to study the air conditioning design for the PBP Conference Hall based on cooling load design and also room air distribution study.

Figure 4. *PBP Conference Building*

Figure 5. *PBP Conference room with student and lecturer*

The PBP Conference Hall is located at the 1st floor of the 2 storey building, 24ft x 46ft room (1,104 sqft), 10ft room height with false ceiling and is capable of holding 50 people. As shown in Figure 6 a well developed modeling of PG12 Data Centers.

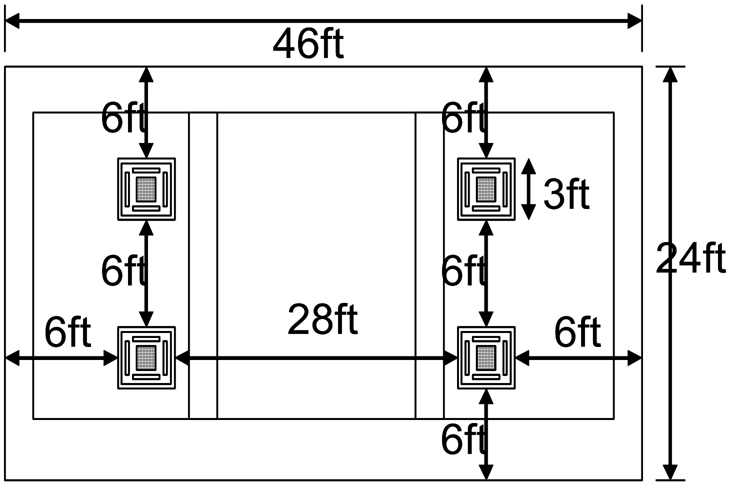

Figure 6. *Pejabat Pembangunang. PBP Conference room layout*

The PBP Conference Hall (Dewan Persidangan PBP) air conditioning system is 4 nos of YSL50A 50,000 Btu/hr cassette air conditioner (split unit) as shown in Figure 7. 4 nos Cassette Type indoor unit of 1029cfm turbo mixed fan each and total cooling capacity of 200,000 Btu/hr or 16.7 Refrigeration Tonne.

Figure 7. *Location of Cassette types air-conditioner*

6. RECOMMENDATION AND FINDING

6.1 Case Studies Finding

The PBP Conference Room cooling capacity is 200,000 Btu/hr or 4 nos of cassette type air-conditioner. Below is the estimated cooling load capacity and required air flow rate (CFM) study. The estimated cooling (Btu/hr) per sqft formula will be used to assist into studying the cooling load distribution Btu/hr per sqft. Required Air flow (CFM) = (Room Volume (Width x Length x Height) x Air Change Rates) / 60 second. Required Air flow (CFM) = 1,104 x 10 x 4 (recommend from BS5720)/60 = 736 cfm (Installed cassette type fan capacity 1029x4 = 4,116 cfm). Cooling Per sqft = Total Install cooling capacity (Btu/hr)/Area Sqft (A) ,Cooling Per sqft = 200,000 (Btu/hr)/1,104 (Sqft) = 181.2 Btu/hr per sqft . [5].Dr Rodzi. Recommended building heat gained Btu/hr per sqft = 60 Btu/hr per sqft. Airflow distribution inefficiency blocked by ceiling beam and distribution of room temperature differences of 2-3 deg C. The temperature recorded at the conference room was 21^{o}C (as shown in figure 8).

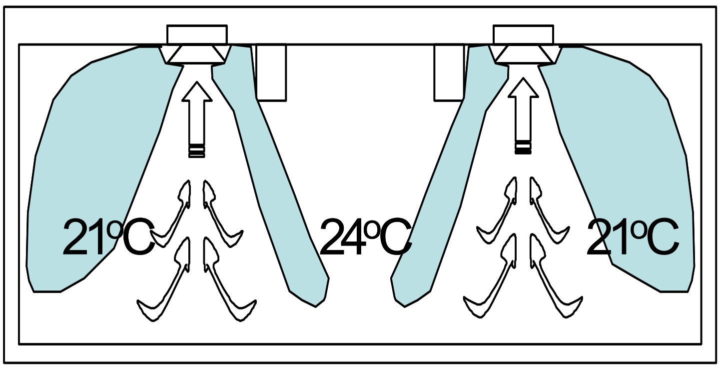

Figure 8. *Obstructed airflows distribution of the PBP Conference (in blue shade)*

Discomfort feeling has been introduced due to the extreme low temperature (21^{o}C) and also it was found the cassette types air swing is not on swing position, instead it was on fixed position. Such airflow distribution has causing short circuit and blocked by the ceiling beam. Top view showing the ceiling airflow distribution showing hot spot area, where the airflow is being blocked by the ceiling beam.

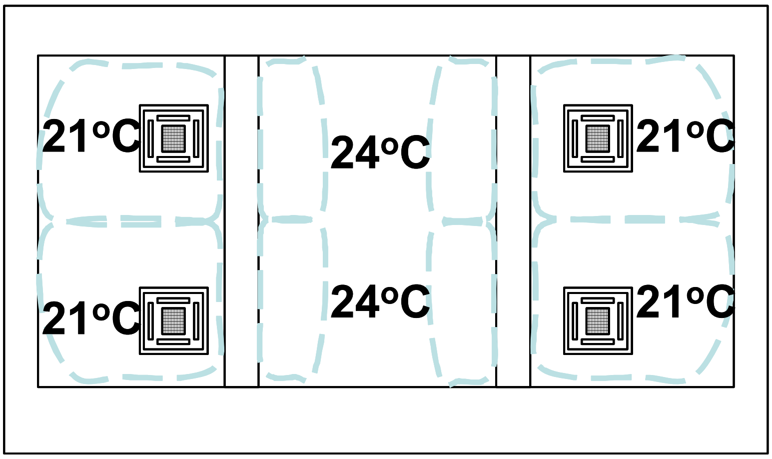

Figure 9. *Airflows distribution coverage and temperature reading taken*

6.2 Case Studies Recommendation

From the observation and finding, it is practical to consider the following recommendation to improve the airflow distribution and cooling load required. The required airflow pattern of the cassette types ceiling need to be re-look into and ensuring that the air swing is always on auto swing mode (this way it will help to circulate the airflow throughout the entire room. Total cooling load required will need to 60 Btu/hr per sqft, thus this will result into running at least 2 nos or maximum of 3 nos of cassette types air-conditioner during any even/conference. Also need to re-consider to relocate 1 nos of the cassette types air-conditioner to the middle section of the conference room (as shown figure 10). This will help to further evenly circulate the airflow distribution effectively. Thus will ensure proper energy saving and help to prolong the cassette types air-conditioner life.

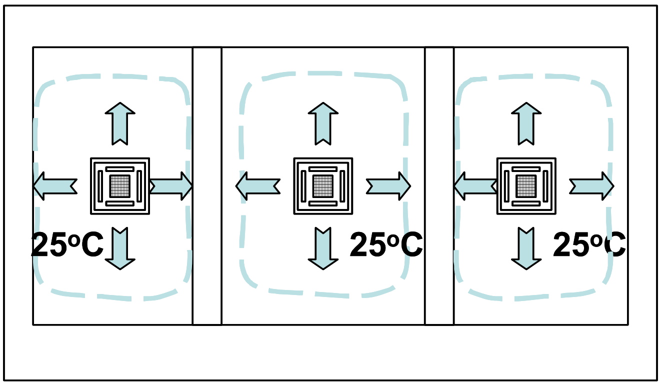

Figure 10. *Recommended location and temperature setting for the air-conditioner*

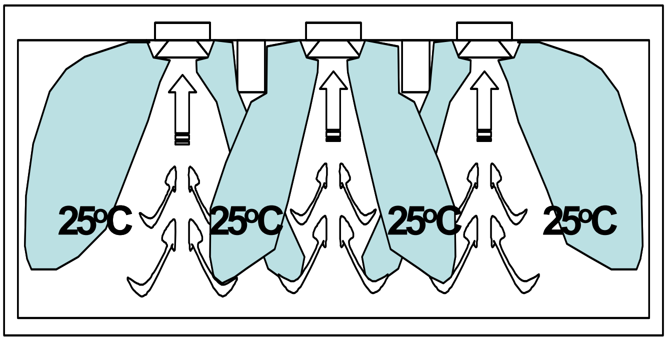

Figure 11. *Expected airflow distribution (in blue shade)*

To reset the room temperature to 25°C instead of 21°C for thermal comfort consideration and gradually increase the cooling with regard to the outdoor ambient temperature. Reduction on the cassette types air-conditioner fan speed can be reduced to 1,000cfm (each unit fan speed to be set at 300-400 cfm). Too much cfm will cause draft and unwanted noise pollution. The results presented in the preceding sections clearly show that the airflow distribution, even for a simple configuration (small area of 1,104 sqft), need much attention. Below the false ceiling area need to be clear of obstacle to avoid any obstruction on the airflow discharge from the air-conditioner. The locations of air-conditioner play an important role for airflow distribution. A clear understanding of the relationship between the airflow distribution and the governing parameters is essential for achieving the desired airflow distribution. The most convenient approach for establishing this relationship is through mathematical modeling (software simulation).

For a software mathematical modeling can be useful as practical tool for designing small or large areas, it must meet 2 criteria: (a) it should be easy to use and (b) it should produce results in short turnaround times. Off course, it should be produce results that agree well with the thermal comfort requirements.

7. CONCLUSIONS

Airflow distribution on a single zone building need to be considered during the designing stage and the ceiling airflow distribution play an important role to ensure the cool air is evenly distributed throughout the entire room. Small and single zone room/building has been a standard design practice to use rule of thumb to estimate the cooling requirement i.e. Btu/hr per sqft, with the help of recommended building heat gained Btu/hr chart, it will further simplified the estimation/design consideration of the cooling load, especially looking at Malaysia weather climate.

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PERFORMANCE OF SAND CEMENT BRICKS INCORPORATING KENAF POWDER AND RICE HUSK ASH

Kartini, K.[1], Ahmad Farhan, H.[2], and Nor Azlina, U.[3]

[1,2,3]Faculty of Civil Engineering, Universiti Teknologi MARA, MALAYSIA.

[1]ce_kartini2002@yahoo.com

ABSTRACT: Green and lightweight material is turn out to be the interest as a global subject concern. A new invention about lightweight brick production is needed and improves our understanding about the composite material especially for building materials. Bricks are one of oldest building material used as construction materials that can be created and used in a long run. Recently, new approaches of design method were explored and differentiation of ordinary brick is needed to be investigated. Composite brick have developed quickly and became progressively more popular in building material. In addition, new innovative idea has been discovered to improve the capability of latest composite building materials in producing lightweight brick. Brick could carry various loading depend on their material mixture and strength. The composition of brick is highly constituent by sand and cement. It is very significant to find the alternative towards sustainable and environmental friendly in order to produce a lightweight brick that could reduce the overall construction costs as well, since bricks could reduce the load of structure. Two materials are identified in producing lightweight brick which are kenaf powder (KP) as additive materials for concrete mixes and rice husk ash (RHA) to partially replaced cement. The combination of such natural resources with cement mortar is expected to cause higher impact on compressive strength and improve the performance of the composite brick as additive materials. In the era of great information transfer by new technology in cement mortar and concrete material, the exploitation of KP and RHA as replacement or addition materials may helps to increase the durability, and take full advantage of composite materials especially in terms of strength. Therefore, demands to use natural material in lightweight bricks manufacturing are certainly needed.

Keywords: ***sand cement brick, kenaf powder, rice husk ash, compressive strength, durability***

1. INTRODUCTION

In the last two decade, the demand on building materials industry had raised up due to the increasing population which causes an unceasing shortage of building materials (Algin and Turgut, 2006). Thus, adaptation of the industrial and agricultural wastes to valuable building and construction materials especially for brick material are welcomed. Bricks are one of oldest building material used as construction materials that can be created and used in a long period of time. Bricks as an in-filled material in wall construction should be of lightweight and durable. Studies on the development of bricks are increasing and had move toward lighter bricks with increase in the thermal insulation (Chiang *et al.*, 2009). The composition of bricks is highly constituent of sand and cement, and in producing a brick, these materials are very expensive and the sources availability is depleting. Therefore, ways in substituting these natural raw materials with recycles agricultural waste need to be exploited. Besides, the production of OPC will also result in the emission of green house gases especially CO_2, which is accounting for approximately 7% to 8% of CO_2 globally (Bronzeoak, 2003). It was reported that about 1 tonne of CO_2 is released per tonne of clinker produced, which translates to about 0.95 to 0.97 tonne of CO_2 per tonne of Portland cement produced (Glasser, 1998). On top of that, it requires high-energy consumption to produce, *i.e.* 1 tonne of cement requires energy of 100-150 kWT. Till to date, the utilization of industrial waste by-products as well as the agricultural residue such saw dust ash (Kartini, 2001), empty fruit bunch ash (Amizan and Kartini, 2009), rice husk ash (Kartini, 2009) and quarry dust fine powder (Norhana *et al.*, 2010) have been carried out in finding the outstanding material as partial replacement or additive material for cement. Hence, it is very significant to use cement together with renewable materials which are sustainable and environmental friendly in order to produce a lightweight brick.

2. EXPERIMENTAL WORK

2.1 MATERIAL USED

The rice husk was burnt in a Ferro cement furnace to produce amorphous rice husk ash (RHA). After burning and left for 24 hours for it to further burning, the burnt ashes were left to cool inside the furnace for another 24 hours before taken out for grinding in a Los Angeles (LA) machine. To produce fine RHA, about 5 kg of RHA were placed in the LA machine, with 40 nos. of steel rod in it and grinded it for 5000 revolutions. Another material used is the kenaf (*Hibiscus cannabinus L.*) powder which was obtained from Everise Crimson (M) Sdn Bhd. This kenaf powder was added in the sand cement mixes in different proportion as additive material. The standard average grain size of kenaf powder particle was 3 mm and with a density of about 123 ± 2 kg/m^3 (Sarah and Mark, 2001). The other materials also used in this study were Ordinary Portland Cement (OPC) and river sand of 5 mm maximum size as fine aggregates.

2.2 TEST CONDUCTED

2.2.1 *COMPRESSIVE STRENGTH*

For hardened concrete, the brick units were subjected to compressive strength after 28 and 60 days of air curing and the test was performed in accordance to BS EN 772-1:2000 was measured on bed face of the brick.

2.2.2 WATER ABSORPTION

Water absorption is subjective by the capillary holes present in concrete surface. Water absorption test is conducted to determine the rate of absorption of water into the bricks. The test was conducted with accordance to BS EN 12390-8:2009. The brick units were dried in oven for 72 hours at temperature condition 105 ± 5°C. Then, the brick was placed in the closed container for 24 hours. Subsequently, the initial brick unit is first weighed before it is completely immersed in water. Depth of 25±5 mm of water over the top of the brick unit was maintained throughout the test. The brick units were removed and weighed at every 30 minutes interval for 4 hours.

2.2.3 SAMPLE PREPARATION

Ten (10) various mixes of cement mortar (excluding specimen control) were prepared to cast the brick specimens. The design of the mortar mix was based on 1:3 mixes with 0.5 water cement (w/c) ratio. For the KP brick, the mix proportion was designed based on different level of kenaf powder addition, *i.e.* 0%, 3%, 5%, 7%, 10%, 15%, while for RHA brick the design is based on the percentages of replacement of OPC with RHA, *i.e.* 0%, 15%, 20%, 25%, 30%, 40%. The details of the design mixes were as shown in Table 1

Table 1. *Mixture Proportion for Kenaf Powder and RHA Concrete Brick*

Mixture Designation	Constituent Materials (kg/m^3)				
	Cement	Water	River Sand	Kenaf Powder	RHA
Control-0	513	256	1539	0	0
KP-3	513	256	1539	69.2	-
KP-5	513	256	1539	115.4	-
KP-7	513	256	1539	161.6	-
KP-10	513	256	1539	230.8	-
KP-15	513	256	1539	346.2	-
RHA-15	436	256	1539	-	77
RHA-20	410	256	1539	-	103
RHA-25	385	256	1539	-	128
RHA-30	359	256	1539	-	154
RHA-40	308	256	1539	-	205

3. RESULT AND DISCUSSION

3.1 PROPERTIES OF MATERIALS

A typical chemical composition of the RHA obtained after burning and grinding is shown in the Table 2. From the table, it can be seen that SiO_2 for RHA is 96.7 % and LOI is 4.81 %. These show that the ashes are of high content of silica and of good quality. It was investigated by Kartini (2009) that the OPC contains calcium oxide and silicon dioxide of 64.6% and 21.4% respectively, while, RHA is 0.49 and 96.7 respectively. The RHA used satisfy the ASTM C618 (2003) requirement for SiO_2, which should be minimum 70% and loss of ignition of a maximum 6%.

Table 2. *Chemical Composition (%) of OPC and RHA (Kartini, 2009)*

Chemical composition (%)	RHA	OPC
Silicon dioxide (SiO_2)	96.7	21.4
Aluminium oxide (Al_2O_3)	1.01	5.6
Ferric oxide (Fe_2O_3)	0.05	3.4
Calcium oxide (CaO)	0.49	64.6
Magnesium oxide (MgO)	0.19	2.1
Sodium oxide (Na_2O)	0.26	0.1
Potassium oxide (K_2O)	0.91	-
Phosphorous oxide (P_2O_5)	0.01	-
Titanium oxide (TiO_2)	0.16	-
Sulphur trioxide (SO_3)	-	2.1
Loss on ignition	4.81	0.64

3.2 WEIGHT OF BRICKS

Figure 1 shows the graph of weight against age of curing of RHA brick. From the figure, it can be seen that the weight of the RHA bricks are decreasing from 15% up to 40% replacement, however, the 25% up to 40% replacement are more pronounced in terms of its reduction compared to the control brick, *i.e*. with a relative difference of 0.78% to 5.6% and 1.3% to 11.6% respectively taken at 28 days and 60 days of air drying. This means that for bricks to be used as an in-filled in wall construction, it is more that enough to have a one made of a lighter material compared to the normal sand cement brick. It is also to be noted that the specific gravity of Portland cement is 3.11, while for RHA is 2.1 (Kartini, 2009), thus this reduction in the specific gravity of RHA indicates lower weight when it is used compared to Portland cement. Hence, this indicates that RHA might be an option to produce a lightweight bricks and reduce the utilisation of cement in brick manufacturing. Furthermore, reduction of cement used will considered environment sustainability towards less carbon dioxide (CO_2) emission.

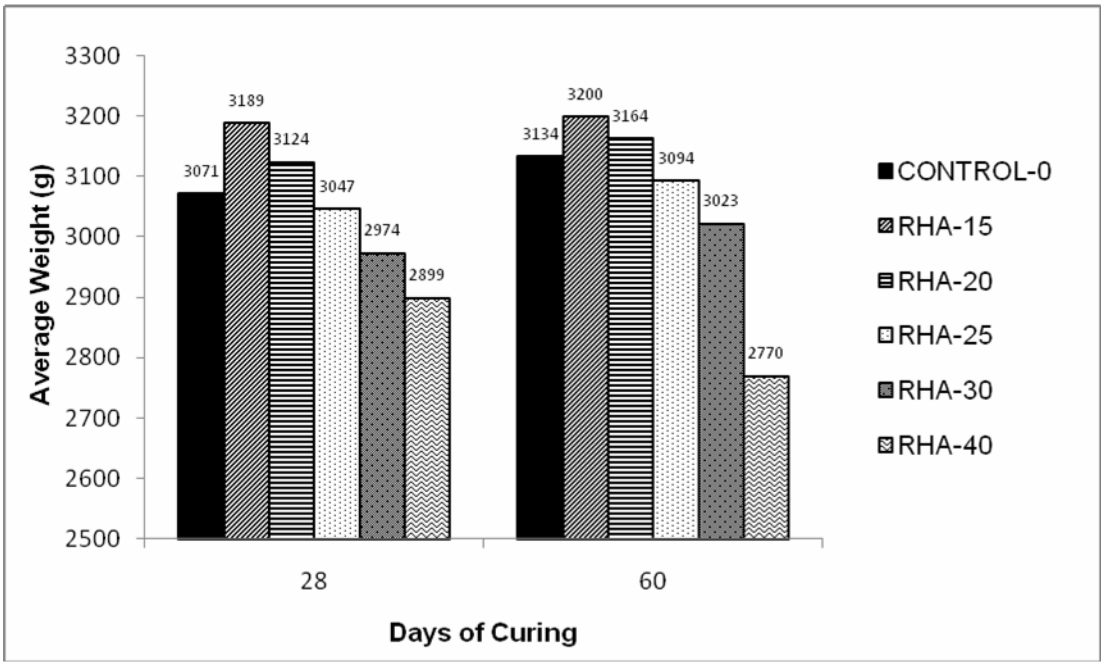

Figure 1. *Weight Measurement of RHA Bricks*

Figure 2 shows the graph of weight against age of curing of KP brick. From the figure, it can be seen that the weight of the KP bricks reduces as the amount of addition of kenaf powder in the mixes increases. The percentages of reduction in weight compared to control brick (sand cement brick) are from 4.9% up to 20.5% for addition of 3% up to 15% taken at 28 days. The amounts of reduction are further increases as the period of air drying increases (60 days). This indicates that kenaf powder might also be an option to produce a lightweight bricks.

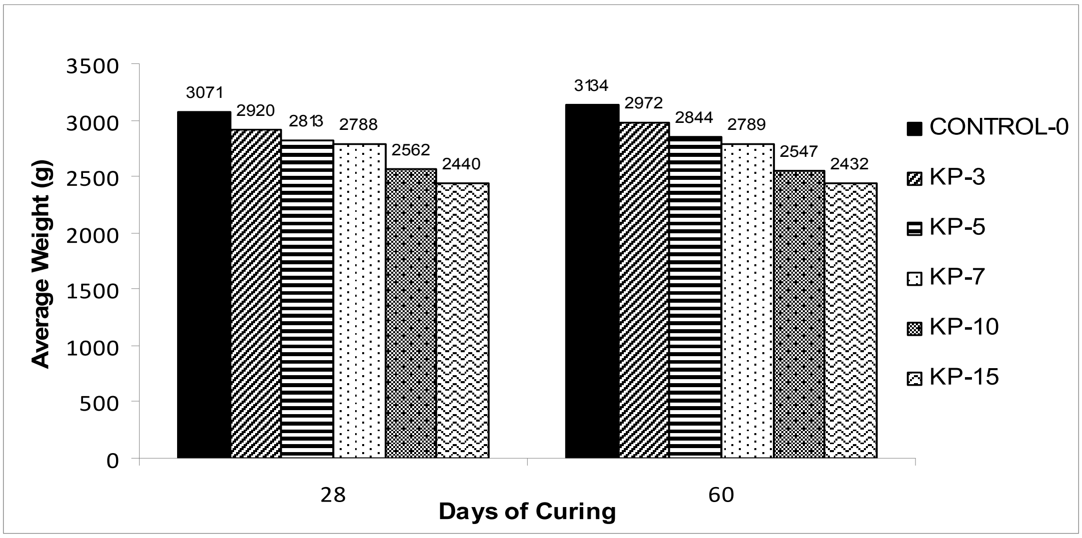

Figure 2. *Weight Measurement of KP Bricks*

3.3 COMPRESSIVE STRENGTH

3.3.1 RICE HUSK ASH (RHA) BRICKS

The results of the compressive strength for the RHA bricks taken up to 60 days air drying is presented in Figure 3. From the bar chart presented, it is clearly seen that at 28 days of air drying it shows that as the percentage replacement of OPC with RHA in the mortar mix increases, the strength increases, however beyond 30% replacement, the strength starts reducing. Comparing with the control sand cement brick, it seems that at 20%, 25% and 30% replacement the strength increases by about 10% -

27%. Comparing with the standard available (BS EN 771-3:2003), these RHA bricks can be classified under the category of strength 20 N/mm^2, and as an in-filled in wall construction, these bricks able to fulfil it purpose. This increase in strength might be due to reduced porosity, reduced $Ca(OH)_2$, and reduced width of the interfacial zone between the paste and the fine aggregate as reported by Zhang *et al.* (1996), however further increase resulted in reduction in compressive strength (Cook and Suwanvitaya, 1981; Kartini, 2009). Rashid *et al*. (2010) in their studies claimed that the incorporation of RHA produces the filler effect due to its fine particle size.

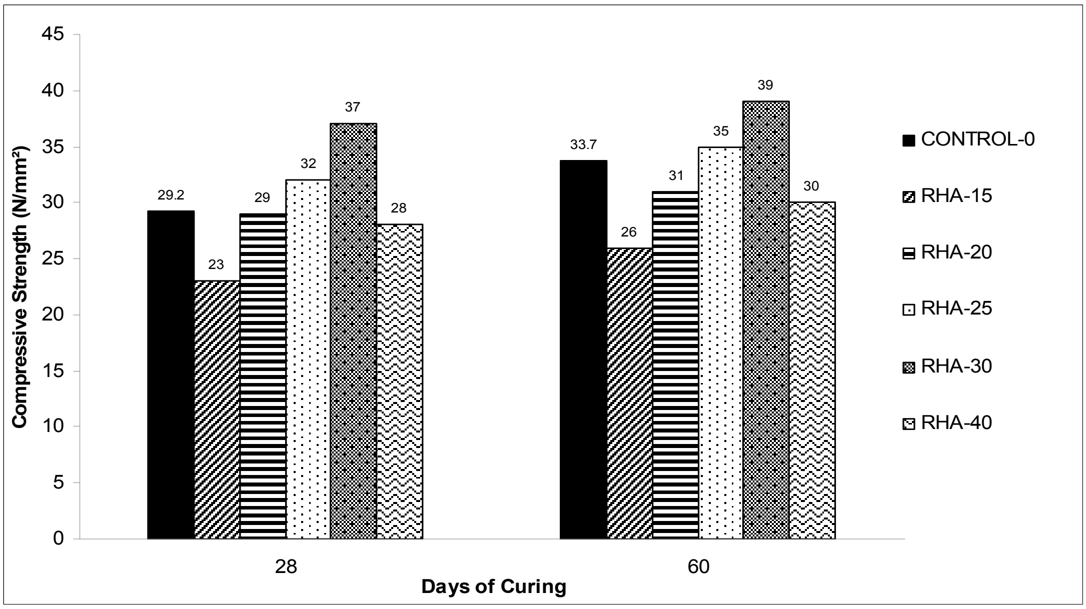

Figure 3. *Compressive Strength of RHA Brick*

3.3.2 KENAF POWDER (KP) BRICKS

Figure 4 shows the results of compressive strength of Kenaf Powder brick with respect to the age of air curing. From the figure, it is clearly seen that the strength reduces as the percentage of kenaf powder addition in the mix increases, and the strength are all lower than the control sand cement brick for all the ages. However, for it to be used as an in-filled in the wall construction, addition up to 7% is still viable. Comparing with the standard available (BS EN 771-3:2003), these kenaf powder bricks can be classified under the category of strength 20 N/mm^2, and used as in-filled in wall construction, The possible reason why the strength reduces might be because firstly, as the kenaf powder used as an addition in the mix, therefore resulted in not enough cementitious material to hold the other constituent materials together, hence resulted in lower bond between them. Addition of kenaf powder in the mix resulted in dry mixture as this kenaf powder is absorptive in nature, thus absorbed water from the mix and made the mix dry.

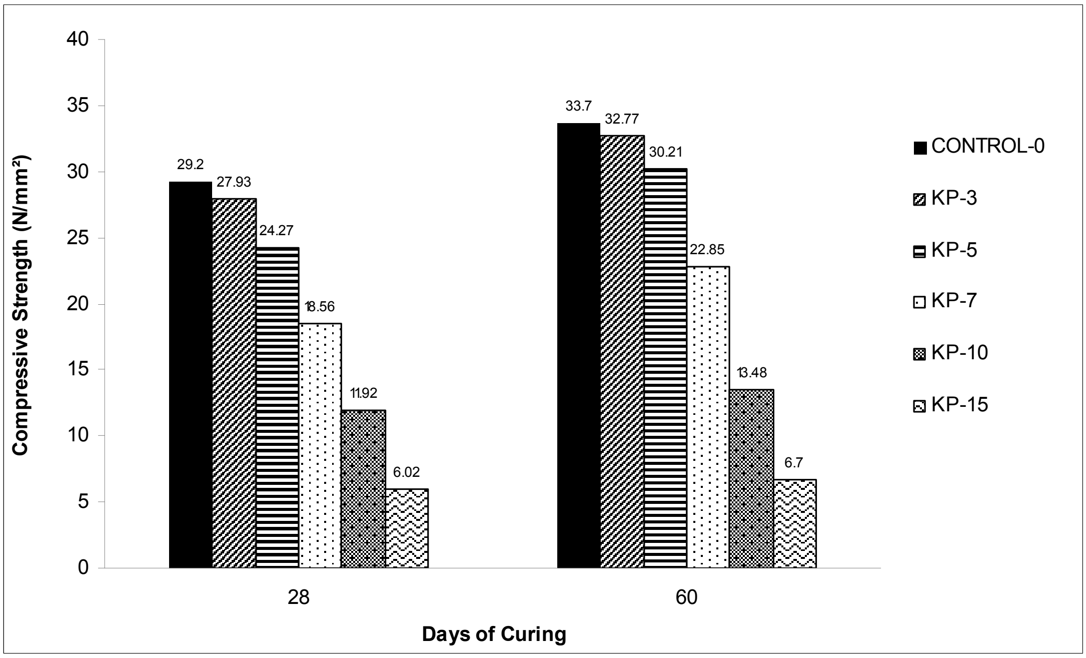

Figure 4. *Compressive Strength of KP Brick*

3.4 WATER ABSORPTION

3.4.1 RICE HUSK ASH (RHA) BRICKS

Figure 5 shows the result of the water absorption for the RHA bricks taken up to 60 days. It can be seen that increase the percentage of replacement of OPC with RHA in the mixes resulted in higher percentage of water absorption. These are more pronounced for higher replacement level. However, the amount of absorption is not greater than 7.5% as stated in BS EN 771-3:2003. The result of this study in terms of water absorption contradicts the findings of other researcher (Manmohan and Mehta, 1981; Sugita *et al.*, 1997; Kartini, 2009) in which they stated that RHA replacement in the mix resulted in pore refinement thus reducing the permeability or absorption of the mortar. It can also be seen that the amount of water absorption reduces with time in which in more matured mortar bricks the amount of water absorbed is less. This can be due to the pore structure in young mortar is coarser and with increasing hydration it becomes refined as stated by Saricimen *et al*. (2000) and Saraswathy and.Song (2005).

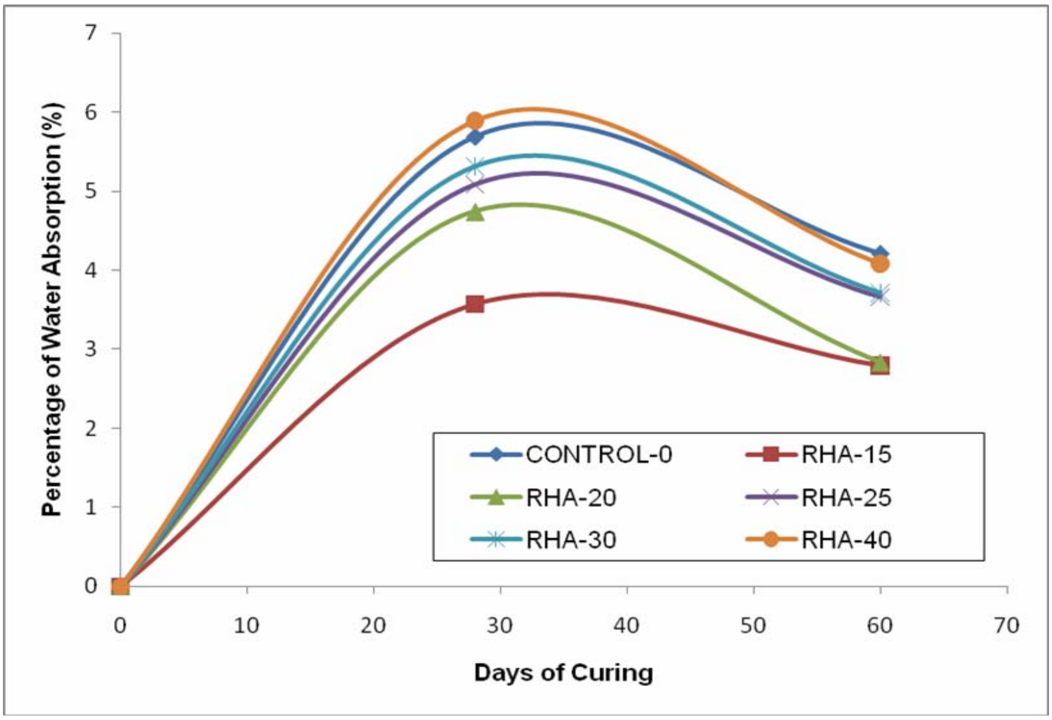

Figure 5. *Water Absorption for RHA Brick*

3.4.2 KENAF POWDER (KP) BRICKS

Figure 6 show that the percentages of water absorption for kenaf powder bricks with respect to time. From the figure it, can be seen that addition of kenaf powder in the mix resulted in increases in the degree of absorption compared to the control sand cement brick. However, at 3% addition of kenaf powder in the mix the absorption is lower than the control. According to BS EN 771-3:2003, the addition of up to 7% in the mortar mix still below the requirement of not greater than 7.5% absorption.

It can also be seen that increased in the period of curing resulted in reduction in absorption. This is due to pore structure in young mortar is coarser and with increasing hydration it becomes refined as mentioned earlier.

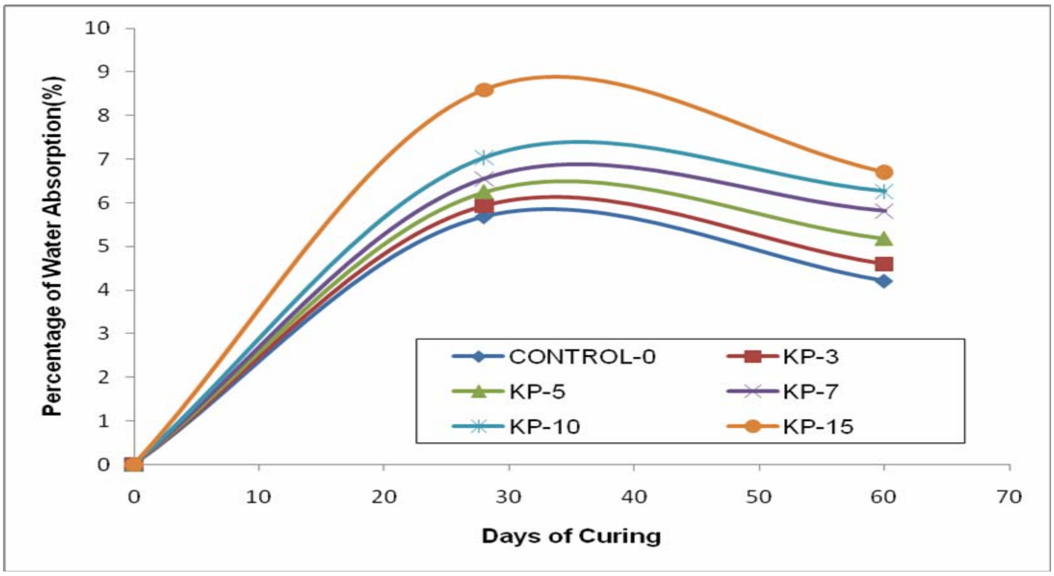

Figure 6. *Water Absorption for KP Brick*

4. CONCLUSIONS

a. The weight of RHA and KP bricks decreasing due to the increasing percentage of the replacement of OPC with the RHA and addition of kenaf powder into the mixes. This indicates that these two materials might be a good option to produce lightweight building materials.

b. Replacement of OPC with RHA up to 30% in the mixes resulted in increased in strength of the brick. The values of the compressive strength obtained are above the category of 20 N/mm^2 of sand cement brick for in-filled in the wall construction.

c. The addition of kenaf powder in the mixes decreased the compressive strength of brick. However, the 5% addition in the mix fulfilled the requirement of sand cement brick of the 20 N/mm^2 strength categories.

d. In terms of water absorption, replacement of up to 40% of OPC with RHA in RHA brick and addition of up to 7% of kenaf powder in KP brick still below the requirement of the standard of 7.5% absorption.

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TRANSPORTATION STRATEGIES FOR URBAN SUSTAINABLE DEVELOPMENT

Mojtaba Zourbakhsh
Islamic Azad University, Estahban Branch, Iran
m_zourbakhsh@iauestahban.ac.ir

ABSTRACT: Traditional transport planning aims to improve mobility, especially for vehicles, and may fail to adequately consider wider impacts. In the past, planning and implementation of such strategies has been slow and spotty, deterred by the complexities of the underlying issues along with uncertainties about the magnitude and timing of impacts, the efficacy of available courses of action, and the consequences of action or inaction. Recently, however, a new interest in actively pursuing these strategies has emerged. Concerns about environmental quality, social equity, economic vitality, and the threat of climate change have converged to produce a growing interest in the concept of sustainable development. Efforts are being made all over the world to increase the sustainability of development patterns. In nations with more advanced economies, particular attention is being paid to the critical roles played by transportation and activity systems. The social costs of transport include road crashes, air pollution, physical inactivity, time taken away from the family while commuting and vulnerability to fuel price increases. Many of these negative impacts fall disproportionately on those social groups who are also least likely to own and drive cars. Traffic congestion imposes economic costs by wasting people's time and by slowing the delivery of goods and services. Strategies for increasing transportation sustainability include demand management, managing traffic congestion, pricing policies, vehicle technology improvements, clean fuels and transportation planning. Sustainable transport systems make a positive contribution to the environmental, social and economic sustainability of the communities they serve. Transport systems exist to provide social and economic connections, and people quickly take up the opportunities offered by increased mobility. This paper reviews current thinking about sustainable transportation and strategies of transportation for sustainability.

Keywords: Transportation strategies, Sustainable transport, Urban sustainable development

1. INTRODUCTION

In developed nations,special attention is being given to the sustainability of current and emerging land use and transportation patterns.this focus reflects both the significant impacts that current patterns of transportation have on the environment and the complex interactions between transportation,land use ,and activity systems.In this context ,sustainable transportation is seen as transpotation that meets mobility needs while also preserving and enhancing human and ecosystem health,economic progress ,and social justice now and for the future.Planning for sustainable development aims to attain all three objectives simultaneously and in a just manner,considering access as well as mobility in the process.Today,sustainable development is widely viewed as development that improves the standard of living and quality of life,while at the same time protecting and enhancing the natural environment and honoring local culture and history.Efforts are being made all over the world to increase the sustainability of development patterns.Concerns about environmental quality ,social equity ,economic vitality,and the threat of climate change have converged to produce a growing interest in the concept of sustainable development.one definition of sustainable development was put forth in 1992 by the Brundtland Commission:”Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”Co2 reduction ,as called for in the Kyoto Protocol and other agreements,is an important objective;however,sustainability has quickly been transformed into a much broader concept having economic ,social,and environmental dimensions. On the other hand concerned with urban space, there are example : Urban population expanding at more than 6 percent per year in many developing countries. Traffic Congestion and Air Pollution is on the rise in mega- cities. Pedestrians and Non-

Motorized Transport (NMT) are poorly served by the cities transport systems.so the number of mega-cities (with over 10 million inhabitants) expected to double within a generation.

2. URBAN SUSTAINABLE DEVELOPMENT AND TRANSPORTATION STRATEGIES

A variety of strategies have been identifiied for potentially increasing transportation sustainability,including demand management,operation management,pricing policies,vehicle technology improvements,clean fules,and integratedland use and transportation planning.The strategies are grouped into several categories based on the component of the transport systems addressed:vehicles,guideways and operations,and demand.

2.1 VEHICLE AND FUEL TECHNOLOGICAL CHANGES

This part attention to reducing the bad environmental impact that is caused by vehicle engines.In the short run,this would most likely focous on strategies for improving the efficiency vehicles inorder to reduce emissions and greenhouse gases.In the longer run,the introduction of new vehicle technologies and new fuels might be an option and could lead to quieter,safer and cleaner vehicles.In either time frame,a variety of specific steps could be considered.For example,changes could be induced through consumer (demand-side)incentives or disincentives,through incentives offerd to vehicle producers(e.g.,tax credits),or through direct regulation of vehicle manufacturers(e.g.,saftey standards).

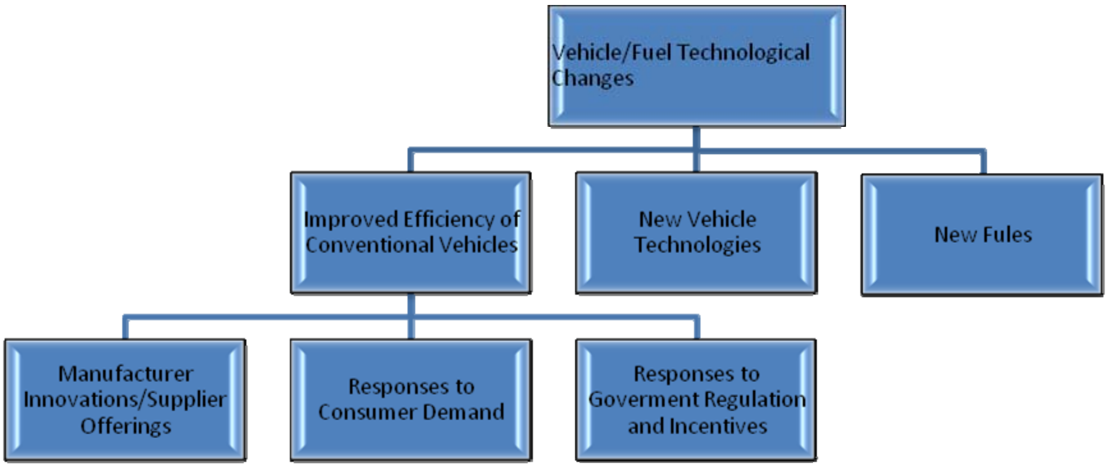

***Figure1**. Measures to Improve The Quality Of Vehicles And Fuel*

2.2 ROAD/VEHICLE OPERATIONS IMPROVEMENTS

A second category of strategies involves improvements to the roadways and vehicle operations.Conventional traffic flow improvements such as traffic signal timing,ramp metering,flow metering,and bottleneck removal all have the potential to cut energy use,reduce greenhouse gasemissions,and lower noise levels somewhat by smoothing the flow of traffic and reducing stop-and-go driving. Driver education could reduce emisssions by training drivers to avoid heavy accelerations and decelerations and to be mindful of the fuel consequences of high speeds.Scheduling trips outside of peak periods could reduce congestion and thereby cut a major source of economic loss.Improved methods of accidents/incident management and improved logistics and fleet management,both relying increasingly on advanced technologies for vehicle location and communication,also have substantial promise for increased efficiency of operations.Information technology-enhanced routing and scheduling can reduce the fuel needed for transport of both passengers and freight.Technological innovations currently under development offer the potential for significantly larger gains.These include the more advanced aspects of intelligent transportation system improvement such as smart highways and smart vehicles.

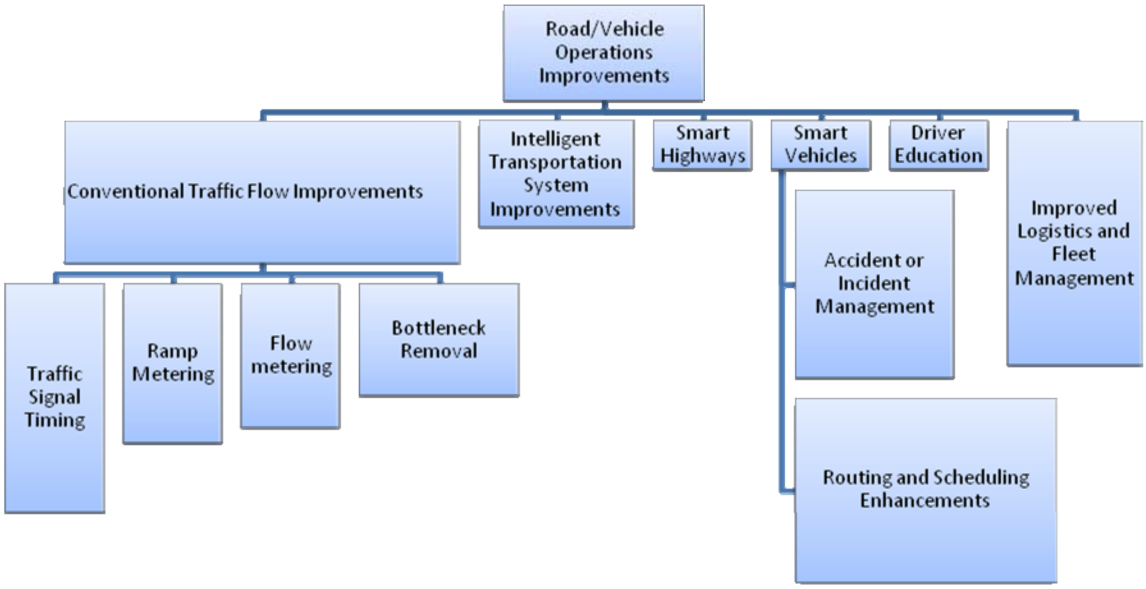

Figure 2. *Road/Vehicle Operations Improvements*

2.3 DEMAND MANAGEMENT

Demand management is a third category of strategies for managing the transportation system. Several subcategories of demand management are in use.Modal substitution, telecommunications substitution,pricing,and land use strategies all can be thought of as forms of demand management. Modal substitution means,for example, replacing car trips with transit,paratransit, ridesharing,biking and air freight.This can be accomplished by providing better modal option(offering services and improving their quality in order to attract travel to alternative modes)or through incentives for the use of the alternative modes(e.g.,subsidies for users of preferred modes). Regulatory requirements(trip reduction ordinances requiring employers to obtain commute mode shares of no more than 50% by drive-alone ,for example)are also a possible way to induce modal substitution.Telecommunications substitutions for travel also can be considered a form of demand management.

Telecommuting, teleshopping, teleconferencing, and distance learning are varieties of telecommunications substitutes for travel.Pricing incentives and disincentives could be used in the short run to reduce demand and encourage the use of alternative modes or the substitution of telecommunications for travel.In the longer run,vehicle technology improvements would likely be induced by the higher prices.Gas tax increases are the pricing strategy most commonly in many countries.Fees and taxes that affect vehicle ownership,such as sales taxes and registration fees,also are common.Variations that base taxes and fees on fuel efficiency, emissions, and expected vehicle life could specifically target the reduction of greenhouse gases as “fee-bate”variations offering tax reductions for efficient,low emissions vehicles along with surcharges for high emitters.Or pricing strategies could base emissions or fuel surcharges on measured or estimated use.Finally ,rather than use pricing to restrain emisssions directly,pricing could take the form of subsidies for preferred modes or for telecommunications

substitutes. Land and urban development strategies alter demand by reducing trip length(by providing a choice of close-by destinations)or by making alternative to the auto more competitive and cost-effective.(These stategies also may reduce emissions associated with building heating and cooling,service provision,etc.)

For example,compact development,mixed use development,and higher development densities can reduce trip lenghts and make transit,pedestrian,and bike use practical and affordable.In some cases,compact development also may facilitate better management of urban freight transport(shipment consolidation,delivery scheduling,etc.)

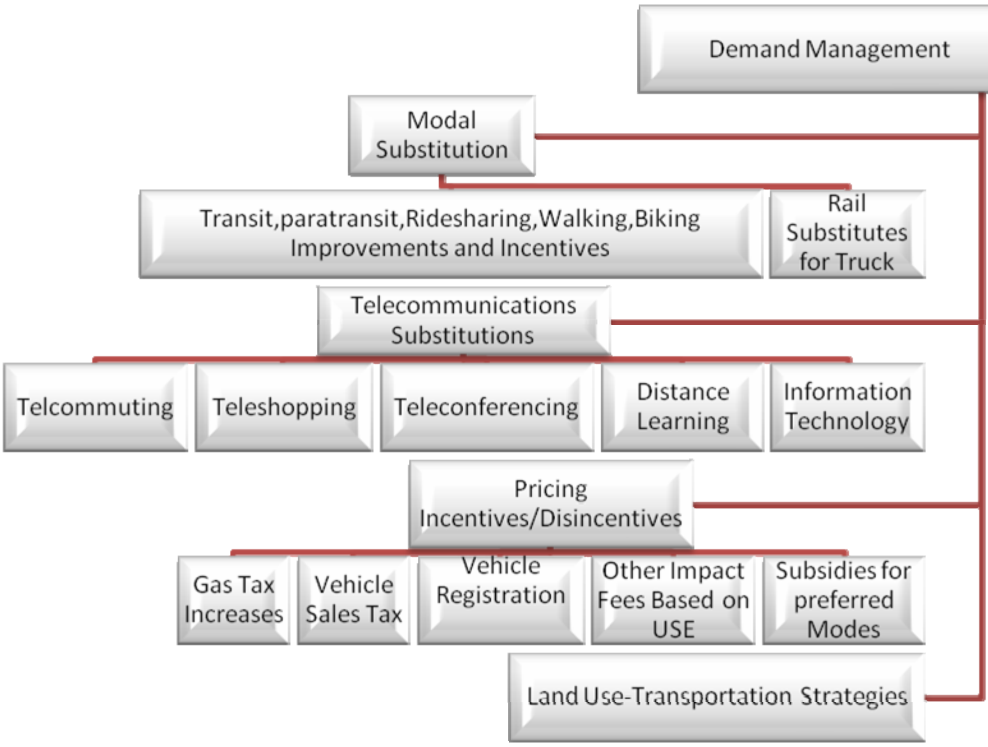

Figure3. *Demand Management Improvements*

2.3.1 URBAN PUBLIC TRANSPORT SERVICES AND INCREASING THE DESIRABILITY

Today, based on patterns of urban sustainable development according to motivate and encourage citizens to use public transport services. but should be considered if the desirability and efficiency are not required, it cause people to use personal vehicles as well as single-passenger trips will increase in the city area . Therefore, measures should be conducted according to the equations and strategies of transportation for encouraging people to use public transport services.

2.3.1.1 IMPROVING THE OPERATIONAL EFFICIENCY OF PUBLIC PASSENGER TRANSPORT

Important in urban bus transport services is primarily responsible for this transportation system demand in the short and medium haul range is remarkable and desirable methods of service delivery is considered upgrading this system is as follows:

- Bus lanes and automatic priority at intersection help.
- Exclusive bus-ways perform nearly equivalent to rail-based systems at much lower cost (except in very high traffic volume corridors).

2.3.1.2 IMPROVING THE OPERATIONAL EFFICIENCY OF MASS TRANSIT SYSTEMS

Major contribution to mass transit urban spatial displacement of long-haul is responsible, and due to high capacity system can meet the bulk of the convention is the urban demand. Strategies to improve the performance of this system are as follows:

- Investment decisions based on comparative analysis of strategic objectives, technological alternatives, and socio-economic and financial implications—not on short-term political or commercial opportunism.
- Rail-based mass transit systems have a role only in very large cities, as these are less congesting and serve the peripherally located in work journeys.

2.3.1.3 Improving the Operational Efficiency of *Non Motorized Transport*(NMT)

The role of urban design should always non-motorized transport systems, special attention is because the most appropriate manner consistent with the transportation policies of urban sustainable development. therefore recommended the following solutions:

- Due Recognition in Transport Planning, including road design
- Provision for the rights and responsibilities of pedestrians and bicyclists in traffic law
- National Strategy on NMT
- Provide Separate Infrastructure where appropriate.

2.3.1.4 The Role of the private sector in improving efficiency of Transport

Private sector has significant capabilities in most transportation projects could assist the government. Also, professional and operational potential of the private sector as the best in ensuring the success of projects will be considered under items proposed to strengthen this section is expressed as :

- In cases of very high demand for faster movement, private capital finance can be secured for investments in roads and metros.
- Public sector must maintain a strategic, planning and regulatory role in such scenarios – need very high quality public institutions to perform this role.

2.3.1.5 Strategic Framework and Compare Infrastructure Capabilities of MRT Systems

Mass Rapid Transit (MRT) Systems with different capabilities are certainly require any special infrastructure.So implementing sustainable development objectives must first meet the needs of each city and town level, then the demand of citizens based on needs assessment to be appropriations act. more specifications and requirements of each system Noted:

- Busways – these are generally segregated sections of roadway within major corridors, with horizontal protection from other traffic, and priority over other traffic at junctions, which are generally signalized.
- Light rail transit (LRT) – this is at-grade, with similar horizontal protection to busways.
- Metros – these are fully segregated, usually elevated or underground. It is the segregation that is critical to providing a rapid service, and the technology that allows a high mass ridership to be carried.
- Suburban rail – these services are usually physically part of a larger rail network, usually at-grade and fully segregated incorporating road-rail segregation or controlled level-crossings.

Continue this topic in all urban public transport systems of various aspects is reviewed:

- ➢ Use of space
  - Busways usually involve reallocation of existing roadspace
  - LRT often does the same, but may also add new capacity, e.g., when using former rail alignment.
  - Metros add new capacity, typically increasing the passenger-carrying capacity of a major corridor by a factor of 3 (they may have no impact on road capacity, or if elevated lead to small reduction)
- ➢ Integration
  - All systems require interchange to provide an integrated public transport system
  - Rail systems, and busways operating ‘trunk-and-feeder’ services require more interchange.
- ➢ Capacity
  - Busways depending on specification, have a practical capacity of 10-20,000 passengers per hour per direction (pphpd), or occasionally higher.
  - There are no examples of LRT carrying flows in excess of 10,000 pphpd, and there is reason to doubt whether they can achieve much higher flows.
  - Metros by comparison carry very large passenger volumes – 60,000 pphpd or higher; and high-specification suburban rail can typically carry 30,000 pphpd.
- ➢ Level of service
  - Rail systems can generally provide a high quality ride, and when segregated, regularity.
  - Bus systems perform less well in these respects.
- ➢ Common Features
  - Physically segregated busways

- Trunk-feeder operations
- Fare prepayment, flat fares, free transfers with feeder buses
- High station platforms
- Mostly operated by private bus companies
- High passenger demand
- Quick implementation
- Much lower cost than LRT or metro alternative
- Metro-like appearance
- Distinct identity and good image

3. CONCLUSIONS

In this paper, transport strategies was evaluated. Based on the concept of sustainable urban development, spatial development framework that will justify the stability of the current needs without spatial risk for future generations the ability to supply their needs have to provide. Sustainable cities need sustainable urban transport , therefore to create the infrastructure for the needs of communities in the principles is essential. Transport planning in cities is focused on passenger transport.Urban freight transport is a significant threat and needs a higher policy and action priority . So strategies for increasing transportation sustainability include demand management,operations management,pricing policies,vehicle transportation planning. In order to achieve sustainable development objectives of urban citizens must first identify local needs and potentials in each region then assess funding and schedule specific transport strategies to be implemented step by step.

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GIS BASED STUDIES OF GEOTECHNICAL PROPERTIES FOR LAND DEVELOPMENT PLANNING AND DESIGN

Wan Nur Syazwani W.M[1] and A. Naser Abdul Ghani[2]

[1]Universiti Teknologi MARA (Perak) Malaysia,

[2]Universiti Sains Malaysia

[1]wannur956@perak.uitm.edu.my , [2]anaser@usm.my

ABSTRACT: The benefits of GIS in terms of data access and representation contribute to the ability to make a quick decision making. It will also provide the basis of design and budgeting as well as creating a platform of communications among practitioners especially in relation to land development. Therefore, this paper describes the use of a Geographic Information System (GIS) in a geotechnical study applied in the area of Seri Iskandar, an upcoming new township in the District of Perak Tengah, Perak Darul Ridzuan, Malaysia. The study is to see how the various soil data information can be stored in GIS format. The database can be retrieved for processing and can be reproduced into meaningful decision support information in terms of geotechnical properties of the area concerned. The ability to access data at any time (now or later) is the main benefit of using this GIS method, moreover it will reduce time and possibly cost of site investigations. The GIS system developed in this study is capable of quickly producing various geotechnical information in any part of the study area. This information can be in the form of maps of soil types, soil strength, and water levels. It can also produce detail information relevant to the site and laboratory test results. This information will be useful to planners, architects and engineers in planning and designing suitable buildings and infrastructures in the area. The paper concludes with discussions of possible future application of the GIS in the technical aspects of land development planning and design.

Keyword: Geographic Information System, GIS, Geotechnical, Land Development

1. INTRODUCTION

Land development becomes increasingly challenging today as our population increases and available land becomes scarcer. Thus, smart development in planning is important in order to sustain the development. In addition, design, technology selection, and any other issue related to construction must be taken into consideration in developing any land. Moreover, the developer spends a lot of money and sets budgets to demonstrate the development programmes and the designs must comply with the regulations needed. According to Budhu (2004), soil investigation process which is an important part before the design stage is being implemented in order to save a million ringgit and to keep the budget accordingly.

However, currently most developers, contractors and consultants are not serious on this site investigation process due to the cost being expensive. As a result, the projects tend to fail and the cost will over-run (Budhu, 2004). Another issue related to site investigation is the report done after the investigation is over. The question is, how the report can be stored and how about ten years later, the report be retrieved.

2. LITERATURE REVIEW

As a developing country, technology becomes important in Malaysia. The development of technology in daily life is also important. Thus, the application of GIS (geographic Information system) is also needed especially, for site investigations preliminary study. As mentioned by Player (2000), the integration of information together with supporting documents such as maps, books, reports, etc. will give important effects during preliminary studies of site investigations. In the geotechnical discipline, the spatial study covers the related space of areas under study. Thus, by using GIS application, the maps, photos, etc. are also linked with their data feature and are analyzed spatially (Player, 2000).

Moreover, in this geotechnical study, the geotechnical data will integrate as database will be created and also visualization in terms of mapping, chart, etc. will also be generated to make the users understand the matter well.

Therefore, in land development planning and design, GIS application is more important especially, in the process of site investigations preliminary study. In addition, GIS acts as a decision tool to determine the future development and foundation available in that certain area of focus. Then, to develop the database of the site, ArcGIS is applied within 1:10,000 of scale. All the site investigation information especially related to soil investigation is also needed and used in order to create the database.

3. STUDY AREA

This paper covers the area of Seri Iskandar, District of Perak Tengah, Perak Darul Ridzuan. Currently, the new township of Seri Iskandar is developing as an administrative capital which consists of a lot of government buildings, commercial buildings, and education centers such as UiTM Perak and Universiti Teknologi Petronas. According to Majlis Perbandaran Seri Iskandar (2009), Seri Iskandar area covers 3,422.14 hectare or 34.22km$^{2.}$ The site locations are divided into four areas such as Bandar Baru Seri Iskandar (600.63 acres), UiTM Perak (11.6 acres), Latihan Kemas (ILK) Seri Iskandar (7 acres), and Government Quarters (63 acres).

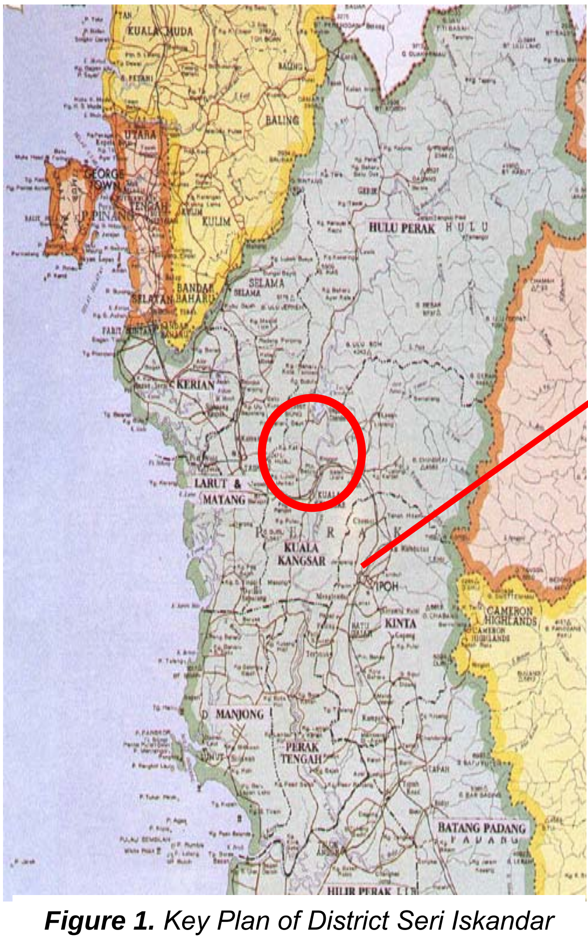

Figure 1. *Key Plan of District Seri Iskandar*
(Source: www.googleimage.com)

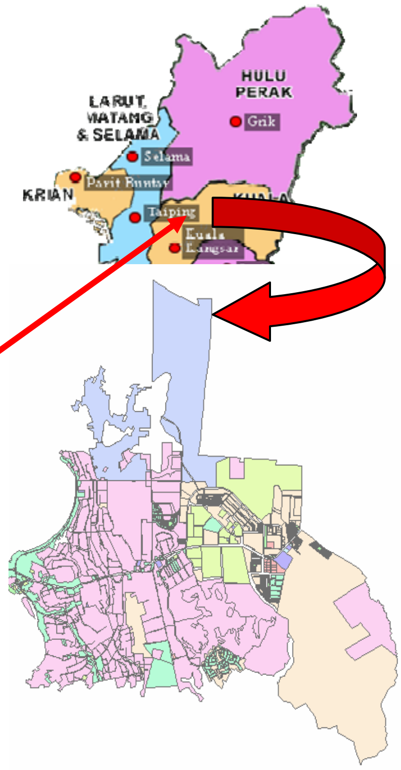

Figure 2. Site Plan of District Seri Iskandar
(Source: www.googleimage.com)
(Source: www.googleimage.com)

4. METHODOLOGY

ArcGIS is one of Geographic Information Systems used in this research. There are two parts in this research where the first is the creation of a database and also a map display or visualization. In the development of database and visualization, most tools inside the ArcGIS are used.

5. RESULT AND DISCUSSION

The figure below shows the attribute table of soil types and also soil penetration test for 3m, 5m, 10m, 15m, 20m and 25m. With the help of ArcGIS, soil information data can be stored and retrieved effectively. For table 1, the qualitative method is applied to explain the soil types rather than the quantity. Meanwhile, for attribute table of SPT (Table 2), the quantitative method is applied as borehole name acts as a dependent variable, and SPT location is the independent variable.

Database

Table 1. *Attribute table for soil types (Source: ESRI, 2008)*

Attributes of SOIL_TYPES_AREA

FID	Shape	Id	NAME	Input_FID	Soil_3m	Soil_5m	Soil_10m	Soil_15m	Soil_20m	Soil_25m
1	Polygon	0	BH3	4	Sand	Clay	Clay	Clay	Clay	Clay
4	Polygon	0	BH8	6	Clay	Clay	Clay	Silt	Silt	
8	Polygon	0	BH4	1	Clay	Sand	Clay			
9	Polygon	0	BH5	0	Clay	Clay	Clay			
10	Polygon	0	BH11	14	Sand	Sand	Clay			
11	Polygon	0	BH12	12	Clay	Clay	Clay	Silt	Fine Soil	
13	Polygon	0	BH14	10	Clay	None	Clay	Clay		
7	Polygon	0	BH1	2	Fine Sand	Fine Sand	Fine Sand	Granite		
5	Polygon	0	BH9	5	None	Fine soil	Fine soil	Silt		
14	Polygon	0	BH15	9	Clay	Fine soil	Fine soil	Gravel		
0	Polygon	0	BH7	7	Sand	Clay	None			
2	Polygon	0	BH10	13	Clay	Silt	None	Sand		
6	Polygon	0	BH2	3	Sand	Sand	Sand			
3	Polygon	0	BH6	8	Sand	Clay	Sandstone			
12	Polygon	0	BH13	11	Clay	Clay	Silt			

Table 2. *Attribute table for SPT (Source: ESRI, 2008)*

Attributes of SPT LAYER

FID	Shape	Id	WTR_LEVEL	NAME	Input	SPT_3M	SPT_5M	SPT_10M	SPT_15M	SPT_20M	SPT_25M
0	Polygon	0	0	BH7	7	3	18	0	0	0	0
1	Polygon	0	0	BH3	4	37	50	5	24	44	50
2	Polygon	0	0	BH10	13	4	35	29	50	0	0
3	Polygon	0	0	BH6	8	21	14	0	0	0	0
4	Polygon	0	0	BH8	6	4	8	11	34	50	0
5	Polygon	0	0	BH9	5	3	15	39	50	0	0
6	Polygon	0	0	BH2	3	50	8	50	0	0	0
7	Polygon	0	0	BH1	2	50	33	31	28	0	0
8	Polygon	0	0	BH4	1	0	18	50	0	0	0
9	Polygon	0	0	BH5	0	12	2	50	0	0	0
10	Polygon	0	0	BH11	14	2	14	50	50	0	0
11	Polygon	0	0	BH12	12	13	3	15	50	50	0
12	Polygon	0	0	BH13	11	19	2	50	50	0	0
13	Polygon	0	0	BH14	10	5	2	10	50	0	0
14	Polygon	0	0	BH15	9	6	2	43	50	0	0

In addition, the data can also be represented in terms of a graph as generated from the attribute table above. ArcGIS helps to create the soil layer in every single borehole as per figure below. The layers drawn depend on the interval of soil type in every borehole (figure 1). Moreover, the bar chart can also represent the water level as in figure 2.

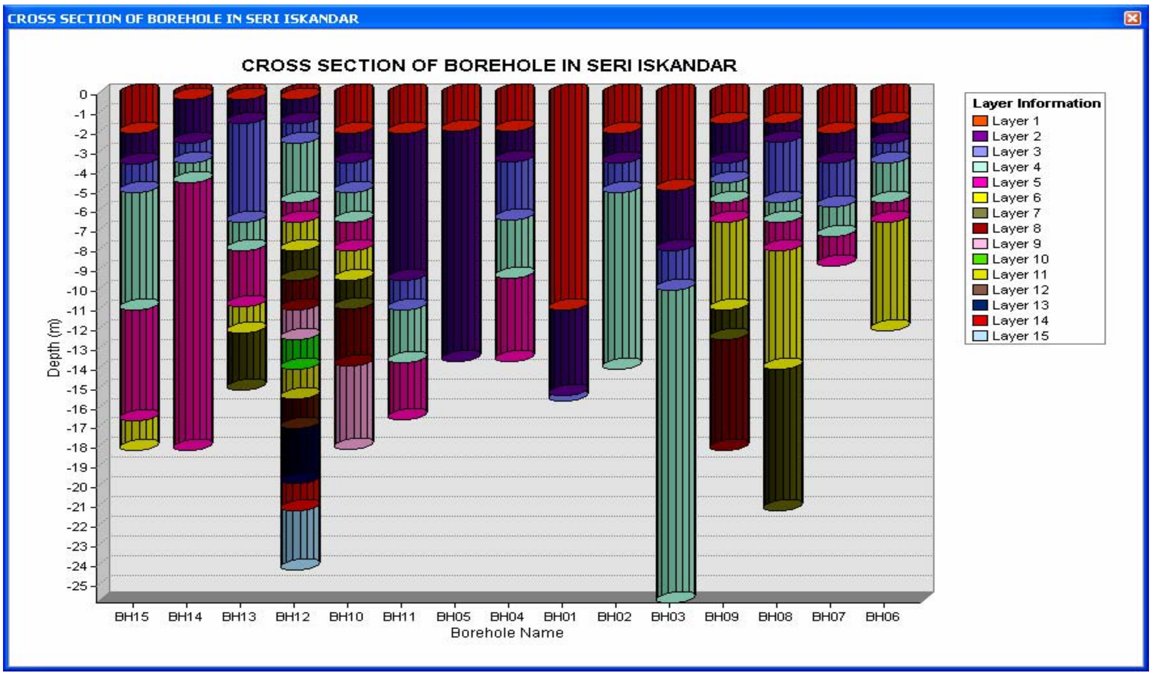

Figure 1. *Bar chart of cross section of boreholes in Seri Iskandar*

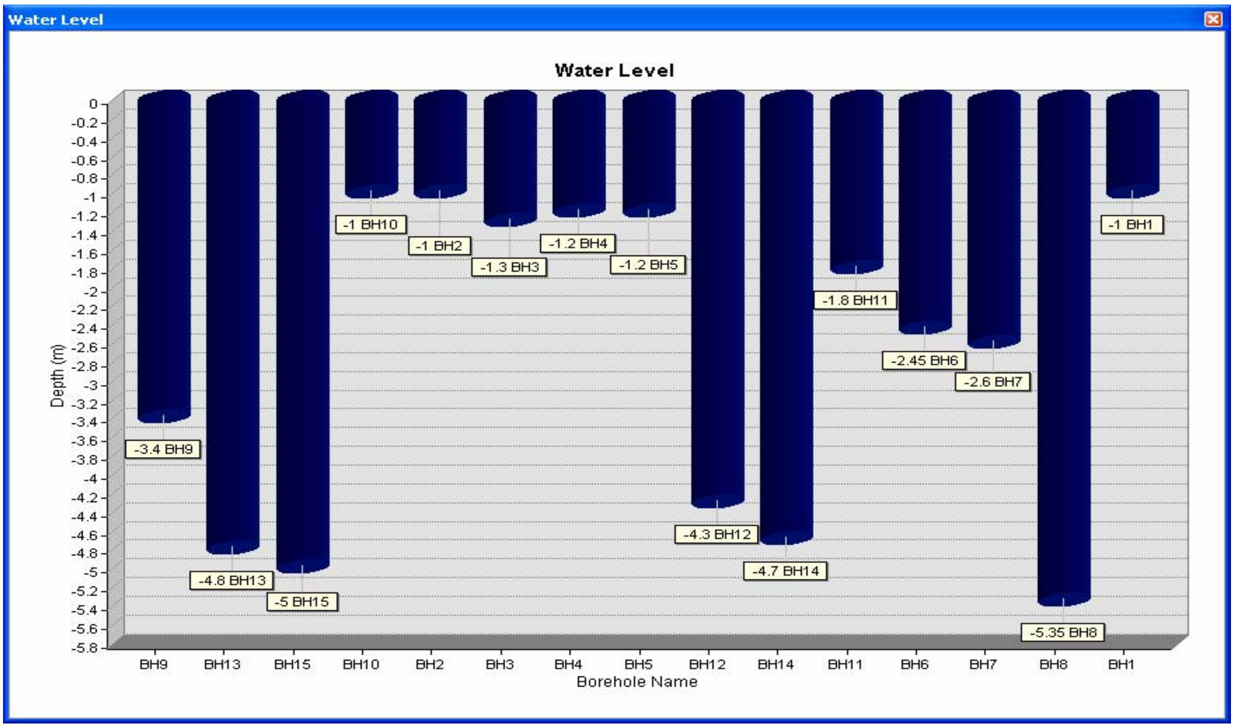

Figure 2. *Bar chart of cross section of water level*

The next visualizer are shows the soil types and SPT in map display as ArcMap is choose.

Map display

The important tools in this ArcMap such as **Arc toolBox-Analysis Tools-proximity-thiessen polygon** are selected. As a result, the point distribution will generate a polygon like the thiessen polygon.

Figure 2, at 25m depth, clay type is shaded at borehole no 3 and another fourteen boreholes are unshaded. It also happens in figure 3, when BH3 only shows the SPT value ie 50 blows/cm and the others do not. In order to classify the various number of SPT, the SPT classification is used. In SPT value map, the result shows that the quantitative map is produced. The various colours show the spatial data with a different value. As shown in figure 5, there are five ranges of classes according to the value of SPT. This really can help the practitioners in the decision making process as they wish to do the development on the area.

It means that, the area of BH3 is harder than other areas and the development can happen here. Thus, this affects the foundation design and budget. Moreover, the type of pile can be proposed at BH3 since it is the deepest and the SPT also reach 50 blows/cm as at this level.

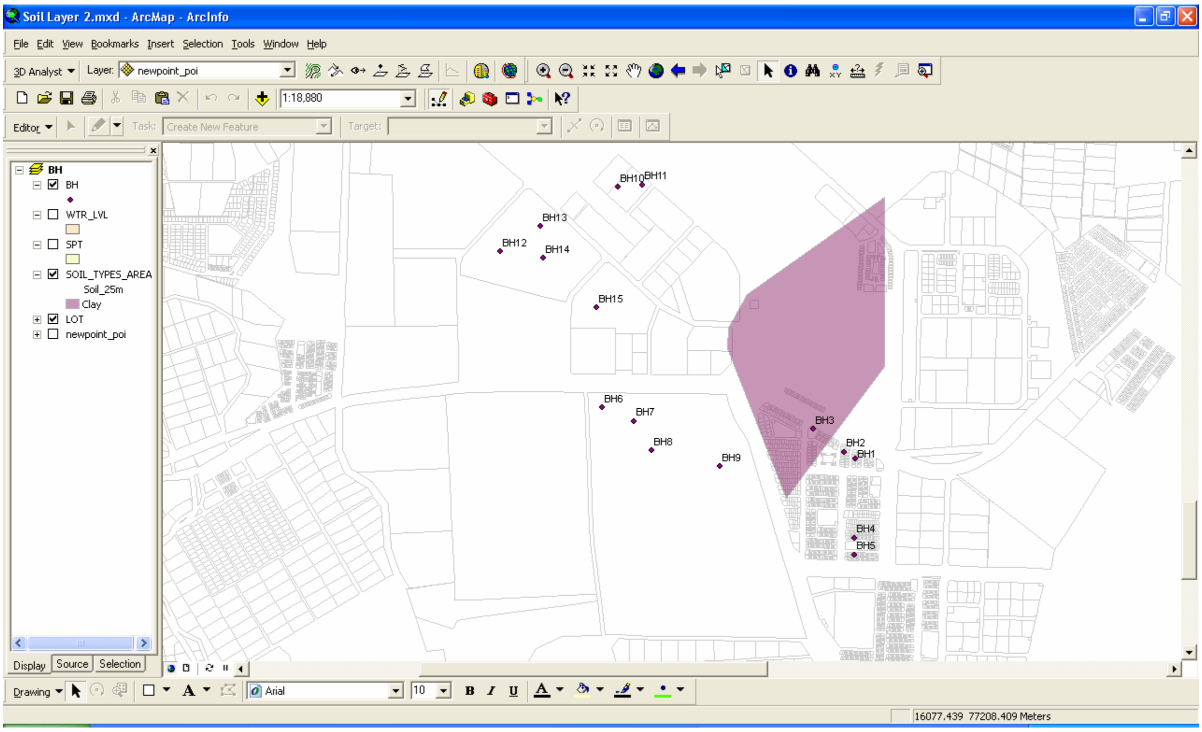

Figure 2. *Soil type area at 25m*

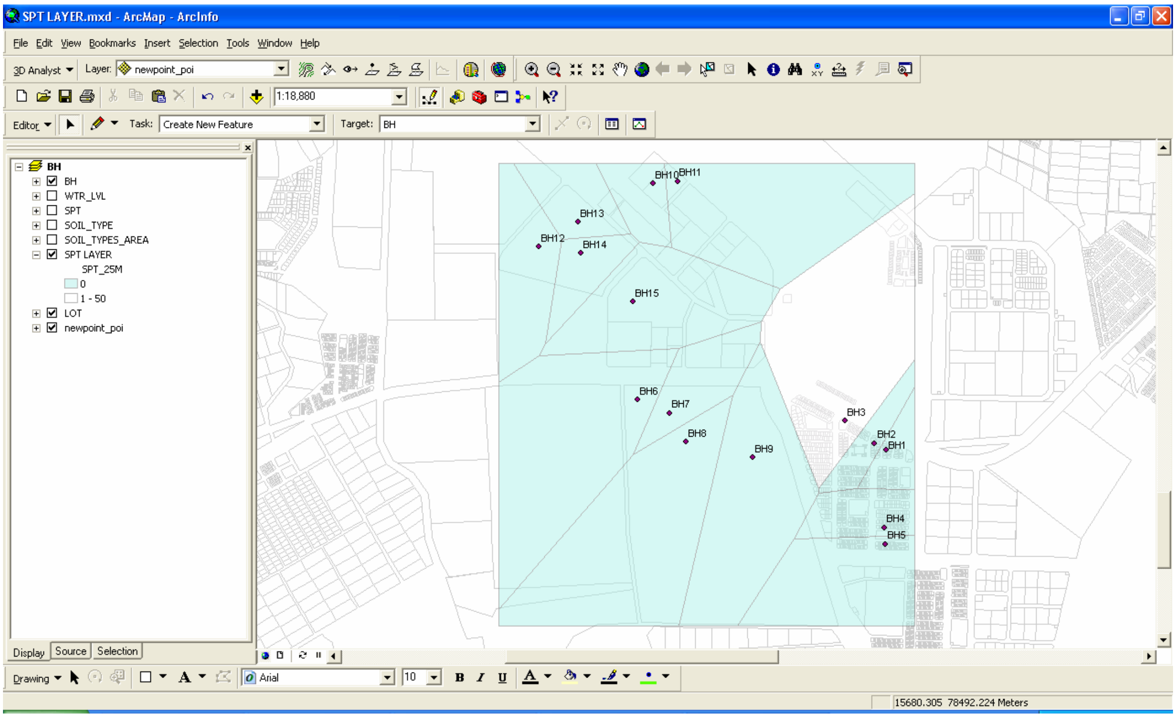

Figure 3. *SPT value at 25m*

Data Presentation

On the other hand, after the soil data are stored in the database form and also produced in the map, then they are ready to be presented to others. Moreover, they can be presented in terms of reports, posters, etc. of which the results are shown in figure 4 and 5 below.

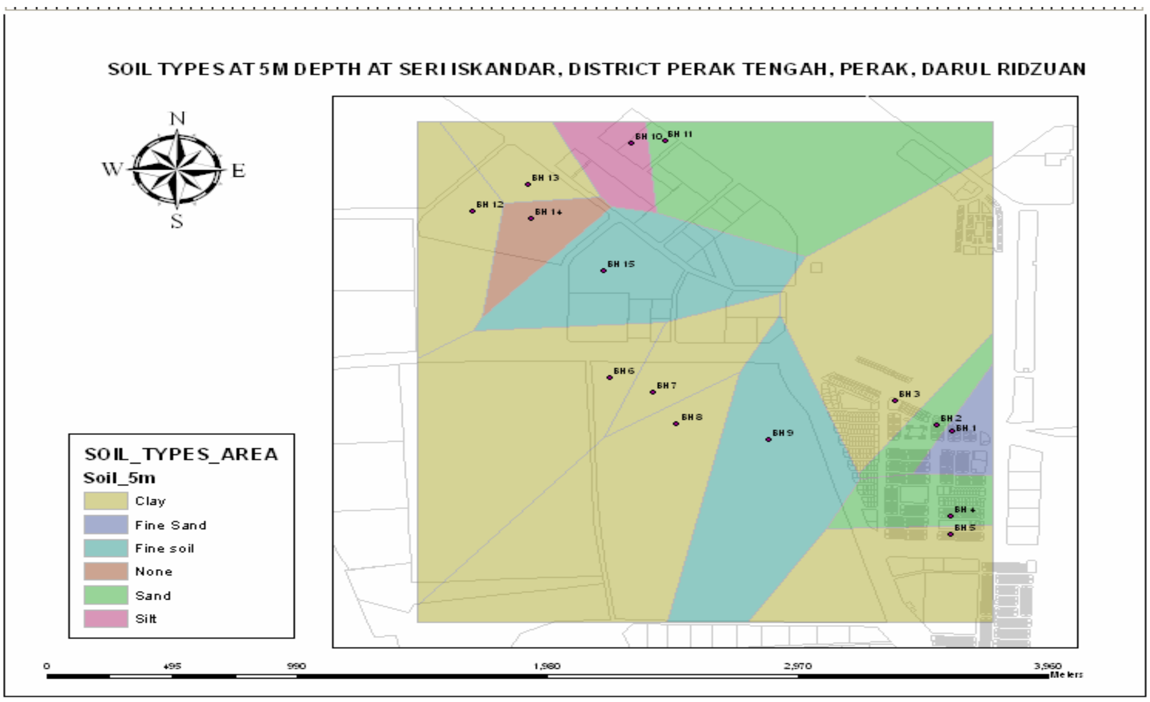

Figure 4. *Map layout design for Soil types at 5 m*

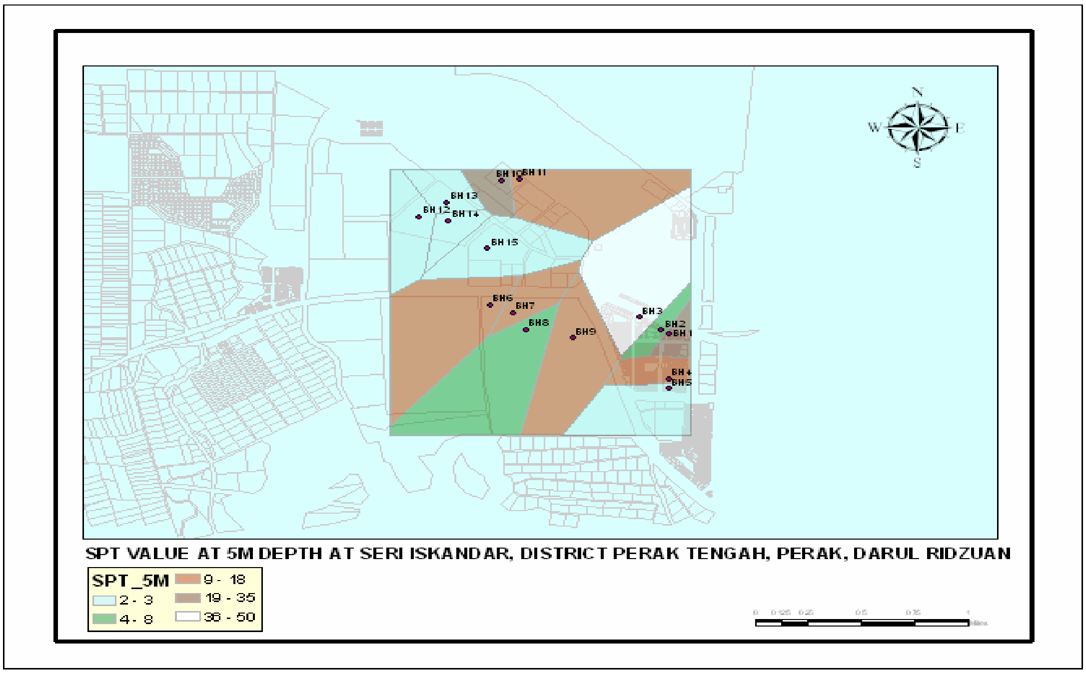

Figure 5. *Map layout design for SPT at 5m*

6. CONCLUSIONS

As a conclusion, ArcGIS tools provide a lot of beneficial information and decision making especially, in determining the development on the area especially at the preliminary stage. With the help of this software, the type of foundation can also be designed according to the soil type and SPT strength. Moreover, the geotechnical data can be saved as storage and can be well managed in the GIS format.

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AN OVERVIEW ON INTEGRATING OF NON-MOTORIZED AND PUBLIC TRANSPORT IN URBAN ENVIRONMENT

Bibie Sara Salleh[1], Riza Atiq Abdullah O.K Rahmat[2] , Amiruddin Ismail[3] ,

[1, 2, 3]Universiti Kebangsaan Malaysia (UKM) , Bangi, Malaysia

[1,2,3]Sustainable Urban Transport Research Centre (SUTRA), UKM, Bangi, Malaysia

bibie_sara@yahoo.com[1]

ABSTRACT: Ownership of cars in Shah Alam as a capital of Selangor, Malaysia is increasing at average rates of eight percent a year. This has resulted the congestion, noise and air pollution. Many of policy measures to reduce the congestion, noise and air pollution have been proposed and implemented such as increased the capacity of road structure, limiting speed, upgrade the public transport system but there is no encouragement on the non-motorized (cycling and walking) as one of mode choice in Malaysia. One of the indicator for sustainable transportation in the state of Selangor is to encourage walking and cycling by the year 2020. Public transport has been seen to be a sustainable transport in the urban areas with the potential of competing with private cars. However, it does not provide door to door service which could mean extending bus routes to all parts of the city. This paper present a brief overview on how walking and bicycling as a feeder mode to increase the coverage of public transport to inaccessible areas. The integrated between public transport and non-motorized vehicles may decrease the number of private cars and increase the quality of life. Malaysia plan towards non motorized also being discussed in this paper**.** As a conclusion, policymakers together with urban planners must approach the challenge of creating a sustainable city with the concerns of pedestrians and cyclists as a feedering mode to access to the public transport.

Keywords : public transport, non-motorized, walking, cycling.

1. INTRODUCTION

In most of developing countries, passenger cars and trucks have become the most important transport modes. The provision of transportation infrastructure in the state of Selangor has been largely oriented towards the needs of private car users. The trends has resulted the limitation of availability, capacity and quality of public transport facilities in Shah Alam and current pattern of town planning and land use. At the same time, the public transport does not provided door to door service where it is inefficient to extend the bus routes to all parts of the city if there is a low demand areas. The choose bicycles and walking as feeder mode can increase the coverage of bus to inaccessible area (Adjei, 2010). Unfortunately, services and facilites for cyclists and pedestrians are mostly inadequated. Therefore, the traffic congestion has worsened in some places such as Batu Tiga, and TTDI Jaya. The problem is usually perceived in terms of safety and inconvenience faced by motorists rather than as a problem for the entire population. The concerns of pedestrians and cyclists are seldom acknowledged by policy makers and planners. This study is to moved away the traditional vies of highway provision that often leads to facilitates car dependent lifestyles where effect the congestion and air quality and hoping that this research can help planners and decision makers in integrated non-motorized and public transport planning in Shah Alam and elsewhere.

2. RESEARCH PROBLEM

The high rate of population increase brings challenges that need attention and action. Inefficient transportation system, inefficient infrastructure supply, insufficient community services and lack of urban design are degrading environmental quality and face issues of decline in quality of life. Inefficient transportation system will increase the private automobile use. Without proper transportation infrastructures, the increase in private automobile may increase in congestion on the roads. High congestions have been attributed to the increase in traffic accident rates, noise and air pollution (Martens, 2004). Approximately 61 percent of the population in Klang Valley live 400 metres from the bus lane (possible walking distance). Roughly from the 4,000 bus stops in Klang Valley,40 percent of them are not roofed and without any signboard. To prevent the problem of less access to public transport, less interaction between mode of transport and high congestion of traffic, people in Malaysia should be alleviated by encouraging to use public transport and non motorized vehicles which are cycling and walking. However, there has been no research on the real factors influencing cycling and walking in Malaysia. Policy measures to eliminate non motorized vehicles from urban arterial and other feeder networks have been implemented in other developing countries such as Delhi, Jakarta, Bangkok, Surabaya, Manila, Bogota and Beijing. To understand the implementation of integrating of non motorized with public transport, this paper present a brief overview on how walking and bicycling as a feeder mode to increase the coverage of public transport to inaccessible areas and how the integrated between public transport and non-motorized vehicles may decrease the number of private cars and increase the quality of life.

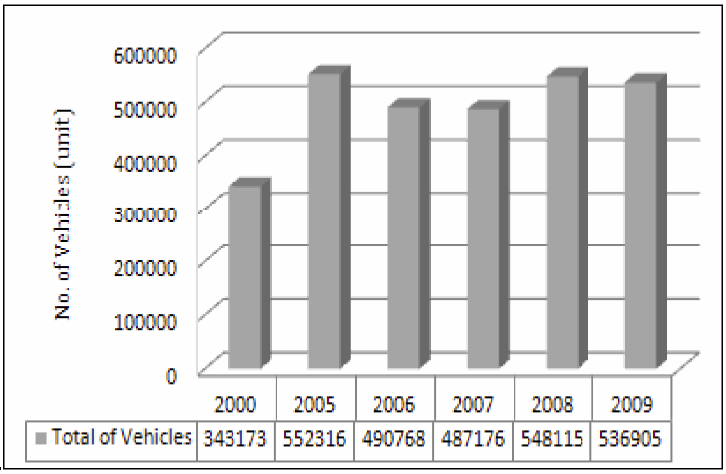

Figure 1. *The increasing number of car ownerships in Malaysia*

(Malaysia Automative Association, 2010)

3. STUDY AREA DESCRIPTION

3.1 BACKGROUND

Malaysia's population in mid-2009 stoode at 28.3 million. From a total of 18.5 million in 2000, the projections from Malaysia's National Physical Plan indicate that the population of Peninsular Malaysia in 2020 would increase to 26.8 million. Selangor's population was 5.07 million in 2008, increasing from 4.1 million in 2007. The projections from Selangor State Structure Plan indicates that Selangor's population would grow by 2.9 percent to 7.3 million in 2020.The major urban centres in Selanogr are Petaling Jaya, Subang Jaya,Shah Alam, Ampang Jaya, Kajang and Klang. Shah Alam city has a population of more than 450,000 people with an area of 293 square kilometres and total revenue collected exceeds RM 100 million yearly. The Shah Alam Municipality has a density of 15.21 persons per hectare (Y.A.B Tan Sri Abd. Khalid bin Ibrahim, 2009).

Shah Alam is a capital of Selangor, one of the state in Malaysia. It is located halfway between Kuala Lumpur and the Port Klang. Shah Alam climate is characterised by a warm, sunny days and cool nights all year round. The temperature ranges from 23°C to 33°C and the annual rainfall is 2670 mm.

Most of the Shah Alam city occupy with housing areas with commercial centres making up most of the rest of the total of 56 city sections (*Seksyen*). The city has five shopping malls such as SACC Mall, (Plaza Alam Sentral), Plaza Shah Alam (formerly known as Shah Alam Mall) Anggerik Mall, Ole-Ole Seksyen 18 and Kompleks PKNS. There are also many schools, shops, banks, , hotels, medical centres and popular hypermarkets (Tesco, Giant and Tesco Extra). The commercial areas are situated mostly at the city centre sections of (Seksyen 14), Seksyen 13 and Seksyen 9.

3.2 PUBLIC TRANSPORT SERVICES

Without access to the Light Rail Transit (LRT) and Monorail, residents have to rely on KTM Komuter, buses and taxis. Shah Alam KTM Komuter serves four stations in Shah Alam that lie on the Port Klang to Sentul line. There are Batu Tiga Komuter station, Shah Alam Komuter station (Seksyen 19), Padang Jawa Komuter station (Seksyen 17) and Bukit Badak Komuter station.

RapidKL_provides bus service to various areas in Shah Alam. Shah Alam KTM Komuter station is also currently served by one local shuttle bus. Batu Tiga KTM Komuter station is accessible via the following rapidKL buses - U62 : Klang - Sunway Pyramid - Klang U64 : Pasaramakota - Taman Sri Muda, Shah Alam - Pasaramakota U65 : Pasaramakota - Taman Alam Megah via Section 20 Shah Alam - Pasaramakota U80 : Pasaramakota - UiTM Section 2 Shah Alam - Pasaramakota T527 : Subang Mas - Section 15 Shah Alam - Subang Mas E4 : Terminal Jalan Sultan Mohamad - Klang - Terminal Jalan Sultan Mohamad. Mini bus service also provided from Shah Alam Kommuter station to Section 16, 17, Kompleks PKNS and Wet World (wikitravel.org, 2011).

3.3 Walking and Cycling Facilities in Shah Alam

Shah Alam Agricultural Park is also known as Taman Pertanian or Bukit Cahaya Seri Alam is one of the largest agriculture park in the world and built around a centerpiece artificial lake. It provides a jogging and biking path that goes around the lakes. Bicycles are available for rent at the park and a free bus service is provided to ferry visitors throughout. Shah Alam Lake Gardens also built around an artificial lake with several playgrounds for children, jogging and biking path around the lakes.

Shah Alam Walk, a pedestrian-friendly area that is one of the attractions in the the city. It covers approximately 0.3 square kilometres which stretching over Jalan Institusi and Jalan Majlis in Shah Alam . It has become the city landmark since 2004 when the city council had decided to spruce up the whole area with beautiful landscape and fountains around the area.

Shah Alam walk is covering three major Shopping Complexes in Shah Alam namely, SACC Mall, PKNS Complex and Plaza Alam Central. The place is also considered as a central business district for the city which consists of private and government offices, banks and shopping centres Shah Alam walk is directly linked with Dataran Shah Alam and other favourite retreats such as the Lake Gardens, Laman Budaya and Galery Shah Alam. For safety and monitoring purposes, the whole area has been equipped with CCTV system which is directly linked to Shah Alam Police Station.

Though Shah Alam walk is popular and attractive for the pedestrian which is located in the middle of the Central and Bussiness District for the city, but vehicles which are parked along the pedestrian way affect the pedestrian. Visitors who come with own vehicles are advised to park their vehicles inside one of the shopping complexes and avoid parking their vehicles along the roadsides in order not to risk their vehicles being towed away by the local authority.

***Figure 1**. Dataran Shah Alam is a part of the walk.*

***Figure 2 .** Walking street with rows of palm trees an Balinese lights in the centre of the city*

***Figure 3**. The walk exudes serenity, especially at night.*

4. BICYCLE AND WALKING AS A FEEDERING MODE

Less interaction between mode of transport effect the inefficiency of public transport service in Malaysia. A very obvious example is less interaction between Monorel and LRT stations at KL Central, Kuala Lumpur. Thuogh daily users approximately 3000 people but they have to walk for a distance of 350 metres nearby construction site and they need to walk along the pedestrian walk which is not well prepared and without roof.Perhaps there are other facilities such as Hand Tuah LRT station and Titiwangsa LRT station where users face the same kind of problems (Transformation Goverment Programme, 2010).

Bike parking is one of the key aspects of integrating bicycling with public transport (Pucher, 2010). The use of bicycle as a feeder mode to bus stop has been seen to be much healthy to the users (Banister, 2008). The combined use of bicycle and public transport could be a competitive alternative to private car (Keijer & Rietveld, 2000). Research found that there is a significant increases in bicycling and public transport use, but mainly for access trip and far less for egress trip (Martens, 2007). Access trip as mention by Martens is bicycle trips between home and suburban rail statio while egress trip means the bicycle trips between the terminal station and the activity end of trip.

Waking is also an important mode to access public transport terminals (Wibowo & Olsxewski, 2005). They found that, the role of walking mode to access rail stations for three cities (BART, San Fransico, Tokyo Metro and Netherlands) as shown in Figure 4. The proportion of walking to access the public transport is more than 20%. There is a higher proportion of walking in egress trip from station to the final destinations since most of the rail stations are located in city centre (Wibowo & Olsxewski, 2005). Most of public transport studies noted that walking as an access mode occurred up to 400 to 800 meters or 10 to 15 minutes of walking time (O’Sullivan and Morrall, 1996).

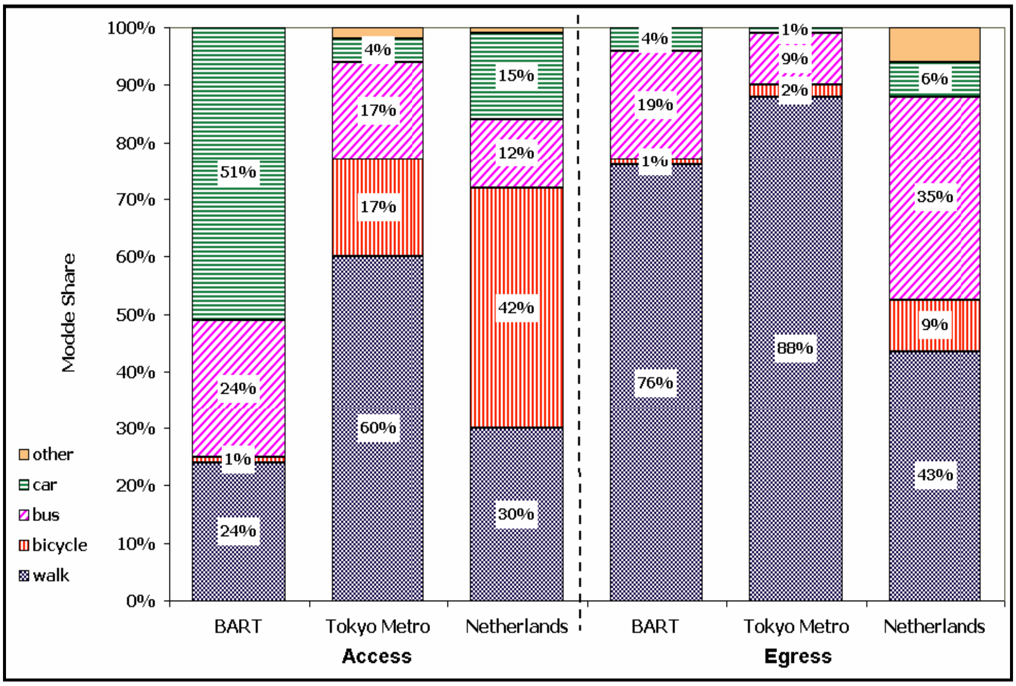

Figure 4 . *Access and Egress Mode Share*
Source : Wibowo and Olsxewski (2005)
BART = Bay Area Rapid Transit, San Francisco

Walking is a portion of total trip mode per day because commuters have to walk to other mode of transport if there is a short distance. According to UK DfT (2007), richer households tend to walk fewer short distance trips. Research found that a 400m walk to bus stops as the most comfortably distance for commuters (Murray et al, 1998). Rising incomes in developing countries will discourage people from cycling and walking in the future (Hook and Replogle, 1996).

5. INTEGRATION OF BICYCLES WITH PUBLIC TRANSPORT

Integration is defined as the movement of commuters from one place to another through rider-friendly intermodal facilities and inter-connections (Ibrahim, 2003). The ability of the cyclist to safely store their bicycle at park and ride or stations contribute to the successful integration of cycling with public transport (Johannesburg Report, 2009). The Bogota transport model is a highly successful, sustainable urban development models promoting non-motorized vehicle and public transport while restricting cars (Joewono &Kubota, 2005). The quality of such facilities in the Netherlands has a positive effect on the perceptions of rail travel and encourage people to use public transport (Givoni and Rietveld, 2007).. The implementation of integrating bicycling with public transport as shown in Table 1.

***Table 1**. Implementation of integrating bicycle and public transport*

Implementation	Strategy	Implementing country	Effects
Bicycle parking at rail stations	• 740.000 bikes parked at Tokyo's metro and train station everyday (Harden, 2008). • 350,000 bikes parked at Dutch train's station (Dutch Nationnal railways, 2009)	• Japan • Netherland	• Provision of good parking at public transport stations increases the number of public transport use as well as levels of bicycling (Rietveld (2000), Netherlands Ministry of Transport (2009))
Bike racks on buses	• 72% of buses equipped with bike racks in US (TRB, 2005) • 80% of Canadian buses equipped with bike racks (TRB, 2005)	• US • Canada	• Public transport system s increases as well as increasing use of bicycle racks (TRB, 2005)
Rental bikes	• With Smart Card and OV-Fiets (public transport bicycle system), bicycle rental provided at 156 Dutch rail stations (Putcher & Buehler,2008) • Call-a-bike rentals at 16 train station in German (Putcher & Buehler, 2008)	• Netherland • German	• Rental programs increased the public transport usage as well as increasing use of bicycle (Litman,2009 & Martens, 2007)
Bikes on rail cars	• In Europe and North America,during off –peak hours,often special space on rail cars that reserved for bikes, sometimes with hooks and bike racks ; many system prohibit bikes during peak hours (TRB, 2005)	• Europe • North America	

Figure 5. *(a) Bicycle rack on front of bus (Burden, 2006) (b) Bicycle rack (Fukushima, 2009)*

These are all examples of integration between public transport and cycling as non-motorized vehicle. While taking lesson from the successful integration elsewhere, it is necessary for Shah Alam or elsewhere to plan a functional integration mechanism depends on the travel demand and traffic characteristic in that area.

6. MALAYSIA PLAN TOWARDS NON MOTORIZED

Malaysia is one the the presentatives of Asian countries at the Fifth Regional Environmentally Sustainable Transport (EST) Forum, which held in Bangkok, Thailand from 23 to 25 August 2010, express the intend to voluntarily develop and realize integrated and sustainable transport policy programmes, options and projects that will help realize the following EST goals and objectives by the year 2020 in the Asian region. One of the strategies to shift towards more sustainable modes is require Non-Motorized Transport (NMT) in transport master plans in all major cities including the improvement to bicycle and pedestrian facilities, adoption of complete street design standards and development of facilities for intermodal connectivity. There are some indicators that countries may considered in achieving a successful Non Motorized Transport Strategy as shown in Table 1.

Table 1. *List of indicators to achieve a successful EST strategy for Non –Motorized Transport (Bangkok Declaration 2020 in Fifth Regional EST Forum, 23-25 August)*

Strategy	Indicator
Non-Motorized Transport	Number of cities with NMT specifically highlighted in transport master plan
	Note the existence of national and local policies requiring drop curbs at interface between intersections and footpaths.
	Note the existence of national and local policies mandating dedicated pedestrian signals at major intersections
	Note the existence of national and local policies mandating minimum footpaths widths.
	Number of cities with dedicated cycleways
	Number of secure bicycle parking spaces
	Number of kilometres of cycleways
	Promote the monitoring and measurement of the quality of pedestrian facilities and the number of cities audited for a walk ability score
	Number of cities with shared bicycle programs
	Number of cities with pedicab (cycle rickshaw) improvement programmes.
	Number of public transport systems with formal integration of pedicabs
	Number of cities participating in a Car-Free Day programme.

In Malaysia, Penang is the first country promoting cycling among locals and tourists. Penang starts proposing three cycling tracks to promote healthy lifestyle, environmental friendly transportation and sports tourism. They was proposing three tracks from Teluk Bahang to George Town, Batu Maung to George Town via Jelutong Expressway and a heritage trail within George Town itself (Cheu, 2008).

Metro Bike is the only one company which provide bicycle rent at RM25 with a deposit of RM100 for 24 hours and people are free to cycle wherever they want. The bicycle for rent is provided at Tanjung City Marina (beside Jetty Bus Terminal in George Town) as a starting point. There are four safety measures for cycling tour in George Town (UNESCO George Town Heritage Cycling Tour Overview, 2010):

a. Weekend morning (les traffic)
b. Safely guided bicycle trails
c. Small group guided cycling
d. Safely pedestrian crossover.

Figure 6. (a)Bicycle for rent at George Town (b) Cycling with small group guided cycling (http://www.metrobike.com.my)

7. CONCLUSIONS

As a conclusion, policymakers together with urban planners must approach the challenge of creating a sustainable city with the concerns of pedestrians and cyclists. The public transport including buses, trains, cabs are provided in Malaysia but it is not provided door to door service where it is inefficient to extend the bus routes to all parts of the city if there is a low demand areas. The choose bicycles and walking as feeder mode can increase the coverage of bus to inaccessible area. This research can help planners and decision makers in integrated non-motorized and public transport planning in Shah Alam and elsewhere.

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A COMPARATIVE STUDY ON RC MULTI-STORIED BUILDING FRAMES DESIGN BETWEEN NON SWAY AND SWAY METHOD USING STAAD. PRO V8i SOFTWARE

Noor Md. Sadiqul Hasan[1], Habibur Rahman Sobuz[2], Costas Ioannou[3] , Md. Shiblee Sayed[4]

[1]Linton University College, Legenda Education Group, Malaysia

[2]Department of Civil Engineering, Universiti Malaysia Sarawak, Malaysia

[3, 4]School of Computing, IT & Engineering, University of East London, UK

[1]s_adiqul@yahoo.com, [2]habibkuet@yahoo.com, [3]c.s.ioannou@uel.ac.uk and [4]shiblee181@yahoo.com

ABSTRACT: The aim of this paper is to investigate the non-sway and sway method multi-storey reinforced concrete frame with the numerical modelling analysis. Two buildings of 15- storeys and 6-storeys having a height of 57.7 m and 23.5 m respectively have been designed by using the design software STAAD Pro V8i for non-sway and sway condition. To make the analysis more practical and economical, column sizes have been changed in every three storeys and beam sizes kept constant. Analysis of the reinforced concrete building frames has shown that how the reinforcement of the beams and columns changed with the effect of wind loading. From the analysis of that reinforced concrete structures, it can be seen that the reinforcement of the beams and columns have been increased in sway frame compare to the non-sway frame. Analysis of the two buildings (one tall and another one short) in reinforced concrete frame show that how the beam and column design is affected by the height of the building. Finally, it is concluded that the software analysis predict the response of the reinforced concrete multi-storied buildings frames very well.

Keywords: column, beam, multi-storey building, non-sway, sway, staad. pro v8i.

1. INTRODUCTION

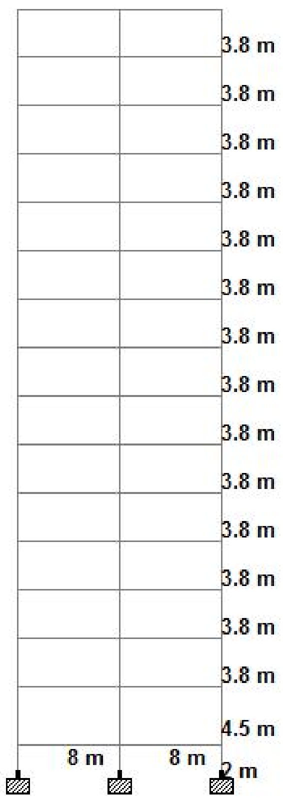
Concrete is one of the most versatile, durable and cost-effective building materials known to people. It has a vast role to play both in the construction and improvement of our civil engineering and infrastructure. Its great strength, durability and flexibility are properties which are utilised in the construction of Buildings, Roads, Bridges, Airports, Railways, Tunnels, Ports and Harbours and many other major infrastructure projects. Concrete is the material which provides the best fire resistance than any other building materials. When it exposed to fire, It does not burn, cannot be 'set on fire' like other materials in a building and it does not emit any toxic fumes, smoke or drip molten particles. Concrete is making an intensive, synchronized effort to further reduce its environmental impact which includes the emergent use of recycled concrete in cement and concrete manufacture. Concrete is classified as an A1 fire resistance material under EN 13501-1 which means that it can sustain itself in highly combustible environment and has a very low rate of heat transfer quality and prove itself as an effective shield against spread of fire. Due to the reason of global warming, sustainability is the most important issue in our present planet. Concrete is the highly sustainable construction material in terms of its thermal mass energy consumption and in energy consumed during its manufacturing. Concrete is the small net contributor which is responsible for 2.6% of UK CO_2 emissions according to 2002 and this compare to 33% and 47% of total CO_2 emission from transport and buildings in use respectively. Having good durability quality, good strength in compression, casting in any shape according to architectural/structural requirement and also cost-effectiveness are some other major reasons for using concrete in the construction industry so much (The Concrete Centre, 2009). According to Wong, et al. (2007, pp. 961-969), the modification of Muto's method has been approached for unbraced

composite frames with semi-rigid beam-to-column connections in multi-storey frames for sway and non sway conditions. Wang (1997, pp. 101-116) has stated beneficial and detrimental effects of structural continuity for non sway multi-storey frames on fire resistant design of steel columns. Li, et al. (2004) has obtained some selected results from the full scale measurements of dynamic behaviour of a 63-storey reinforced concrete tall building.
In this paper, the numerical modelling analysis of the reinforced concrete multi-storied building frames for non sway and sway method using Staad. Pro V8i software is investigated. Using the software the 15 and 6 storey's building have been designed at the condition of without sway and with sway and the analysis results are compared for beams and columns.

2. METHODOLOGY

In reinforced concrete construction, effect of wind in the multi-storey building has played an important role. Sometimes, it may happen to change the mind to allow or not to allow wind loading in the structure. In non sway buildings wind loading may be resisted by shear walls, service cores, housing lifts services and stair cases in the building. In sway buildings, wind loading will be resisted by the beams and the columns of the building which have to be designed considering the wind loading. The main aim of this study is to analyse and design of two multi-storey buildings in concrete (one tall and another one short) both for non sway and sway condition and compare the obtained results (Allen, 1988) and (Mosley & Bungey, 1990).

2.1 Plan and sectional elevation of the proposed building



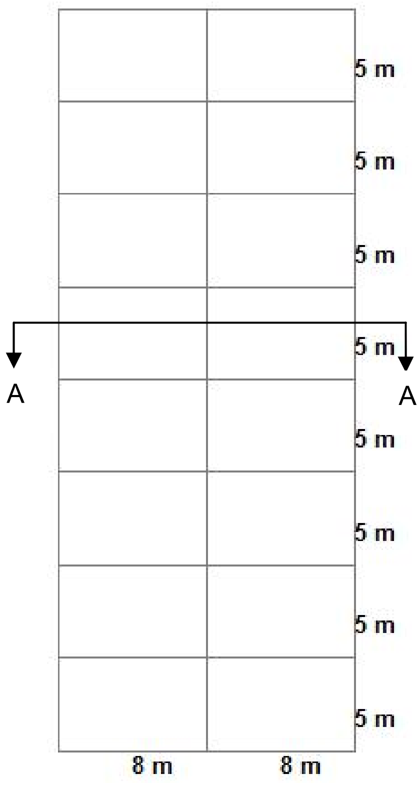


Figure 1. *Plan view and sectional elevation (A-A) of the 15 storey building*

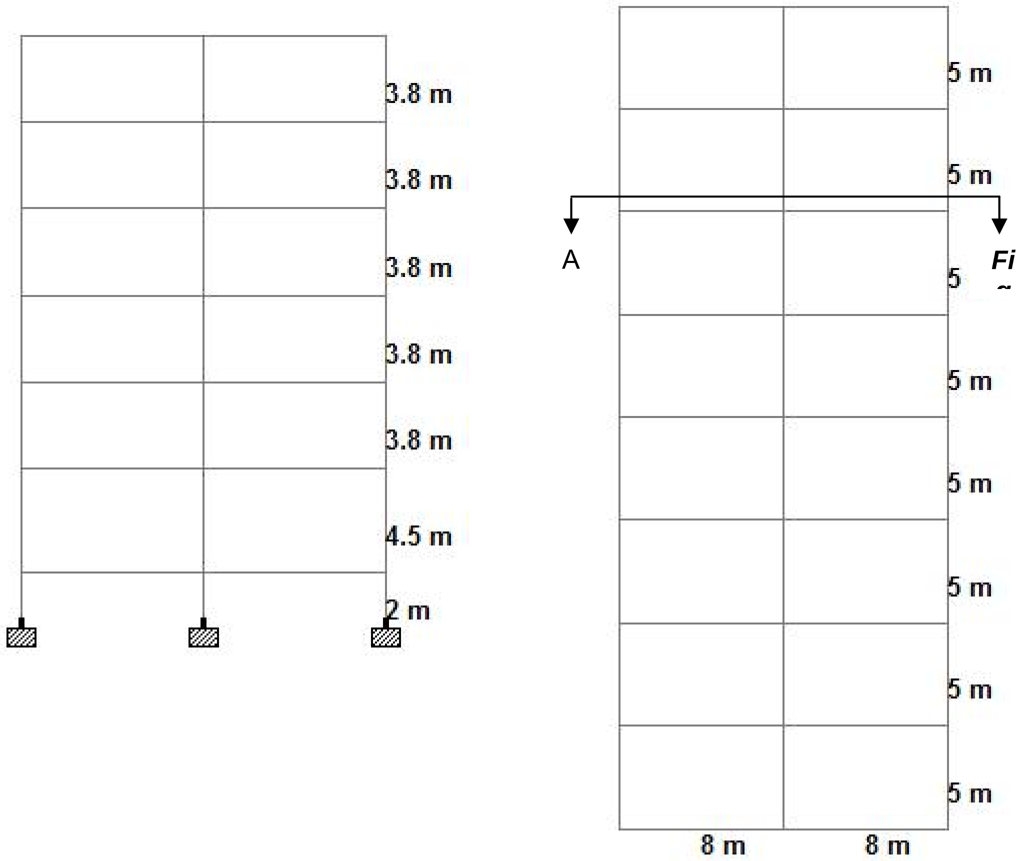


***Figure 2**. Plan view and sectional elevation (A-A) of the 6 storey building*

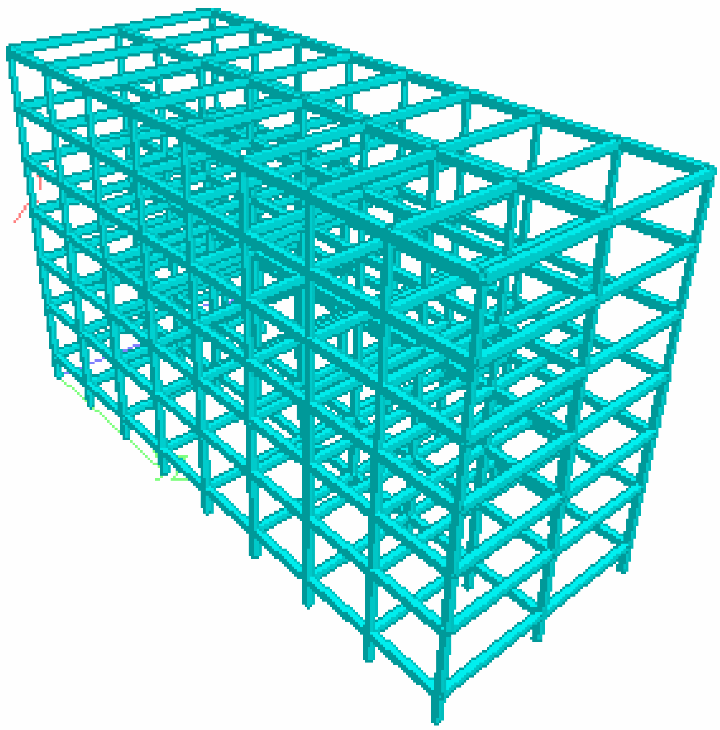


***Figure 3**. Three dimensional rendered view* of *the 6 storey building*

2.2 ANALYSIS OF RESULTS (BRITISH STANDARDS INSTITUTION, 2002)

2.2.1 COMPARISON BETWEEN MIDDLE FRAME WITHOUT SWAY AND WITH SWAY FOR 15 STOREY BUILDING

Table 1. *Comparison of middle column without sway and with sway for middle frame*

Floor	Column size(square) m^2	Middle column without sway			Middle column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.58	11931	12566	10 T 40	11931	12566	10 T 40
3^{rd} Floor	0.51	9908	10053	8 T 40	9908	10053	8 T 40
6^{th} Floor	0.44	7703	7854	4 T 50	7703	7854	4 T 50
9^{th} Floor	0.37	5151	6434	8 T 32	5151	6434	8 T 32
12^{th} Floor	0.26	2860	2945	6 T 25	2860	2945	6 T 25

Table 2. *Comparison of side column without sway and with sway for middle frame*

Floor	Column size (square) m^2	Side column without sway			Side column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.58	3580	3619	18 T 16	5050	5655	18 T 20
3^{rd} Floor	0.51	3658	3770	12 T 20	3759	3770	12 T 20
6^{th} Floor	0.44	3504	3770	12 T 20	3617	3770	12 T 20
9^{th} Floor	0.37	3136	3142	10 T 20	3136	3142	10 T 20
12^{th} Floor	0.26	2479	2945	6 T 25	2633	2945	6 T 25

Table 3. *Comparison of beam (depth = 0.50 m and width = 0.40 m) for middle frame*

Floor	Without sway			With sway		
	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2
Ground Floor	6T20(1885)	5T16(1005)	4T25(1963)	3T40(3770) 6T16(1206)	6T16(1206)	3T40(3770) 6T16(1206)
3^{rd} Floor	5T20(1571)	6T16(1206)	5T25(2454)	4T32(3217) 6T16(1206)	6T16(1206)	3T40(3770) 6T16(1206)
6^{th} Floor	5T20(1571)	6T16(1206)	5T25(2454)	5T25(2454) 6T16(1206)	6T16(1206)	4T32(3217)
9^{th} Floor	5T20(1571)	6T16(1206)	5T25(2454)	6T20(1885)	6T16(1206)	5T25(2454)
12^{th}Floor	6T20(1885)	5T20(1571)	5T16(1005)	6T20(1885)	5T20(1571)	6T16(1206)

**Note: In each row, upper one shows top Reinforcement and lower one shows bottom reinforcement of the beam.*

2.2.2 COMPARISON BETWEEN SIDE FRAME WITHOUT SWAY AND WITH SWAY FOR 15 STOREY BUILDING

Table 4. *Comparison of Middle Column without Sway and with Sway for Side Frame*

Floor	Column size(square) m^2	Middle column without sway			Middle column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.58	4844	4909	10 T 25	4844	4909	10 T 25
3rd Floor	0.51	4267	4399	14 T 20	4267	4399	14 T 20
6th Floor	0.44	3607	3770	12 T 20	3607	3770	12 T 20
9th Floor	0.37	2716	2945	6 T 25	2716	2945	6 T 25
12th Floor	0.26	1842	1885	6 T 20	1842	1885	6 T 20

Table 5. *Comparison of side column without sway and with sway for side frame*

Floor	Column size(square) m^2	Side column without sway			Side column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.58	1346	1358	12 T 12	1792	1810	16 T 12
3rd Floor	0.51	1040	1207	24 T 8	2104	2200	28 T 10
6th Floor	0.44	1007	1207	24 T 8	1735	1810	16 T 12
9th Floor	0.37	1475	1571	20 T 10	2574	2715	24 T 12
12th Floor	0.26	1616	1810	16 T 12	1989	2413	12 T 16

Table 6. *Comparison of beam (Depth = 0.50 m and Width = 0.40 m) for side frame*

Floor	Without sway			With sway		
	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2	Left hand side * mm^2	Middle * mm^2	Right hand side * mm^2
Ground Floor	6T16(1206)	8T10(628)	6T16(1206)	4T32(3217) 7T12(792)	7T12(792)	4T32(3217) 7T12(792)
3rd Floor	4T20(1257)	4T16(804)	3T20(942)	4T32(3217) 7T12(792)	7T12(792)	5T25(2454) 7T12(792)
6th Floor	5T20(1571)	6T12(679)	5T16(1005)	5T25(2454) 6T12(679)	6T12(679)	6T20(1885) 6T12(679)
9th Floor	6T16(1206)	7T12(792)	7T12(792)	6T20(1885)	7T12(792)	6T16(1206) 7T12(792)
12thFloor	8T10(628)	3T20(942)	4T20(1257)	7T12(792) 3T20(942)	3T20(942)	4T20(1257)

**Note: In each row, upper one shows top reinforcement and lower one shows bottom reinforcement of the beam.*

2.2.3 COMPARISON BETWEEN MIDDLE FRAME WITHOUT SWAY AND WITH SWAY FOR 6 STOREY BUILDING

Table 7. *Comparison of Middle Column without Sway and with Sway for Middle Frame*

Floor	Column size(square) m^2	Middle column without sway			Middle column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and Size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.36	5949	7540	6 T 40	5949	7540	6 T 40
3rd Floor	0.26	3078	3217	4 T 32	3078	3217	4 T 32

Table 8. *Comparison of Side Column without Sway and with Sway for Middle Frame*

Floor	Column size(square) m^2	Side column without sway			Side column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.36	2208	2413	12 T 16	2423	2514	8 T 20
3rd Floor	0.26	1873	1885	6 T 20	1971	2945	6 T 25

Table 9. *Comparison of beam (Depth = 0.50 m and Width = 0.40 m) for middle frame*

Floor	Without sway			With sway		
	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2
Ground Floor	5T25(2454)	6T16(1206)	5T20(1571)	5T25(2454)	6T16(1206)	6T20(1885)
3rd Floor	5T25(2454)	5T20(1571)	7T12(792)	5T25(2454)	5T20(1571)	5T16(1006) 5T20(1571)

**Note: In each row, upper one shows top reinforcement and lower one shows bottom reinforcement of the beam.*

2.2.4 COMPARISON BETWEEN SIDE FRAME WITHOUT SWAY AND WITH SWAY FOR 6 STOREY BUILDING

Table 10. *Comparison of middle column without sway and with sway for side frame*

Floor	Column size(square) m^2	Middle column without sway			Middle column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.36	2325	2413	12 T 16	2325	2413	12 T 16
3rd Floor	0.26	1413	1885	6 T 20	1413	1885	6 T 20

Table 11. *Comparison of side column without sway and with sway for side frame*

Floor	Column size(square) m^2	Side column without sway			Side column with sway		
		$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size	$A_{s\ required}$ mm^2	$A_{s\ provided}$ mm^2	Bar number and size
Ground Floor	0.36	518	604	12 T 8	1062	1207	24 T 8
3rd Floor	0.26	1224	1257	4 T 20	1433	1608	8 T 16

Table 12. *Comparison of beam (Depth = 0.50 m and Width = 0.40 m) for side frame*

Floor	Without sway			With sway		
	Left Hand side* mm^2	Middle * mm^2	Right hand side * mm^2	Left hand side* mm^2	Middle * mm^2	Right hand side * mm^2
Ground Floor	7T12(792)	7T12(792)	4T20(1257)	6T16(1206)	7T12(792)	5T20(1571)
3rd Floor	4T12(452)	5T16(1006)	5T20(1571)	5T12(565) 5T16(1006)	5T16(1006)	5T20(1571)

*Note: In each row, upper one shows top reinforcement and lower one shows bottom reinforcement of the beam.

2.3 DISCUSSION OF RESULTS

2.3.1 FOR 15 STOREY BUILDING

According to Table 1, it can be seen that middle column of middle frame without sway and with sway have the same amount of reinforcement in each floor. This is because, large amount of axial load from load combination 1.4 Dead Load + 1.6 Imposed Load is governing rather than the load combination 1.2 Dead Load + 1.2 Imposed Load + 1.2 Wind Load.
From Table 2, it can be seen that the reinforcement in the side column of middle frame has increased to 18 T 20 (5050 mm^2) in sway frame from 18 T 16 (3580 mm^2) at ground floor because of the governing load combination which includes wind load. And for all other side columns though area of reinforcement has slightly (approximately 3%) increased, same size of bar are used.
From Table 3, it can be found that the bottom reinforcement at middle of the beam is almost the same in sway and non sway cases but extra amount of reinforcement has provided in the left and right hand side bottom of the beam for sway cases except 9th and 12th floor. This is happened so to develop the necessary moment of resistance because the moment has increased significantly (approximately 50-60%) due to wind loading. And in 9th and 12th floor, the amount of reinforcement (approximately 20%) has also increased for wind loading in sway cases.
According to Table 4, it can be seen that middle column of side frame without sway and with sway have the same amount of reinforcement in each floor. This is because, large amount of axial load by load combination 1.4 Dead Load + 1.6 Imposed Load is governing rather than the load combination 1.2 Dead Load + 1.2 Imposed Load + 1.2 Wind Load.
From Table 5, it can be observed that reinforcement has increased (approximately 33-83%) for all the side column of the side frame because of the governing wind load combination.
From Table 6, it can be found that the bottom reinforcement at middle of beam is almost the same in sway and non sway cases but extra amount of reinforcement is needed in the left and right hand side bottom of the beam for sway cases. This is happened so to develop the necessary moment of resistance because the moment has increased significantly (approximately 20-50%) due to wind loading.

2.3.2 FOR 6 STOREY BUILDING

According to Table 7 and 10, it can be found that middle column of middle and side frame for sway and non sway cases, the amount of reinforcement is same in each floor. As because large amount of axial load by load combination 1.4 Dead Load + 1.6 Imposed Load is governing rather than the load combination 1.2 Dead Load + 1.2 Imposed Load + 1.2 Wind Load. And from Table 8, it can be observed that the amount of reinforcement has increased (approximately 4%) in sway cases for side column of middle frame because of wind loading.

According to Table 9, it can be observed that bottom reinforcement at middle of the beam and top reinforcement at right hand side of the beam are almost the same in sway and non sway cases. But at ground floor, approximately 20% more top reinforcement is needed at right hand side of the beam in sway frame due to wind loading. And at 3rd floor, approximately 27% more top reinforcement is needed at right hand side of the beam and also extra amount of reinforcement (5T20) has provided at right hand side bottom of the beam for sway cases due to wind loading.

From Table 11, it can be seen that the amount of reinforcement has increased significantly (approximately 100%) and slightly (approximately 4%) for ground floor and 3rd floor respectively due to wind loading.

From Table 12, it can be found that bottom reinforcement at middle of the beam is almost the same in both sway and non sway condition. But at ground floor, approximately 52% and 25% more top reinforcement is needed at left and right hand side of the beam respectively in sway frame due to wind loading. At 3rd floor, approximately 25% more top reinforcement is needed at left hand side of the beam and also extra amount of reinforcement (5T16) has provided at left hand side bottom of the beam for sway cases due to wind loading.

According to the design results of the 15 and 6 storey building, it can be seen that the middle column became same in both sway and non sway cases. In 15 and 6 storey building, the amount of reinforcement has increased slightly (approximately 4%) in sway cases for side column of middle frame. But for side column of side frame, the reinforcement has increased significantly (approximately up to 100%) for both 15 and 6 storey building in sway cases due to wind loading. In 15 storey building, beam reinforcement has increased approximately up to 60% and in 6 storey building, it has increased approximately up to 30% in sway frame. This is so happened as because of the significant effect of wind loading in 15 storey building.

3. CONCLUSIONS

The following conclusion can be made based on the analysis of the RC multi-storied building frames for non sway and sway method using Staad. Pro V8i software:

- Because of load increasing from top to bottom, the size of column has also increased in every three storeys as economy is a big issue.
- As because of symmetry, the design results of the right hand side and left hand side column of every frame is similar.
- The design results of the left hand side and right hand side beams for all frames are the mirror of each other due to symmetry of load and plan of the building.
- It can be observed that if the height of the building has shortened, column sizes become much smaller but beam size remained constant.
- In sway and non-sway cases, the reinforcement of middle column remains constant but for side column, it has increased in sway cases due to wind loading. Side column of side frame of the building had shown more change of reinforcement in sway cases for wind loading.
- The bottom middle reinforcement of the beam is almost the same for both sway and non sway cases but extra amount of reinforcement has provided at the left and right hand side of the beam for sway conditions due to wind loading.
- Beam reinforcement has not changed significantly in the upper floors because of smaller effect of wind loading at the upper floors.

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SUSTAINABLE CONCRETE FROM MALAYSIA'S INDUSTRIAL BY-PRODUCTS AND BIOGENIC WASTES

M. R. Karim[1], M. F. M. Zain[2], M. Jamil[3], F.C. Lai[4] and M. N. Islam[5]

[1,2,3]Universiti Kebangsaan Malaysia, Malaysia.

[4]Regional Technology Support Centre, Sika Kimia Sdn Bhd, Malaysia

[5]Dhaka University of Engineering and Technology, Gazipur, Bangladesh.

[1]reza_civil128@yahoo.com , [2]fauzi@vlis.eng.ukm.my , [3]lin@eng.ukm.my , [4]lfcclm@yahoo.com.sg , [5]nazrul2100@yahoo.com

ABSTRACT: In the modern society, for the dwelling house and construction of infrastructures, the role of concrete is unavoidable. Besides, sustainability and sustainable concrete is a prime issue in concrete industry in recent decades. Sustainable concrete could be produced using waste material in cement or concrete and it will be a valuable step for: reduction of CO_2 emission, making more durable and cost effective concrete, saving energy, and solving disposal problem. In this paper, a critical review and proposal on the consumption of Malaysia's waste materials as a supplement of cement for the production of sustainable concrete have been presented. The physical, chemical properties waste materials and merits of supplementary use of waste materials in cement and concrete have also been pointed out. It can be concluded, depending on the information from published literatures, that effective utilization of these waste materials as a substitute of cement will influence researchers to examine a sustainable way of saving materials including cement as well as manufacture of sustainable concrete.

Keywords: s**ustainable goncrete, waste materials, CO_2 emission, energy**

1. INTRODUCTION

In the present civilized world, due to growing population and living requirements the construction of buildings and infrastructures has been increased rapidly. Among the other construction materials, concrete has gained popularity for its multiple advantages: easiest manufacturing procedure, strength and durability properties at normal environment. The majority of commonly used cement - ordinary Portland cement (OPC) - is a key ingredient in concrete production. It is reported that Portland cement is accountable for the emissions of approximately 7% world's carbon dioxide (CO_2); in consequence, global warming is rising gradually (Malhotra 2000; Mehta, 1999). In addition to CO_2 emissions, the burning of Portland-cement clinker at temperatures around 1400 ^{0}C is costly in terms of fossil fuel usage (Nehdi 2001). These effects of cement and concrete production on environment and the depletion of the world's most valuable fossil energy resources have necessitated the exploitation of sustainable construction materials (Kartik et al. 2003). In consequence, nowadays, utilization of waste materials as a potential alternative in the construction industry is a great research interest in cement and concrete technology and it would be a valuable step for solving the mentioned issues. Waste materials could be used effectively as construction materials and have the capability to satisfy the design specifications provided that it would be processed properly. In this regards, various researches have been performed for the use of biogenic wastes including palm oil fuel ash (POFA), rice husk ash (RHA), sawdust ash/ash from timber (AFT) and industrial waste (slag, fly ash) as constituents of cement and concrete.

The challenges listed above are more a result of the fact that Portland cement is not particularly environmentally friendly. One can therefore reduce these challenges following simple formula: use as much concrete, but with as little Portland cement as possible, this means to replace as much Portland cement as possible by supplementary cementitious materials, especially those that are industrial by-products, and biogenic wastes (POFA, RHA, AFT). This study summarizes the use of supplementary cementing materials in sustainable concrete and a proposal to produce sustainable concrete from Malaysia's industrial by-product and biogenic wastes.

2. MALAYSIA'S BIOGENIC WASTES AND INDUSTRIAL BY-PRODUCT

There are many biogenic wastes such as wastes from palm oil industries, rice husk ash, are produced in Malaysia. It is estimated that about 0.3 million ton of POFA is generated per year in Malaysia. About 2.6 million tonnes of solid waste in the form of oil palm shells is produced annually in Malaysia (Basri et al., 1999). It was reported that will be about 20 tons of nut shells, 7 tons of fibers, and 25 tons of empty bunches discharged from the mill for each of 100 tonnes in processing of fresh fruit bunches (Tay and show, 1995). About 650 million ton of rice produced in the world, of which 2.2 million is from Malaysia. After milling about 20% is converted to rice husk; and finally, 15-20% ash is produced as RHA, after burning (FAOSTAT, 2008). A huge amount of industrial by-products (slag, fly ash) are also produced from this country, as well as in the world. It is observed that, till to date, there are no remarkable uses of these ashes, the POFA. These ashes are only dumped into surroundings without any commercial feedback; hence contamination and disposal problem has also been observed. Slag, RHA, and AFT have also been claimed for similar trouble. It is already proven that all of these waste materials contain a high amount of silicon dioxide in amorphous form and they could be used as pozzolanic materials in concrete production. The physical and chemical properties of these materials, as documented in different literatures, are mentioned in Table 1. It is seen from the table that RHA contains maximum silica (about 90%), although POFA has more than 50% silica content. Los of ignition (LOI) value of POFA is large as compare to other materials, fineness value of the materials are also greater than 4000 cm^2/gm. All of these properties are valuable for the production of blended cement and blended concrete as well as sustainable concrete, as suggested in different articles.

Table 1. *Chemical and Physical Properties of POFA, RHA and Slag (Wt. %)*

Material	Chemical Properties									Reference
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O_3	K_2O	LOI	
POFA	43.60	11.50	4.70	8.40	4.80	2.80	0.39	3.50	18.0	Awal, 1997
	63.60	1.60	1.40	7.60	3.90	0.20	0.10	6.90	9.6	Chindaprasirt et. al., 2008
RHA	87.6	0.68	0.93	1.30	0.35	-	0.12	2.37	-	Givi et. al., 2010
	92.5^a 88.5^b	1.2 1.1	2.1 2.0	0.9 0.9	0.4 0.4	0.1 0.1	2.0 2.6	0.0 0.0	0.9 0.9	Chatveera and Lertwattanaruk, 2009
Slag	39.18	10.18	2.02	32.82	8.52	-	1.14	0.30	1.0	Oner and Akyuz, 2007

	Physical properties			
	Material Specific gravity (gm/cm^3)	Mean particle size (μm)	Fineness (Blaine)	Reference
POFA	2.22	-	519 (m^2/kg)	Awal, 1997
	2.43	7.40	99.00 (retained 1%)	Jaturapitakkul et al., 2007
RHA	2.30^a; 2.27^b	-	4750(cm^2/gm); 5750(cm^2/gm)	Chatveera and Lertwattanaruk2 009
Slag	2.87	-	4250(cm^2/gm)	Oner and Akyuz, 2007
	a, b - RHA obtained from electric power station and rice mill respectively			

3. SUSTAINABLE CONCRETE AND WASTES

It is well known that sustainable concrete is a vital issue in concrete industry all over the world and its development could meet the following important matters: reduction of the amount of polluting and CO_2 gases emitted during the manufacture of concrete; more efficient use of waste materials; development of low-energy, long-lasting, flexible buildings and structures; exploiting the thermal mass of concrete in a structure to reduce energy demand. Probably the most effective means of achieving significant reductions in CO_2 emission lies in the replacement of Portland cement clinker by other suitable materials. These replacement materials can be added separately to the concrete or used to replace the clinker in composite cements. The latter is more commonly the situation in Europe, whilst in the US replacement materials are more commonly added to the concrete. For sustainable concrete production, more efficient use of waste materials has been emphasized here in the following ways;

Direct use of waste in cement: Waste materials from other industries are increasingly being used to replace the traditional raw materials used in the production of Portland cement clinker. These include foundry sands, fly ash and bottom ash from coal fired power plants, spent catalysts and filter clays, mill scales, etc. For example, blast furnace slag (BFS) is rich in calcium oxide. The maximum level of limestone replacement by BFS is between 20 and 30%. In practice, replacement levels of about 10% are more commonly reported. When concomitant reductions in fuel consumption are also considered (since less heat is needed to de-carbonate the limestone), total CO_2 reductions can in theory be as high as 25%. High CaO (class C) fly ashes can also be used to replace limestone by up to 10%. According to embodied CO_2 study (2007), the embodied CO_2 emission is reduced significantly due to incorporation of slag into cement. It was found that embodied CO_2 emission is reduced by 56 kg/ton (36.6%) for addition of 50% slag. Ehrenberg and Geiseler (1997) concluded that the total CO_2 emission would be reduced by incorporation of slag in the cement manufacturing. They stated that total CO_2 emission is reduced to 53.3% for 50% slag addition (for example: CEM I without slag, CO_2 emission is 1011 kg/ton but for 50% slag, emission is reduced to 539 kg/ton). This figure is much better for 75% slag replacement. They also mentioned the energy requirements for the cement manufacturing as shown in Table 2. It can easily be observed from this table that, 1587kWh/ton energy is required for CEM I (without slag), but for the addition of 30, 50 and 75% slag with cement the energy consumption is reduced to 1206, 938 and 602 kWh/ton respectively. In case of 50% slag replacement, energy requirement is reduced to 59.1% with comparison to CEM I (100%), alternatively, 40.9% energy can be saved for the use of 50% slag. This quantity is much better for the 75% slag replacement (i.e., only 37.9% energy required as compared to CEM I that means 62.1% energy could be saved).

Table 2. *Energy Consumption in Cement Production (Ehrenberg and Geiseler 1997)*

Sl No.	Type of cement	Energy requirement			
		Total		Electrical energy (%)	Thermal energy (%)
		(kWh/ton)	(%)		
1	CEM I	1587	100.0	39.6	60.4
2	CEM II/B-S with 30% slag	1206	76.0	44.6	55.6
3	CEM III/A with 50% slag	938	59.1	48.9	51.1
4	CEM III/B with 75% slag	602	37.9	60.2	39.8

Utilization of waste in concrete: Although a huge amount of industrial by products are produced worldwide, for example 450 million ton of fly ash per year, but only a few quantity (35 million ton, less than 8%) is used (Mehta 1999). Similar trend have been happened for slag, only a small fraction of the 100 million tons produced worldwide is used (Nehdi, 2001). These by products act as pozzolanic materials and could be used as supplement of cement in concrete as stated in different literatures. Slag is usually used in concrete due to the following benefits: it improves durability and reduces porosity; improves the interface with the aggregate; lowers cement requirement; saves energy; and shows good performance as well as better engineering properties (Oner and Akyuz, 2007). Addition of fly ash increases strength of concrete (Memon et al., 2002; Oner et al., 2005). The POFA, RHA could be used as cement replacement materials in concrete production that are claimed by several researchers. Incorporation of RHA improves compressive strength (Givi et al., 2010) and resistance against sulfate attack (Chatveera and Lertwattanaruk, 2009).

In some cases, their contributions not only save cement but also make better performance in the concrete properties – strength, durability. For example, POFA can be used as a cement replacement to produce good resistance against sulfate attack (Chindaprasirt et al., 2008; Jaturapitakkul et al., 2007). The simple equation is: more utilization of waste in cement or concrete which means less use of cement and hence saving fuel, clinker as well as reduction of CO_2 emission; that are the objectives of sustainable concrete. It is understandable from above discussion that incorporation of wastes (fly ash, POFA, slag, RHA) in cement or concrete is a great achievement in terms of CO_2 reduction and energy saving without sacrificing the strength of concrete.

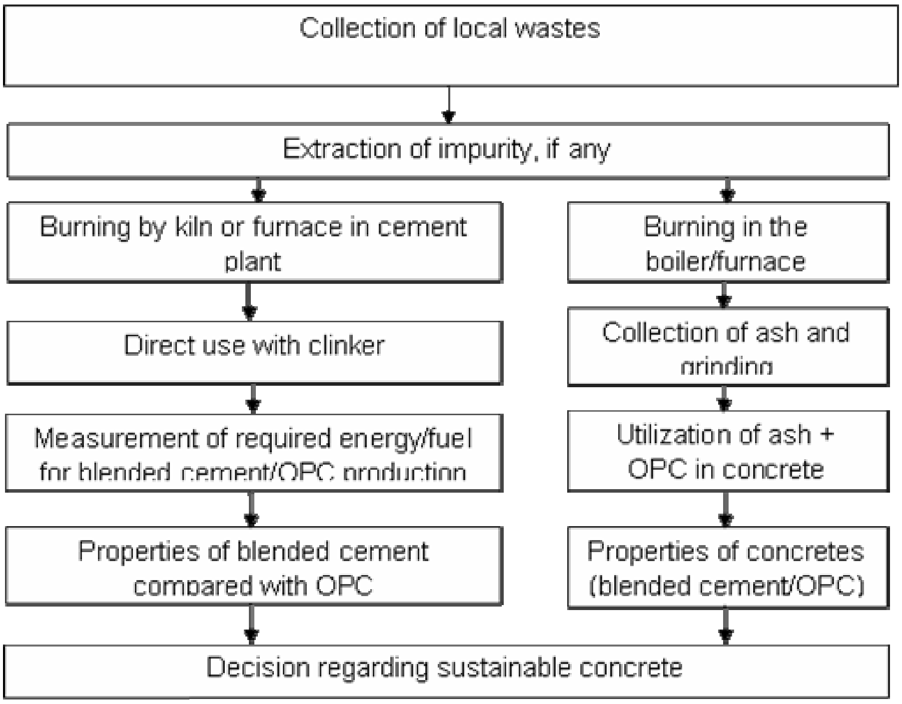
4. WAY OF SUSTAINABLE CONCRETE

As a goal of sustainable concrete, the lower energy consumption and less CO_2 emission should be established in sustainable concrete production. Lower energy requirement have been observed in a study of Ehrenberg and Geiseler (1997) for slag replacement in cement production as shown in Table

1. They found that energy requirements could be reduced by addition of wastes in cement production. This waste replacement method in cement production will be valuable and potential step for the production of sustainable concrete as well as sustainable development. Thus, the authors propose to produce sustainable concrete using locally available wastes in Malaysia in the following way (Fig. 1). As shown in flow diagram, sustainable concrete could be possible because of the following reasons:

- Effective use of waste in cement and concrete
- Lower utilization of cement, hence lower energy consumption
- Reduction of CO_2 emission due to less cement utilization

It is relevant to mention here that POFA, RHA, AFT are produced at a temperature about 500-600^{0}C but for burning clinker the required temperature is about 1400^{0}C. Hence, at the preliminary investigation, it can be estimated that fuel energy could be saved. Besides, Ehrenberg and Geiseler (1997) findings also support this proposal regarding energy saving as well as sustainable concrete production.



**Need not to burn, could be directly used with clinker or in grinding*

Figure 1. *Flow-chart of sustainable concrete production*

5. CONCLUSIONS

It is well known that the shifting of wet process to dry process of cement manufacturing, about 30% energy can be saved. Now involving high volume of biogenic wastes and industrial by-products in cement and concrete manufacturing will be promising venture in construction industry for the sustainable concrete production. This waste utilization process not only saves energy but also reduces the CO_2 emission and global warming – a crucial issue facing by the inhabitants of the planet. Thus, for the production of sustainable concrete, consumption of these wastes in cement and concrete is rational, essential and valid issue for comfortable survival of human community in the world.

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THE STATE-OF-ART OF DECISION SUPPORT SYSTEM DEVELOPMENT: PROGRESS OF REQUIREMENT ENGINEERING IN CONSTRUCTION

Mohd Faizal Omar[1], Bambang Trigunarsyah[2] and Johnny Wong[3]

[1, 2, 3]School of Urban Development, Faculty of Built Environment & Engineering, Queensland University of Technology

[1]mfaizal.omar@gmail.com , [2]bambang.trigunarsyah@qut.edu.au , [3]johnny.wong@qut.edu.au

ABSTRACT: Recent advances in decision making technology have spread into a wide and diverse area particularly in project management. A growing interest of Decision Support System (DSS) in construction project management has been identified as a promising and interesting research area. Much research have been conducted within construction project life cycle phase including the initiation, planning, design and development, detailed design, procurement, manufacture and construction, commissioning and operation and maintenance. However, most DSS model developed were lacked of many part of computing fundamental such as Requirement Engineering (RE). RE is particularly essential for a successful software development. The academic literature on DSS in construction is largely focused on applications area, and only a few articles examine RE issues. Thus, this paper reviews and summarizes current usage of DSS in construction, recent technology developments related to RE, and trends in the deployment of such systems.

Keywords: Construction Project Management, Decision Support System, Software Engineering, Requirement Engineering

1. INTRODUCTION

Designing useful artefacts is complex due to the need for creative advances in domain areas in which existing theory is often insufficient as technical knowledge grows. IT is applied to a new application area that was not previously believed to be amenable to IT support (Hevner, March, Park, & Ram, 2004). This is especially true in the construction industry today as the industry increasingly adopts information technology for use in new and innovative ways.

In the area of Information System, DSS has a long history, started back in 1960's. From time to time, it has widely grows to many application area. This includes construction project management. Significant efforts were done during 1990's and it keep continuing in construction area. However, the trend only focusing on model development and discard many aspect of computing and DSS which essentially produce unusable model in this area. The purpose of this paper is to promote formal technique to DSS development in construction. In addition, the current practice of DSS in construction is also investigated. This paper will start from the historical perspectives of DSS. Later, it introduces some Software Engineering terminology such as Requirement Engineering which can be a useful tool to adopt in construction area. Later, this study investigates the current practice of DSS in construction and reveals some flaws from planning to operation and maintenance phase.

2. HISTORICAL PERSPECTIVES OF DECISION SUPPORT SYSTEM

Historically, DSS research began in the 1960's where the emphasis of the researches was more on conceptual aspect of decision making (Simon, 1960, as cited in Angehrn & Jelassi, 1994; Anthony, 1965, as cited in Shim et al., 2002). Later, a more refined decision support framework has been developed which incorporated categories of management activities and description of decision type (Gorry & Morton, 1989). This includes management activities such as strategic planning, management control and operational control. In addition, the propose framework is the combination of Antony's

(1965) management activities and Simon's (1960) explanation of decision types which consist of structured, unstructured and semi-structured.

Since then, various DSS related researches have been carried out in diverse area to decision making problem. A dramatic DSS improvement and development have been evolved in nearly half century and it seems to be growing to application area and its supportive technologies. In 1970's, researches were more concentrated heavily on decision model development and technology to support single user (Bhargava, Power, & Sun, 2007). Later, in mid 1980's, Group Decision Support Systems (GDSS) or Group Support System (GSS) were developed to facilitate brainstorming, idea evaluation and communication to support team problem solving (Shim, et al., 2002). Furthermore, the concept of Executive Information Systems (EIS) were established to expand the concept of DSS to support decision making from personal use to small group or corporate level (Turban, Aronson, & Liang, 2005). From 1990's, the communication infrastructure has been enhanced by the concept of client-server computing. Thus, this technology has influenced DSS research to adopt this technology and the previous mainframe based DSS became obsolete (Bhargava, et al., 2007). In addition, several firms introduced a new kind of data-oriented decision support via the use of online analytical processing (OLAP) tools (Turban, et al., 2005). In the 21st century, the vast usages of internet have influences DSS research to a new dimension. Within this period, a lot of supportive technologies have been developed. The integration concept of data warehouse, data visualization and Artificial Intelligence (AI) techniques have drawn attention many researchers from diverse area (Turban, et al., 2005). In the near future, the trend of DSS development are projected to experience a rapid growth on the application area and the supportive technologies (Bhargava, et al., 2007).

According to a survey by Eom & Kim (Eom & Kim, 2006), the DSS application areas can be broadly divided into corporate functional management fields (73.33% of the total 210 applications articles) and other areas (26.67%). In corporate area, DSS applications are developed for Production Operation and Management (44.16%) followed by marketing, transportation, and logistics (17.53%), MIS (13.64%), and multifunctional areas (8.44%). Meanwhile, in the non-corporate area, usage of DSSs was widely spread between two dominant sectors of government (19.64%) and educational institutions (16.07%). Other non-corporate areas are appears in miscellaneous, (14.29%), natural resources (12.5%), hospital and healthcare (12.50%), military (10.71%), urban/community planning and administration (7.14%), and agriculture (7.14%). Figure 1 illustrates the distribution of DSS application area that exists in the literature. These results were compiled from two well-known online data sources: (1) The Global edition of ABI/INFORM, which is available on the web through ProQuest and (2) Academic Search Premier database is the world's largest scholarly which managed by the EBSCO information services. In Figure 1(a), all articles are broadly divided into corporate functional management (154 article) and non-corporate areas (56 article). Corporate functional management accounts for 73% of the application articles and is further divided into eight functional areas. Next, in Figure 1(b), 27% of the application concern non-corporate subjects and it is subdivided into eight areas.

Based on the above result and due to the multidisciplinary nature of DSS research, we can assume that DSS for construction project management may lies in the area of strategic management, government, natural resources, urban/community planning and agriculture. These applications might include DSS for construction problems such as contractor selection, consultant selection, project procurement evaluation, preservation of civil infrastructure, etc. As a result, DSS has been proven to be a significant and popular approach to solve construction management decision making problem. Since most of researchers come from non-computing background, most DSS research in construction tend to focus in decision model development rather than computing aspect of DSS. Clearly, it is identified that computing aspects in DSS development for construction area are not well covered in the literature.

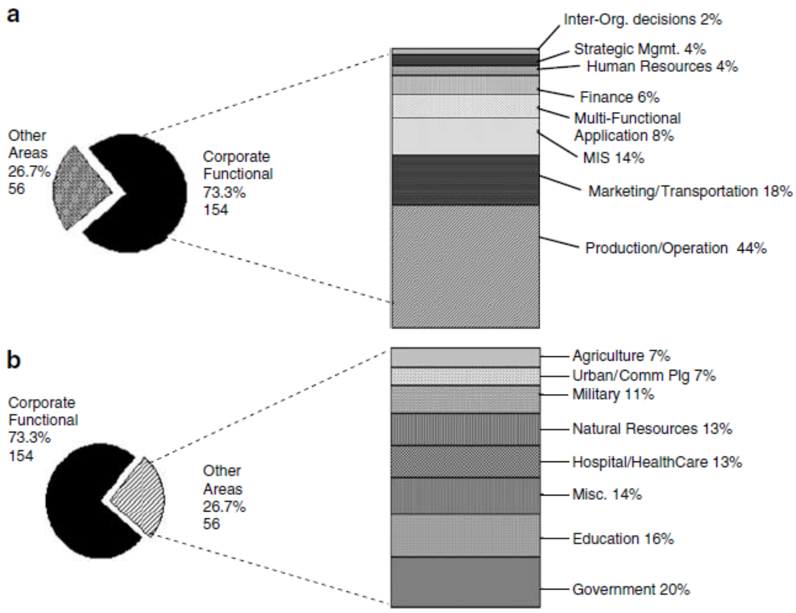

***Figure 1**. Distribution of DSS Application Area in the Literature (Eom & Kim, 2006)*

From the late 1990s onwards, it is obvious that contribution of DSS applications in construction project grows significantly. With faster hardware and advanced software, DSS has been used as a tool to support decision making throughout project delivery life cycle phase. Most of these researches have been applied in planning phase with different kind of applications and decision models. Yet, there were also efforts to deploy DSS in construction management stages such as at design (Sarka, Zavadskas, Ustinovicius, Sarkiene, & Ignatavicius, 2008; Tam, Tong, & Chiu, 2006), procurement (Kumaraswamy & Dissanayaka, 2001) and operation & maintenance phase (Oad, Garcia, Kinzli, Patterson, & Shafike, 2009; Shen & Grivas, 1996). At this stage, combinations of Artificial Intelligence, multi criteria decision making and statistical techniques have been utilized. These decision models were explored extensively to solve a particular problem in construction. The recent practices of DSS in construction have shown that more complex decision models were deployed whereas there are only little computing aspect were discussed. Therefore, the following sections will illustrates some software issues for DSS in construction project management.

3. THE FUNDAMENTAL IN REQUIREMENT ENGINEERING

Few researches have shown that many large projects fail because of inadequate requirements (van Lamsweerde, 2000). These projects were found to have insufficient software specification. Software specification can be refer as a formulation in terms of objects manipulated by the software, in a vocabulary accessible to programmers (van Lamsweerde, 2000). They capture required relations between input and output software objects. In computing area, the process of capturing software requirement is fall under Software Engineering branch of discipline that so called Requirement Engineering (RE).

According van Lamsweerde (2000), RE concerned with the identification of the goals to be achieved by the envisioned system, the operationalisation of such goals into services and constraints, and the assignment of responsibilities for the resulting requirements to agents such as humans, devices, and software. Meanwhile, Nuseibeh & Easterbrook (2000) suggest RE is a process of discovering the purpose of the software, by identifying stakeholders and their needs, and documenting these in a form that is amendable to analysis, communication, and subsequent implementation. In short, RE is the process by which the requirements of the software are determined (B. H. C. Cheng & Atlee, 2007).

Successful RE involves understanding the needs of users, customers, and other stakeholders; understanding the contexts in which the to-be-developed software will be used; modelling, analysing, negotiating and documenting the stockholder's requirements; validating that the documented requirements match the negotiated requirements and managing requirement evolution (B. H. C. Cheng & Atlee, 2007). From these definitions, it is obvious that the quality of software require a step-by-step or structured process of software development. Therefore, a framework of RE process has been proposed in Figure 2.

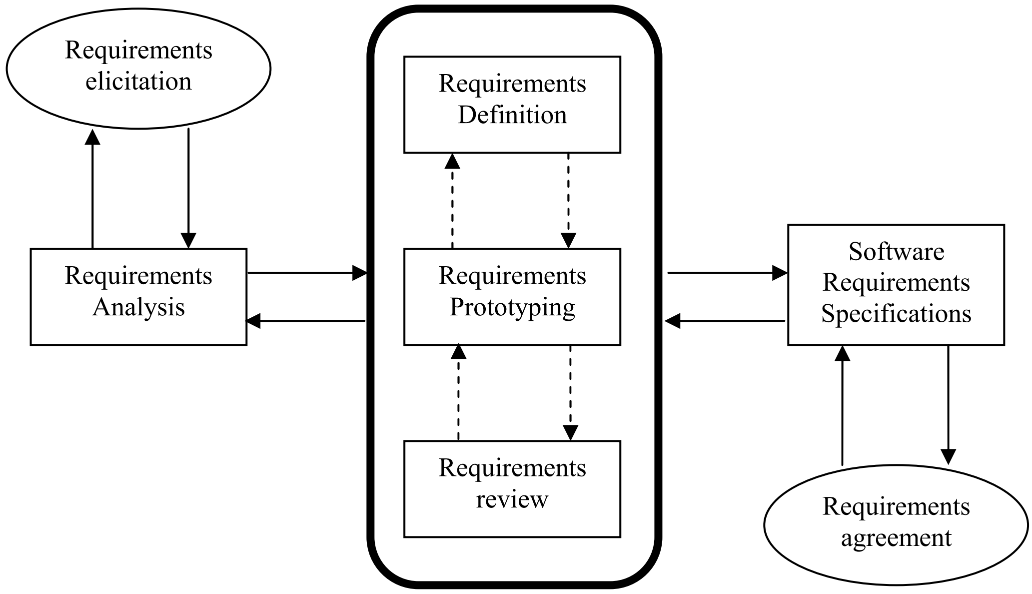

Figure 2. *A Requirement Engineering Process (Tsui & Karam, 2007)*

Based on Figure 2, there are six basic RE process i.e. Requirement Elicitation, Requirement Analysis, Requirement Definition, and Requirement Documentation. The first step is requirement elicitation. Elicitation of requirement can be regarded as an initial step in the RE process where the name itself suggest to "capture" the needs of problem domain. It is at this point the process where the needs of the users and goals for the system are determined. Requirement elicitation can be broadly defined as the acquisition of goals, constraints, and features for a proposed system by means of investigation and analysis (Coulin & Zowghi, 2005). Requirement analysis is the second step of requirement engineering. The purpose of this step is to develop software conceptual model and analysis for DSS system. Tsui & Karam (2007) reveal that even after the requirement are elicited and collected, there are still just an unorganized data and need to be analysed. Requirement analysis is rarely being discussed and not well defined in most DSS literature. Only a few attempts have been made to adopt requirement analysis for DSS in their implementation (Giorgini, Rizzi, & Garzetti, 2008; Lee & Rao). While many of DSS studies in different areas overlooked the importance of software engineering, Giorgini et al. (2008) have come out with a better framework to integrate conceptual modelling and decision model in requirement analysis.

Requirement definition is a technique before proceeding to the prototyping phase in a typical Information System development. According to Yeates & Wakefield (2004), as the requirements are identified, two models of the system are built. The first represents the processing necessary to meet those requirements, and the second portrays the underlying structure of the data that is needed to support this processing. Tsui & Karam (2007) suggest that these processes can be done after the requirement analysis. For each of these models the distinct techniques of Data Flow Modelling and Logical Data Modelling are prescribed (Tsui & Karam, 2007; Yeates & Wakefield, 2004). Next is the Requirement Prototyping process. It is a process of building a model of a system and it is usually employed to help system designers build an information system that intuitive and easy to manipulate for end users. Prototyping comes in many forms - from low tech sketches or paper screens (pictive) from which users and developers can paste controls and objects, to high tech operational systems using CASE (computer-aided software engineering) or fourth generation languages and everywhere in between(Base, 2011). According to literature, there are many component of prototyping such as data management and data communication (Tsui & Karam, 2007).

The rapid expansion of the software applications, specifically "tailor-made" software such as DSS has brought user-centred computing into prominence. The best practice of software development is to balance and sit in the middle between two distinct areas i.e human centred computing and software engineering. In order to achieve that, few study in software engineering literature adopt the usability concept into RE process particularly for requirement review phase (Adikari, McDonald, & Collings, 2006). Requirement review is usually performs after the prototyping process (Tsui & Karam, 2007). Reviews can be achieved by conducting the usability testing and obtained useful feedback from the domain expert (Adikari, et al., 2006; Komarkova, Visek, & Novak, 2007; Paech & Kohler, 2003). This can help to refine the software requirement specifications(Adikari, 2008). Lastly, the requirement agreement is a legitimate document between the end user and the software developer.

By completing the aforementioned steps, it is anticipated that the software is ready to be developed as all the requirement has been compiled and will minimize issues or problems during post implementation and delivery period. Nevertheless, formal RE process is rarely discussed in construction area. Thus, the following section will investigate the current practice for DSS development in construction.

4. REQUIREMENT ENGINEERING IN DSS FOR CONSTRUCTION PRACTICE

In recent years, DSS researchers in construction are moving forward to solve MCDM problems particularly in planning phase. Typically, MCDM techniques such as Analytical Hierarchical Process (AHP), Analytical Network Process (ANP) and Technique for Order Preference by Similarity to Ideal Solution (TOPSIS) with a combination of fuzzy technique are the most utilized for qualitative based DSS. It is applied to a wide range of construction application such as project delivery selection (Al Khalil, 2002; E. W. L. Cheng & Li, 2005; Dey, 2006; Molenaar & Songer, 2001; Sarka, et al., 2008), contractor selection (Ibrahim, Mike, Sami, & Alex, 2002), supplier selection (Kahraman, Cebeci, & Ulukan, 2003), equipment selection (Shapira & Goldenberg, 2005), consultant selection (Al-Besher, 1998; Cheung, Kuen, & Skitmore, 2002; Kit, 2005). Apart from that, there is also a DSS that has been implemented to evaluate concession project investment by using mathematical modelling and finance analysis (McCowan & Mohamed, 2002). Other applications were dedicated to selection of best value bid (Lin, Wang, & Yu, 2008) and to support value management workshop (Shichao, Qiping, & Kelly, 2008). These full details of DSS applications have been listed in the Appendix.

The current practices of DSS in construction have shown that more complex decision models were deployed whereas there are only little computing aspect were discussed. It is also identified that decision models were complex and does not represent a robust DSS as it heavily emphasis in decision model rather than the practical usage in real world i.e. via computerized system. Decision models were impractical and unused as it is complicated or difficult for a layman such as project manager to use it (Peters & Zelewski, 2008; Qijia, Jian, Jiazhi, Kwok, & Ou, 2005). It is desirable to hide the abstractions of decision models via DSS implementation. Theoretically, in order to develop a robust and complete program, DSS researcher should define and analyses the requirement of the system with a series of DSS prototypes development (Tsui & Karam, 2007). The deliverables of these activities is the software specifications which have been reviewed by the system users. The main problem is identified where most DSS in the area of construction project management were lacking of software specifications to encapsulate the abstraction of highly complex decision models.

As mentioned in previous section, RE process consists of a few basic steps such as requirement elicitation, requirement analysis, requirement definitions, prototyping, requirement evaluation and requirement specifications. These fundamentals in DSS development are often ignored by literatures in construction. In this research we have assess 20 articles that has been published in the area of DSS in construction. Most paper were compiled from major database such as Proquest, Emerald, Elsevier and ITcon open access journal. As a result based, huge gaps were identified in RE process for solving construction project management problems. Table 1 illustrates the situations. Some process were not executed, partially done or fully done. This table has been simplified from various DSS applications that have been developed within the infrastructure life cycle i.e. planning, design, procurement, operation and maintenance

Table 1. *The Current State in RE Process for Construction Project Management Problems*

Project Mgmt. Phase	Typical examples of construction problems	Requirement Engineering in Decision Support Systems										
		Req. Elicitation	Software Requirement Analysis				Req. Def.	Prototyping		DSS Evaluation		Soft. Spec
			Conceptual Model	Decision Models								
				AHP /ANP	TOPSIS	Other		Data Mgmt	Data Comm.	Utility	Usability	
Planning	Supplier selection (Boer, Wegen, & Telgen, 1998), consultant selection (Al-Besher, 1998), contractor selection (Sonmez, Holt, Yang, & Graham, 2002), project selection (Dey, 2006), project delivery selection (Al Khalil, 2002), equipment selection (Shapira & Goldenberg, 2005), personnel selection (Gungor, Serhadlioglu, & Kesen, 2009).	●		●	●	●		●	◗	●		
Design	DSS for design problem in construction (Tam, et al., 2006), A development of generic DSS model for construction management (Sarka, et al., 2008)	●				●						
Procurement	DSS for building project procurement (Kumaraswamy & Dissanayaka, 2001)	●				●		◗				
Operation & Maintenance	Preservation for civil infrastructure DSS (Shen & Grivas, 1996), DSS for Efficient Irrigation (Oad, et al., 2009)	●				●	●	◗		●		

Abbreviations: Mgmt.= Management; Req. = Requirement; Comm.=Communication Soft.= Software; Def.= Definition; Spec.= Specifications

Legends: ●= Fully execution; ◗ = Partial execution

5. DISCUSSIONS AND FUTURE WORKS

Sufficient software engineering efforts are vital to provide efficient and better software development. However, most literature overlooked the importance of software specification in construction. A number of studies show that systems fail due to an inadequate or insufficient understanding of the requirements they seek to address (van Lamsweerde, 2000). Furthermore, the amount of effort needed to fix these systems has been found to be very high (Johnson, 1995). RE process is dedicated to elicit, model, prototype, evaluate and specify the software requirements in a certain problem domain. However, most research in construction area does not exploit this aspect before proceed to the implementation phase of DSS. This scenario has led to the ignorance of many computing aspects.

Throughout the project life cycle, requirement elicitation phase seems to be dominance as an initial step in DSS development. Most literature mentions their approach on the elicitation process and it is well covered. However, in the requirement analysis phase, most study discards the conceptual model where none of them describe on how to leverage the software modelling aspect together with the decision modelling. It is vital for a newly system to design by mapping the decision model to its environment such as actor, data, business logic, constraint, etc. Only a few literatures were identified to describe the requirement definitions for their DSS. However, the designs were just an ad-hoc integration of data management and data communication features. There are no details discussed on the conceptual model or design. Furthermore, researchers were only interested to evaluate the utility of the decision model rather than DSS as a whole. As a result, another gap was identified where there are no usability inspection has been conducted. For these reasons, most study in construction was incapable to figure out their software specifications that can be useful to hide the complexities of decision models in DSS. By considering these motivations, the next sections strive to illustrate the state-of-art in RE process which can be useful to guide the development of DSS. This investigation is the strategy to explore in depth on the RE process which will be utilized in the development of DSS for Consultant Selection Problem in the later chapters.

At the moment, we are rapidly in the process of developing a program that so called consultDeSS. This software if a DSS based program that will be utilised to help client from public sector to select the best consultant for public sector infrastructure project. It is anticipated that the implementation process of this software will fill in the gaps and will follow a formal procedure of requirement engineering process

6. CONCLUSIONS

This paper explained the state-of-art in developing DSS from Software Engineering point of view. Research shows that many software programs failed due to lack of requirement. A requirement is necessary attribute in a *system*, a statement that identifies capabilities, characteristic, or quality factor of a system in order for it to have value and utility to a customer or user. Requirements are important because they provide the basis for all of the development work that follows. Once the requirements are set, developers initiate the other technical work: system design, development, testing, implementation, and operation. Therefore, RE process should be integrated in DSS development. Unfortunately, current practice of DSS in construction shows only little work that has been done to deploy RE in their research. Thus, this paper has demonstrated a huge gaps for RE process in construction DSS. It ranges from the inception of the construction project until operation and maintenance phase. It is anticipated that this paper may encourage for more construction project management studies to be done for this promising area.

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A REVIEW ON THERMAL PERFORMANCE OF ROOFING MATERIALS IN MALAYSIA

Ahmad Al Yacouby[1], Mohd Faris Khamidi[2], Muhd Fadhil Nuruddin[3], Arazi Idrus[4], Syed Ahmad Farhan[5] and Azrul Esfandy Razali[6]

[1,2,3,4,5]Civil Engineering Department, Universiti Teknologi PETRONAS, Bandar Seri Iskandar, 31750 Tronoh, Perak, Malaysia.

[6]Monier Sdn. Bhd., Lot 1908, Batu 7, Jalan Bukit Kemuning, 42450 Klang, Selangor, malaysia.

[1]aalyacouby@gmail.com

ABSTRACT: Houses and residential buildings in Malaysia are experiencing overheating of roof space due to inappropriate selection of roofing systems and materials. Consequently, the indoor thermal comfort of housing is badly affected, especially for low rise buildings where the roof is representing around 70% of heat gain. The thermal performance of a building is affected by the solar absorbance of roof, during a clear sky conditions up to 1kW/m^2 of radiation can be incidental on a roof surface and between 20% and 90% of this radiation is typically absorbed. This paper is intended to review the roofing materials and roof colors used in Malaysian building industry and determine whether they are suitable for the hot humid climate zone. The study aims to review the thermal performance of roofing materials in Malaysia. Roof colors on some existing buildings and home buyers preferred roof tiles' colors have been determined. A survey was conducted among home buyers in Malaysian to determine the most preferred roofing colors. The survey results suggest that blue and brown are the most preferred ones followed by red. The study also revealed that 70% of households in Malaysia are using air-conditioning. The observation survey on some existing buildings revealed that 90% of roof tiles are red followed by 10 % black color.

Keywords: cool roof, hot humid climate, Malaysia, SRI, thermal comfort, white roof.

1. INTRODUCTION

Residential buildings in Malaysia are subject to significant cooling requirements due to high intensity of heat transmission from building envelope, especially for low rise buildings where the roof is representing around 70% of heat gain (Vijaykumar et al., 2007). Accordingly, the selection of appropriate roofing materials with higher performance in solar energy rejection will be a good alternative solution to make indoor temperature more comfortable and reduce the increasing demand in air conditioning by the building sector. The results of observation survey conducted by Allen et al. (2008) revealed that concrete roof tiles are the most commonly used roofing finish in Malaysia as it represents 85 % followed by clay tiles and metal deck with 10% and 5% respectively. In another observation survey conducted by the authors of this paper from October 2010 to February 2011 on roof color for residential buildings within Universiti Tcknologi PETRONAS (UTP) campus, Ipoh, Tronoh, Seri Iskandar and Petaling Jaya in Malaysia suggests that 90% of roof tiles in residential buildings are red followed by black color roof tiles which represent around 10 %. White color roof tiles were found to be 0% in the surveyed areas. Hence, introducing alternative roofing materials with higher Solar Reflectance Index (SRI) became a top urgent necessity in Malaysia. SRI is a composed measure that accounts for surface's solar reflectance and emittance measured on a dimensionless scale from 0 to1 (Marceau, 2007).

2. BACKGROUND

2.1 Building Impact on the Environment

Houses and buildings impact the environment in several ways as a result of both their energy use and material consumption. The emission of greenhouse gases and the creation of heat islands in urban areas are among their major impacts. Therefore, the adoption of building design strategies and construction materials that address these issues will lead to huge difference to global warming and reductions in a building's overall environmental impact. Further, the use of mechanical ventilation and air conditioners in buildings is arising two major problems as shown in Figure 1. First, air conditioning system is consuming a lot of energy and consequently increasing the households in the utility bills. Second, the electricity generation from fossil fuels is increasing the greenhouse gases concentration in the atmosphere which affects the global warming and climate changes. This paper is intended to review the performance of roofing materials used in Malaysia in terms of solar heat rejection.

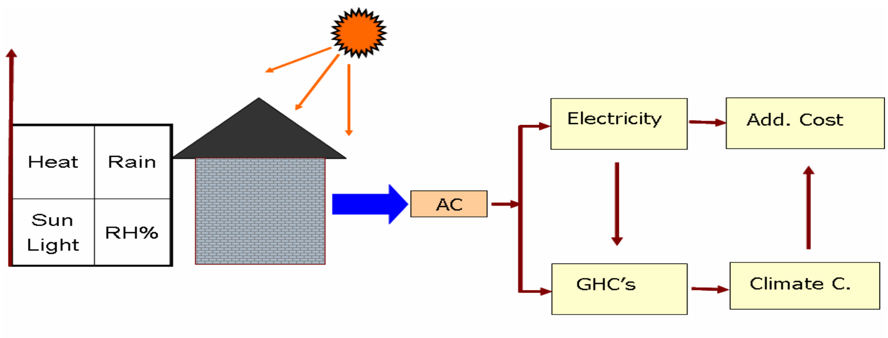

Figure 1. *Cause and Effect Diagram*

2.2 Building Envelope Heat Gain

The concept of using cool roof in residential, commercial and industrial buildings is growing in the last decade as one of the most cost effective solutions to mitigate the global warming issue and reduce the greenhouse gases (GHG's) emittence resulting from burning of fossil fuels in electricity generation. The thermal performance of a building is affected by the solar absorbance of roof. During a clear sky conditions up to about 1kW/m^2 of radiation can be incidental on a roof surface and between 20% and 90% of this radiation is typically absorbed (Suehrcke, H., et al., 2008). Heat entering to the houses and low rise building's structure is the major cause of discomfort in hot humid climate zone countries. The building roof is about 50-70% of the total heat entry of rooms as shown in Figure 2 (Vijaykumar et al., 2007). Further, In May 2009, Dr. Steven Chu Secretary of Energy " United States Department of Energy" advised that painting roofs white to reflect sunlight can make a huge difference to global warming (Mark, 2009). Hence, in order to meet challenges in mitigating global warming and climate change, the use of light color roofing materials with higher SRI would be highly recommended in Malaysia, as roof is the most important part of the building envelope which is exposed directly to sun light during the day.

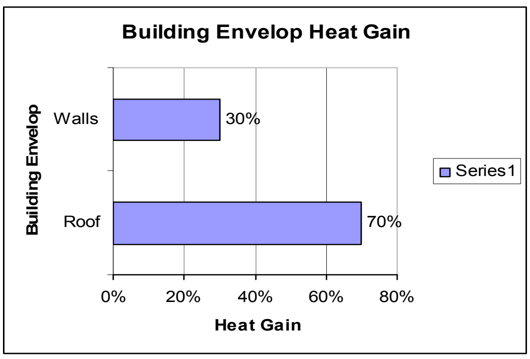

***Figure 2**. Low Rise Building Envelope Heat Gain*

2.3 Review of Some Existing Literature

The review of some existing literature has revealed that several successful researches on solar reflectance and heat transmittance of different materials have been conducted during the last decade. For instance, a research done by Lawrence Berkeley National Laboratory in 1997 revealed that a whiter roof results in savings of $ 51 per year per 1000ft^2 roof area of air conditioned buildings (Konopacki et al., 1997). Experiments in California and Florida have produced summer direct cooling energy savings of 10-70% (Sara and Akbari, 1997). On the other hand Suehrcke *et al.*(2008) using numerical simulation and deriving equations suggested that in hot climates a significant reduction in downward heat flow can be achieved by using a light or reflective roof color instead of a dark one.

The benefits of using light color roofing materials have been noted by Jayasinghe *et al*. (2003), Levinson *et al*. (2005, 2010), Filho et al., (2010) and Bryan and Roth (2010). Furthermore, Levinson *et al.* (2007) have demonstrated in their research that non white near-infrared-reflective architectural coating can be applied in-situ to concrete or clay tile roofs to reduce temperature. The study suggests that under typical condition e.g. 1 KW/m^2 summer afternoon insolation, R-11 attic insulation, no radiant barrier, and 0.3 reduction in solar absorptance, absolute reduction of roof surface temperature, attic air temperature, and ceiling heat flux were about 12K, 6.2K and 3.7W/m^2 respectively. In the same study Levinson *et al.* (2007) reported that for a typical 139m^2 houses with R-11 attic insulation and no radiant barrier, reducing the roof absorptance by 0.3 revealed whole house peak energy saving of 230W in Fresno, 210W in San Bernardino and 210W in San Diego. Detailed explanation on how does cool roofing help is also provided in Cool Roof Design Brief (2006). The above review of the literature raises the following questions:

- Light color roofing materials reflect more sunlight and improve the indoor thermal comfort. Is this technique applied in Malaysian construction industry?
- What is the most preferred roofing color by home buyers in Malaysia?
- Is the necessary information related to benefits of using white color roof tiles in Malaysian building industry available?
- More studies are needed to address the benefits of cool roofs in tropical climate zone where the average daily temperature is very hot all over the year, and experiencing high level of relative humidity. So what are the challenges facing the use of white roofs in Malaysia?

3. METHODOLOGY

In order to answer the questions mentioned above, this paper provides a systematic review and evaluation of current roofing materials and roofing colors used in Malaysia. An observation survey was conducted to determine the roof colors on some exhibiting buildings. Another survey was conducted to determine the most preferred roofing color by home buyers in Malaysia. The usage of air conditioning in households was also covered in the survey. The survey results have been analyzed and compared with the global trends of white roof techniques introduced to mitigate the global warming and climate change. Finally, suitability of currently used roofing color for hot humid Malaysian climate was also discussed in this study.

4. COMMONLY USED ROOFING MATERIALS IN MALAYSIA

Concrete, clay, and metal sheets are the roofing material types that are available in the market and widely used in hot humid climate zone; especially in Malaysia. The decision on roofing materials are normally made according to home buyer's requirements and climatic conditions. The budget also plays an important role in selecting the type of roof tiles. However, the roof material's ability to reject solar energy is not a governing factor during the selection of roof color. This is due to lack of information on the significant influence of roof color on thermal heat gain. As depicted in Table 1. below, an observation survey was conducted by Allen et al.,(2008) from December 2006 to March 2007 on roofing systems and materials for residential buildings within the state of Selangore and Wilayah Persekutuan Kuala Lumpur in Malaysia. The study revealed that 85% of residential uildings have concrete roof tiles, followed by 10% clay and 5% metal deck sheets used mainly in bungalow building types.

Table 1. *Observation Survey on Roofing Materials in Malaysia. Source Allen et al. (2008)*

Building Types	*Concrete tiles*	*Clay tiles*	*Metal Deck*
Terrace Apartment	45%	2.5%	-
Semidetached	20%	2.5%	-
Cluster house	2.5%	-	-
Bungalow	17.5%	5%	5%
Total	85%	10%	5%

5. HEAT TRANSFER

a. Modes of Heat Transfer

Where there is a temperature difference between two places heat tends to flow from the higher temperature to the lower and the heat transmission can occur in three ways; (1) conduction, (2) convection, and (3) radiation (Esmond, 1999) as described below:

1) Conduction: Heat transfer due to temperature difference in a material (but it most often associated with solids).

$$q''_x = k\frac{T_1 - T_2}{L} = \frac{T_1 - T_2}{R_{Mat}} \qquad (1)$$

2) Convection: Energy transfer from a solid surface due to a fluid (liquid or gas) in motion.

$$q'' = h_c(T_s - T_\infty) \qquad (2)$$

3) Radiation: Energy transferred by electromagnetic waves as photons (Potter and Scott, 2004).

$$q''_{rad} = \varepsilon\delta(T^4 - T^4_{sur}) \qquad (3)$$

Often the heat transfer is denoted as in equation 4 and has unit of W/m^2.

$$q'' = \frac{Q}{A} \qquad (4)$$

The following terms are normally used to rate the materials insulation performance of building envelopes.

- The thermal resistance (R) is a measure of a material's ability to resist heat transfer. It is the direct measure of its resistance to transfer energy or heat. R values are expressed using the metric units (m^2.K/W).
- Thermal conductivity (K) is defined as the ability of material to conduct heat; its unit is (W/m. K).
- U-Value is defined as a measure of the rate of heat loss or gain through a material or assembly, it is the inverse of R, therefore, *U =1/R* and its unit is W/(m^2.K)

b. Solar Reflectance Index (SRI)

The Solar Reflectance Index (SRI) is defined as the ability of material to reject solar energy; it is a composed measure that accounts for surface's solar reflectance and emittance (Marceau et al., 2007). The SRI is the fraction of the sun light that a surface reflects, sun light which is not reflected is absorbed as a heat (Bryan and Kurt, 2010). Solar reflectance is measured on a scale of 0 to 1. For example a surface that reflects 66% of sun light has a solar reflectance of 0.66. Most dark roof material reflects 5% to 20% on incoming sunlight, while light colored roof materials typically reflects 55% to 90% (Bryan and Kurt, 2010). Solar reflectance has the biggest effect on keeping roof cool in the sun (Carolyn, 2008). Comparative solar heating properties for some common materials for steep roofing are provided in Table 2 (Edward, A., and Joseph, I., 2009)

Table 2. *Solar Heating Properties for Some Roofing Materials- Steep Roofing*

Data source [Edward, A., and Joseph, I., 2009]

Material Description	Solar Reflectance	Thermal emissivity	Solar Reflectance Index (SRI)	Roof Surface Temperature Rise
Metal Panel with Cool white coating	0.70	0.85	85	25°F(1°C)
Aluminum Uncoated	0.60	0.05	40	56°F(31°C)
Red clay tile	0.35	0.90	35	60°F(33°C)
Metal Panel with cool color coating.	0.30	0.85	30	66°F(37°C)
Unpainted concrete tiles	0.25	0.90	25	70°F(39°C)

Further, another survey was conducted by the authors to determine the home buyers' most preferred roofing color in Malaysian. The survey was conducted during Malaysia Property Expo (MAPEX 2011). From 500 target population 204 participants have responded to the questionnaires survey. The preliminary analysis of study revealed that 32% of home buyers in Malaysia prefer blue color roof tiles, 20% brawn color followed by 29% red, 15% green, 7% yellow and 5% for grey and white as shown in Figure 3.

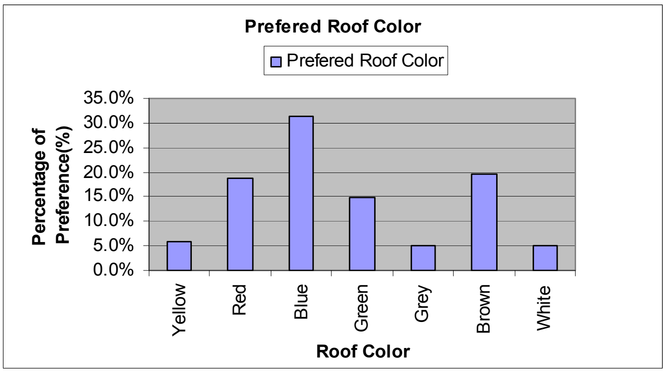

Figure 3. *Preferred Roofing Colors in Malaysia*

The majority of home buyers who decided to select dark colors roof tiles as their preferred color justified that this is because white color tiles are quite difficult for maintenance. As white color roof tiles tend to discoloration and fungi growth associated with monsoon and high level of humidity that characterize the Malaysian climate zone. However, this issue can be resolved by developing stainless light color roofing materials. In addition when we compare the preferred roof colors summarized in Figure 3 with some solar heating properties of roofing materials in Table 2, we can conclude that most of dark color roof tiles are falling within SRI value below 40. This indicates that the roofing colors used in Malaysia are not the optimum in terms of solar reflectance as most of them are dark materials. This means the solar performance of roofing materials in Malaysia are not in line with the global trends for light color roofing system implemented in USA and Europe in order to mitigate the global warming.

c. Thermal Comfort

Thermal comfort can be defined as the range of climatic condition considered comfortable and acceptable, and in accordance with ANSI/ASHRAE Standard, (2005) the thermal comfort is state of mind that expresses satisfaction with the surrounding environment. Thermal comfort is very important to many work-related factors. It can affect the distraction levels of the workers, and in turn affect their performance and productivity of their work. The thermal comfort might be affected by the (1) environmental (air and ambient) temperature, (2) humidity, (3) air speed, (4) clothing and metabolic rates. The thermal sensation of humans is mainly related to the thermal balance of the body as a whole. Thermal balance exists when the internal heat production in the body is equal to the loss of heat to the environment. PMV index (Predicted Mean Vote) and PPD index (Predicted Percentage Dissatisfied) are the most widely used indices in moderate thermal environment. Examples of recent studies in thermal comfort include Jorn (2005), Roonak et al. (2009) and Cao et al. (2010).

6. ENERGY CONSUMPTION

Electricity is a necessity factor for developing and civilization of nations, but the energy generation from fossil fuels has two major effects. On one hand, the fossil fuels reserves are depleting, for instance, in 2004, Malaysia was ranked 24th in terms of worlds oil reserves and 13th of natural gas, and in January 2007, PETRONAS reported that oil and gas reserves in Malaysia amounted to 20.18 billion barrels equivalent. In accordance with the Malaysian

government estimates; at the current production rate, Malaysia will be able to produce oil and gas up to 18 years and 35 years respectively (Cha and Oh, 2010) with respect to 2004 level. On the other hand, the process of electricity generation using fossil fuels is damaging the ecosystems by producing greenhouse gases (GHG's). Further, in accordance with the International Energy Agency's CO_2 emission report published in 2010 (International Energy Agency, 2010). It is obviously evident that CO_2 emission is related directly to electricity consumption, with China being the biggest CO_2 emitter in the word with 6508.24 MT of CO_2 in 2008, followed by USA with 5595.92 MT of CO_2.

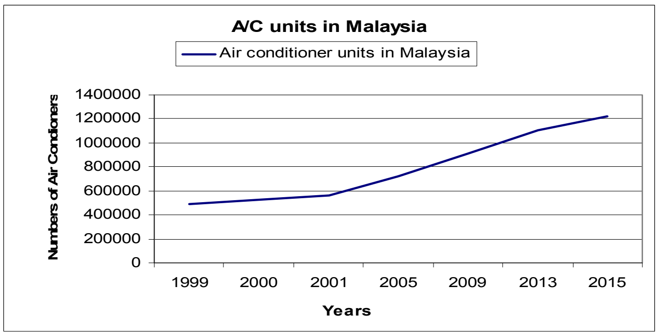

Figure 4. *Air conditioner Units in Residential Building in Malaysia, Data source: Saidur. R., et al., (2007)*

Moreover, Ong et al. (2010) found that Malaysia's energy sector is still heavily depending on non renewable fuels such as crude oil and natural gas and coal, and these non renewable fuels are gradually depleting and contribute huge amount of greenhouse gas emissions. In addition, Saidur et al. (2007) in their study on energy and associated greenhouse gas emissions from household appliances in Malaysia for estimated period of 17 years (1999-2015) suggested that refrigerator–freezer is the major energy consuming appliances followed by air conditioners. The number of air conditioner units in Malaysia was 493,082 in 1999 while the projected number of air conditioners in 2015 is expected to reach 1,271,746 units (Saidur et al. ,2007), an increase of 68% compared to 1999 level as shown in Figure 4. Further, in a recent study conduct by the others of this paper during MAPEX 2011 in Kuala Lumpur to determine the percentage of air condoning usage among Malaysian households. 500 questioners have been distributed to home buyers and 208 participants have responded. The preliminary analysis of study suggests that 75.5% of Malaysians households are using air conditioning, while about 24.5 % are depending on natural ventilation or fans.

7. CONCLUSIONS

The thermal performance of a building is affected by the solar absorptance of roof. During a clear sky conditions up to about 1kW/m^2 of radiation can be incidental on a roof surface and between 20% and 90% of this radiation is typically absorbed. Results of observation survey on roof colors suggest that 90% of roof tiles in low rise Malaysian residential buildings are red followed by 10% black color tiles. In addition 32% of home buyers in Malaysia prefer blue color roof tiles, 20% brown color followed by 29% red, 15% green, 7% yellow and 5% for grey and white. The preference of dark color roof tiles by home buyers is mainly due to lack of information as well as the absence of systematic researches which address the benefits of light colors roofing materials in Malaysia. The maintenance difficulties associated with white color roofing system could also be another reason. Therefore, in order to make a total shift from dark color roof tiles to light color roofing system in Malaysia, private initiatives from researchers together with governmental support to conduct research on Solar Reflectance Index (SRI) of roof tiles and developing stainless light color roof tiles will be essential to make possible a sustainable development future.

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ULTRA HIGH PERFORMANCE CONCRETE (UHPC) TECHNOLOGY FROM MATERIAL TO STRUCTURE: A REVIEW

Behzad Nematollahi[1], Raizal Saifulnaz M.R.[2] and Yen Lei Voo[3]

[1,2]University Putra Malaysia, Serdang, Malaysia.

[3]Dura Technology Sdn. Bhd., Perak, Malaysia.

[1]behcom62@gmail.com , [2]raizal@eng.upm.edu.my , [3]vooyenlei@dura.com.my

ABSTRACT: In the 21st century, one of the significant breakthroughs in concrete technology is the development of Ultra High Performance Concrete (UHPC) with compressive strength over 150 MPa and flexural strength over 30 MPa; and enhanced durability similar to natural rocks. In brief, UHPC is a cementitious based composite material that consists of the distinctive characteristics of the ultra-high performance concrete and high tensile strength steel fibers. UHPC is a sustainable construction material with considerable amount of durability, ductility and tensile capacity. UHPC is mostly appropriate for the use in the fabrication of precast members in civil engineering, structural and architectural applications. This paper presents an overview of the material characteristics of a Malaysian blend of UHPC (i.e. Dura®), the principles of UHPC development, its mix design, its advantages, and its applications.

Keywords: ultra high performance concrete (UHPC), ductility, durability, sustainable construction material

1. INTRODUCTION

Since last two decades, astonishing advancements has been made in the field of concrete technology. One of the great break-through was the development of fiber reinforced reactive powder concrete (RPC), and more commonly known as the ultra-high performance concrete (UHPC). This advanced material was firstly introduced by Richard and Cheyrezy (1995) in the mid 1990's with compressive strength greater than 200 MPa and modulus of rupture of 25-50 MPa. Although vast progress in UHPC technology has been achieved in recent decades (Voo & Foster 2009; Graybeal 2006; Schmidt, Fehling & Geisenhanslüke 2004); however, its application in many developing countries is still in its infant stages. So far many researchers around the world have developed such concretes that could be categorized as UHPC such as Ductal®, Ceracem®, BSI®, Densit®, Ducon® and Dura®. Although there are differences among these types of UHPC; however, there are many overall similarities (Dura Technology Sdn. Bhd. n.d.).

2. MATERIAL CHARACTERISTICS OF UHPC

Table 1 (Voo & Foster 2010, p.170) shows the material characteristics of two commercials UHPC blends known as Ductal® and Dura® compared against Normal Strength Concrete (NSC), and High Performance Concrete (HPC).

The comparison shows that UHPC has higher mechanical properties over NSC and HSC in all disciplines (Voo & Foster 2010).

HPC is a high strength, ductile, and sustainable construction material formulated by combining Portland cement, silica fume, fine washed/sieved sand, superplasticizer, water, and steel fibers. UHPC is an extremely homogenous cementitious blend without using coarse aggregates that can attain compressive strength of 150 MPa and above (Voo & Poon 2009; Richard & Cheyrezy 1995).

Table 1. *Material characteristics of UHPCs compared to Normal Strength Concrete (NSC) and High Performance Concrete (HPC)*

Characteristics	Unit	Codes/ Standards	NSC	HPC	DURA	DUCTAL
Specific Density, □	kg/m^3	BS1881:Part 114-1983	2300	2400	2350 – 2450	2440 – 2550
Cylinder Compressive Strength, f_{cy}	MPa	AS1012.9-1999	20 – 50	50 – 100	120 – 160	123 – 210
Cube Compressive Strength, f_{cc}	MPa	BS6319: Part 2-1983	20 – 50	50 – 100	130 – 170	158 – 220
Creep Coefficient at 28 days, $□_{cc}$		AS1012.16-1996 ASTM C512	2 – 5	1 – 2	0.2 – 0.5	0.29 – 0.66
Post Cured Shrinkage	□□	AS1012.16-1996	1000 – 2000	500 – 1000	< 100	0
Modulus of Elasticity, E_o	GPa	BS1881:Part 121-1983	20 – 35	35 – 40	40 – 50	50 – 53
Poisson's Ratio, □			0.2	0.2	0.18 – 0.2	0.2
Split Cyl. Cracking Strength, f_t	MPa	BS:EN 12390-6-2000 ASTM C496	2 – 4	4 – 6	5 – 10	8.6 – 12.4
Split Cyl. Ultimate Strength, f_{sp}	MPa		2 – 4	4 – 6	10 – 18	18.3 – 26.5
Flexural 1st Cracking Strength, $f_{cr,4P}$	MPa	ASTM C1018-1997 (Four-Point Test on Un-notched Specimen)	2.5 – 4	4 – 8	8 – 9.3	9 – 9.7
Modulus of Rupture, $f_{cf,4P}$	MPa		2.5 – 4	4 – 8	18 – 35	40 – 50
Bending Fracture Energy, $G_{f,□=0.46mm}$	N/mm		< 0.1	< 0.2	1 – 2.5	N/A
Bending Fracture Energy, $G_{f,□=3.0mm}$	N/mm		< 0.1	< 0.2	10 – 20	N/A
Bending Fracture Energy, $G_{f,□=10mm}$	N/mm		< 0.1	< 0.2	15 – 30	N/A
Toughness Indexes I_5			1	1	4 – 6	5.3 – 6.2
Toughness Indexes I_{10}			1	1	10 – 15	11.8 – 14.4
Toughness Indexes I_{20}			1	1	20 – 35	25.9 – 32.8
Modulus of Rupture, $f_{cf,3P}$	MPa	JCI-S-002-2003 (3-Point Test on Notched Specimen)	2.5 – 4	4 – 8	18 – 35	40 – 50
Bending Fracture Energy, $G_{f,□=0.46mm}$	N/mm		<0.1	<0.2	1 – 2.5	N/A
Bending Fracture Energy, $G_{f,□=3.0mm}$	N/mm		<0.1	<0.2	10 – 20	N/A
Bending Fracture Energy, $G_{f,□=10mm}$	N/mm		<0.1	<0.2	15 – 30	N/A
Rapid Chloride Permeability	coulomb	ASTM C1202-2005	2000–4000	500– 1000	< 200	< 50
Chloride Diffusion Coefficient, D_c	mm^2/s	ASTM C1556-2004	4 – 8 x 10^{-6}	1 – 4 x 10^{-6}	0.05–0.1 x 10^{-6}	0.02 x 10^{-6}
Carbonation Depth	mm	BS:EN 14630-2003	5 – 15	1 – 2	< 0.1	< 0.5
Abrasion Resistance	mm	ASTM C944-1999	0.8 – 1.0	0.5 – 0.8	< 0.03	< 0.03
Water Absorption	%	BS1881:Part 122-1983	> 3	1.5 – 3.0	< 0.2	N/A
Initial Surface Absorption	ml/(m^{2}s)	BS1881:Part 208-1996	0.7/10 min 0.2 120min	0.1/10 min 0.05/120 min	< 0.02/10 min < 0.01/120min	N/A

↑ *refer to Voo & Foster 2010 for references of codes and standards*

3. STANDARD MIX DESIGN OF UHPC

Ordinary Portland cement, silica fume, fine aggregates, water, steel fibers and high-range water reducing agent are the components of UHPC. Table 2 (Voo & Foster 2010, p.171) demonstrates the standard UHPC mix design with 2% by volume of steel fibers. The high-range water reducing agent used is Polycarboxylate ether (PCE)-based superplasticizer and no recycled wash water shall be used in the mixture. Based on the Concrete Committee of Japanese Society of Civil Engineering recommendation for design and construction of UHPC structures (JSCE 2006), the steel fibers used in UHPC are needed to have a tensile strength over 2000 MPa. Besides, specimens or members made of UHPC are required to be steam cured for 48 hours at a temperature of 90˚C (Voo & Foster 2010).

***Table 2.** Mix design of standard UHPC (quantity in kg/m^3)*

Ingredient	Mass (kg/m3)
UHPC Premix	2100
Superplasticizer	40
Steel Fiber	157
Free Water	144
3% Moisture	30
Targeted W/B Ratio	0.15
Total Air Void	< 4%

4. PRINCIPLES OF UHPC DEVELOPMENT

UHPC is established on the principle that a material with a minimum of weaknesses such as micro-cracks and pore spaces shall be capable to reach a superior quantity of the potential ultimate load carrying capacity as defined by its component materials (Richard & Cheyrezy 1995).

UHPC is founded on the four principles that can be summarized as follows: 1- Optimized granular packing which improves homogeneity and cause ultra-dense matrix. 2- Extremely low water cement ratio (W/C< 0.15) which reduces amount of pores and capillary, pore sizes, concrete cancer issues e.g. carbonation, improves impermeability, and results in remarkable durability and strength. 3- Inclusion of very high strength micro-fibers which enhances tensile strength and ductility, improves impact and abrasive resistance, and bridge micro-crack more effectively. 4- Steam cured for long period of time which accelerates all early and drying shrinkage, improves overall material properties which cause volumetrically stability, minimal creep, and negligible shrinkage.

5. UHPC VS. CONVENTIONAL STEEL FIBER REINFORCED CONCRETE (CSFRC)

UHPC, with respect to general mechanical characteristics, is superior to Conventional Steel Fiber reinforced Concrete (CSFRC). The steel fibers used in CSFRC have tensile strengths usually up to 1000 MPa and the fiber fracture may take place during cracking; whereas the steel fibers used in UHPC have tensile strengths above 2300 MPa, hence fracturing of the fibers will never occur, which will result in ensuring that the UHPC matrix keeps high ductility during cracking. Due to its greater fracture properties, UHPC demonstrates “Strain-Hardening” and “Displacement Hardening” behaviors during tensile stresses, whereas such behavior is not revealed in CSFRC (Dura Technology Sdn. Bhd. n.d.).

6. STEEL FIBER IN UHPC

Even though UHPC demonstrates considerably improved compressive strength and lower porosity; however, UHPC matrices tend to be fragile. Thus, micro steel fibers with various dimensions and mechanical properties are commonly used in UHPC at different concrete volume percentage to develop tensile and flexural strength, resistance to impact or toughness, cracking control, and modify the mode of collapse by escalating post cracking ductility (Victor, Lawrence & Kimberly 2009).

Different type and volume of steel fibers are commonly used in UHPC which may result in different mechanical properties in which l_f is the total length of the fiber, d_f the diameter of the fiber, α_f the aspect ratio of the fiber ($\alpha_f = l_f / d_f$), and σ_{fu} is the ultimate tensile strength of the fiber. For instance l_f, d_f, α_f, and σ_{fu} may vary from 13 to 35 mm, 0.20 to 0.50 mm, 60 to 80, 1200 to 2500 MPa respectively (Voo & Foster 2009).

7. RUSTING OF STEEL FIBERS IN UHPC

Generally, some rust staining may come into view on the outer surface or skin of the structural elements due to oxidation of steel fibers that lie right at the surface of the concrete. Corrosion of steel fibers at the surface or skin of the concrete is not structurally considerable. Experimental tests have shown that even in an aggressive environment with high potential corrosion, the steel fibers will not rust further than a depth of 2 mm from the outer surface or skin of the concrete, because the matrix of

UHPC is at least 20 times more impermeable in comparison to conventional concrete. Although, the steel fibers at the surface of the concrete will rust and expand 30% of its original volume; however, due to the small size of the steel fibers, this increased volume due to the expansion will not produce adequate internal expansive stress to spall the contiguous ultra high strength concrete. Since rusting, i.e. iron oxide is water insoluble and does not conduct electricity; therefore rusting of steel fibers will stop further than the depth of the surface rust zone and will not develop further because oxygen, moisture and chloride ions are not able to penetrate deeper into the concrete. Hence, at serviceability conditions, the possibility of rusting of the internal steel fibers is insignificant (Dura Technology Sdn. Bhd. n.d.).

8. ADVANTAGES OF USING UHPC

Based on extraordinary characteristics of UHPC and as a new generation of sustainable construction material it offers remarkable benefits. For instance, using UHPC will result in elimination of the use of conventional steel reinforcement bars and stirrups which needs a considerable amount of human labors, supervision and quality control. Therefore, in terms of construction management, the construction time and labor costs may also be drastically reduced, which will result in saving immediate project costs. In addition, it will lead to save considerable maintenance costs and also long-term service costs (Voo & Foster 2009).

Secondly, with using UHPC designers will have flexibility and freedom of innovative design which enables design of more interesting and aesthetically expressive concrete structures due to the absence of conventional reinforcement and the self-heating potential after cracking of UHPC. Also, UHPC elements are ultra-light weight, typically by a factor of two, in comparison to conventional reinforced concrete or prestressed concrete elements, which leads the ease in handling, transportation and installation. As a consequence, it will lead to additional cost savings and develop safety margins in the construction procedures. Besides, UHPC offers excellent durability, with high impermeability against physical and chemical aggressive environment, high resistance to corrosion, abrasion and impact loads which will result longer service-life, minor maintenance and contribute to the sustainable development concept (Dura Technology Sdn. Bhd. n.d.).

In terms of structural design, concerning long-term behavior, UHPC demonstrates remarkable characteristics such as low creep and shrinkage which enables the elimination of most of the design considerations associated with time dependent strains (Voo & Foster 2009; Cheyrezy 1999; Rechard & Cheyrezy 1995)

9. FEATURES OF UHPC PRODUCTS

Based on the material characteristics of UHPC, products made from UHPC are supposed to have the following features:

9.1 Aesthetic

In terms of aesthetic, based on the elimination of coarse aggregates, superior homogeneity and granular packing distribution of the UHPC matrix, products made from UHPC are capable of achieving extraordinary finished surface in comparison to conventional concrete. Painting or coating is also not essential as the natural fair-face concrete finish will maintain its properties over time.

9.2 Workability

In regard to workability, with using UHPC, a water-binder ratio of 0.13 is possible. In order to have adequate flowability, an ultra high-range superplasticizer is used, which leads UHPC products behave like self-consolidating concrete. Some experts even refer to this as "liquid stone". This characteristic enables the opportunity of casting very slender elements.

9.3 Durability

Concerning durability, UHPC is a very impenetrable material with very small and discontinuous pores which avoid the movement of vapors within the concrete matrix. In another words, impermeability of

UHPC is less than one-tenth of conventional concrete. This chemical and physical composition results in a dense matrix with corrosion resistance greater than granite. Therefore, UHPC products are highly resistant to weathering, chemical attack, hydraulic scouring and salt-water entrance. Hence, coating or painting is not essential.

9.4 Ductility

Regarding ductility, the use of ultra-high strength steel fibers gives UHPC the flexural toughness property equivalent to structural steel. The flexural strength at first crack varies from 10 to 12 MPa, while ultimate flexural strength of 15 to 35 MPa can be reached. Experimental studies illustrate that the crack widths are not more than 0.05mm and 0.5mm at serviceability and ultimate limit states respectively. Besides, no fiber fracture has been observed at any stage of cracking.

9.5 Engineering maximization

In the matter of engineering maximization, unique material characteristics of UHPC facilitate the possibility of designing innovative structures, such as creating structures with longer span, and irregular shapes which are thin and slender, light and with more pleasing view. Experts believe that with using UHPC technology in well designed and planned projects not only would lead to exceptional service life for 200 years or more, also would bring at least 70% discounts on the total life-cycle costs.

9.6 Sustainability

On the subject of sustainability, UHPC technology is a green technology which supports the visionary of sustainable development. In another words, using UHPC enables using less cement in the concrete and using less concrete in the products. According to shocking reports of many scientists worldwide, Global Warming is the most destructive problem which people encounter nowadays. Hence, with using UHPC some preliminary saving in terms of cost, lower energy consumption and waste emissions can be achieved in comparison to conventional concrete. In addition, in terms of life-cycle, its sustainability is even more considerable than others types of concrete (Dura Technology Sdn. Bhd. n.d.).

10. COMMERCIAL UHPC BLENDS

Within the last two decades, significant research projects had been conducted by the academics and engineers around the world in order to industrialize UHPC technology as an alternative construction material that supports the concept of sustainable development (Voo & Poon 2009). As a consequence, so far there are five commercial UHPC blends in the world as shown in table 3

***Table 3**. Commercial UHPC blends*

UHPC Blends	Established Date	Established Place
DensitD®	Developed in 1960’s	Denmark (Grout)
Ductal®	mid1990’s-current	France, license to Canada, Australia, Japan, Europe, USA
Ceracem®	2000-current	France, Swiss
Ducon®	2004-current	Germany
Dura®	2006-current	100% Malaysian owned

11. Evolution and application of UHPC

The first ultra high performance cementitious mortar recognized as Reactive Powder Concrete (RPC) was founded in the mid 1990’s, with concrete compressive strengths greater than 200 MPa. The first RPC filled steel tube composite footbridge in the world was built at Sherbrooke in Canada in 1997 (Adeline et al. 1998). Also, the first fully RPC footbridge spanning 120 meters in the world was constructed in Seoul, South Korea in 2002 (Deem 2002). In Australia, the first RPC highway bridge at

Shepherds Gully Creek was designed by VSL (Australia), and was inaugurated to traffic in 2005 (Cavill & Chirgwin 2003) (Voo & Foster 2009). Figure 1 (Dura Technology Sdn. Bhd. n.d.) presents a schematic drawing showing the evolution of UHPC technology.

Accomplished projects using UHPC such as Sherbrooke footbridge (in Canada in 1997), Seonyu footbridge (in South Korea in 2002), Bourg-Les-Valence Bridge (in France), and Shepherds Gully Creek Bridge (in Australia in 2005) emphasize the possible applications of UHPC in civil and infra-structural projects (Voo & Foster 2009).

In terms of durability, low and non-continuous porosity of UHPC with minimum mass transfer will result in almost negligible penetration of liquid, gas or radioactive elements. Improved durability and abrasion resistance of UHPC in comparison to conventional and high strength concrete makes UHPC as a unique material suitable for the storage of nuclear waste or other dangerous materials (Voo & Foster 2009; Matte & Moranville 1999; Torrenti et al. 1996; Rechard & Cheyrezy 1995)

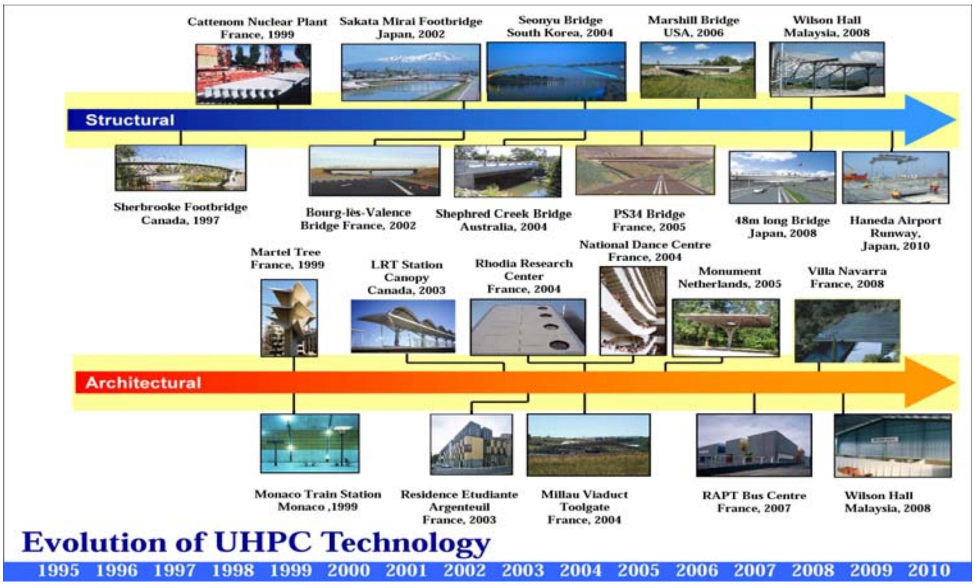

Figure 1. *Evolution of UHPC Technology*

The exceptional properties of UHPC make it a distinctive sustainable material capable to use in a broad variety of structural and non-structural applications because of its superior strength and high durability in terms of corrosion protection in aggressive environments (Voo & Foster 2009; Matte & Moranville 1999; Roux et al. 1996; Torrenti et al. 1996; Rechard & Cheyrezy 1995).

UHPC as a new generation of ultra-high performance sustainable construction material can be used in the fabrication of precast elements for civil engineering, structural and architectural applications (Voo and Poon, 2009). The application of UHPC can be classified into the following categories: 1-Infra-structural application such as ultra-light and slender section for pedestrian and highway bridges. 2-Impact resistance structures such as security panel against impact, seismic and blast loads, crash safety barriers. 3-Prestressed elements such as piles, culverts, retaining walls, pipes, safety vaults and etc. 4-Building application such as ultra-slender beam, slab and column systems, long span floors and roofs. 5-Other application such as architectural features, acoustic barrier, structural walls, marine/sea walls and decks, anchorage plates, leave in-place forms/moulds, container, storage tanks (Dura Technology Sdn. Bhd. n.d.).

12. CONCLUSIONS

During the last two decades incredible development had been made in concrete technology. One of the major progresses in 21st century is the development of a new generation of highly cementitious based composite material known as Ultra-High Performance Concrete (UHPC) with compressive strength over 150 MPa and flexural strength over 30 MPa and remarkably improvement in durability similar to natural rocks. Even though lots of research regarding the material characteristics and application of UHPC has been done so far; however, there is a need for future research and development in order to lead to the ordinary application of UHPC based on comprehensive technical regulations

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RELATIONSHIP BETWEEN ROADSIDE DEVELOPMENTS AND ROAD TRAFFIC ACCIDENTS

Intan Suhana Mohd Razelan[1], Adnan Zulkiple[2] and Azlina Ismail[3]

[1,2,3]Faculty of Civil Engineering and Earth Resources, Universiti Malaysia Pahang, 26300, Lebuhraya Tun Razak, Pahang, Malaysia

[1]intan@ump.edu.my, [2]adnanz@ump.edu.my , [3]azlinai@ump.edu.my

ABSTRACT: Studies on the causes of traffic accidents that focus on the driver's carelessness, road physical conditions and vehicle faultiness had been carried out extensively by the local authorities and researchers. While accepting human behaviours as one of the traffic accidents causes, other factor such as the mismanaged of roadside area should not be set aside. This study intends to explore the relationship between rapid roadside development growth and number of accident cases along Federal Road 2, Jalan Kuantan-Gambang which is one of the gateways to Kuantan town. The research area covers of 24 kilometer stretch road where most of the roadside areas are residential areas, business areas and educational institutions. It is dual-carriageway roads which experiences a heavy traffic movement and consequently has recorded a high number of accidents from year to year. Based on these analyses, models are developed to represent the relationship between roadside developments and accident cases. In conclusion, it is proven that roadside development does have a positive relationship with an increased in number of accident cases along the studied area.

Keywords: road safety, Federal Road, roadside development

1. INTRODUCTION

In year 2004, a total of 6,223 road fatalities recorded in Malaysia were caused by accidents part of which Pahang state recorded 445 cases. The major contributor was from cars and motorcycles with 69.8% out of total vehicles involved in road accidents.

Every year road and transportation agencies as well as local authorities have conducted a lot of awareness programs which involved the analysis of roadside feature as one of the methods in preventing accidents. As a result, from year 1990 to 2004, the number of accidents with fatalities has decreased from 4.6% in year 1990 to only 1.8% in year 2004.

In Pahang State, total road accident rates showed a tremendous increase with 5,140 accidents cases in year 1994 to 13,349 cases in year 2004. This 160% increase mainly involved cars and motorcycles. These accidents normally occurred at certain sections of the road and this trend remained the same for five consecutive years starting from the year 2000 to 2005.

In most cases, accidents occurred due to a number of reasons such as drivers' fault, vehicle mechanical failure, road physical condition, the environment and weather condition (Kowtanapanich, 2005). About 90% of the accidents were blamed on drivers' fault which is an easy way out of the traumatic problem. The balance of 10% was shared by vehicle mechanical failure, road physical condition, the environment and weather condition. Vehicle manufacturers in particular those producing cars and motorcycles were developing better vehicle with improved safety features such as antilock brake system (ABS) that could minimized skidding incidents and safety bags that could prevent fatalities during accidents.

The Government was also committed to focus on enforcing traffic management regulations such as reduction of vehicle operating speed by 10 km/h on the expressways and the highways. In addition to that, frequent surveillance by traffic police and road transport department officers was also conducted in the efforts to prevent reckless driving on the road. Road physical conditions were maintained through scheduled pavement resurfacing and improvement of the hazardous road sections, for example by increasing the sight distance on the sharp corners and geometrically steep gradients road sections.

Although various awareness programs have been introduced, however road accidents cases remain the same and surprisingly, the number of accident cases kept on increasing from year to year. This trend was believed to be contributed by other factors and one of them is an inefficient management of roadside area.

Combining the State of Pahang's accidents database obtained from Malaysian Royal Police (PDRM) and Kuantan development data from Kuantan Municipal Council (MPK), an initial study to examine the relationship between roadside development and accident rates focusing on Jalan Kuantan-Gambang was initiated.

2. ISSUES ON UNCONTROLLED ROADSIDE DEVELOPMENT

Roads in Malaysia can be classified according to its functions or jurisdictions. By functions, roads can be classified as primary, secondary and minor roads. By jurisdiction, roads can be classified as Toll Expressway, Highway, Federal, State and Municipal roads. Roads may also be classified as rural and urban. Rural roads are roads outside the Municipality limits or if it is located 5 km away from the Municipality limits (A Guide on Geometric Design of Road, REAM, 2006).

Otherwise, it is defined as urban roads that comprise of all roads within the gazette of the Municipality having a population of 10,000 and above. Accessibility from/to the roads is through proper channel such as junctions. There are two categories of access junction which is major and minor access. A major access is a properly designed junction catering all required traffic turning movements. Often these types of junction are signalized particularly if it is located along a Federal Routes.

A minor access in form of ingress/egress and it is the lowest category of junctions that prohibit all types of right turning movements. As such, traffic will have to travel for about few kilometers before reaching the nearest u-turns or the signalized junction where u-turns is normally provided. This form of access causes the local traffic in particular motorcyclists to disobey the road's regulations since they can make the u-turn at undesignated location which is convenience to them.

Moreover, the motorcyclists are seem to go against the traffic flow (run-off the roadway) on the unpaved pedestrian walkway and more daring they use the outer lane endangering the other vehicle that flow in the opposite direction. This behaviour is the likely cause of run-off roadway accident as indicated by Lee and Mannering (2000). Run-off roadway accident severity is a complex interaction of the vehicle with roadside features such as the guardrails, fixed objects, signboards, trees and utility poles along the road (Lee and Mannering, 2000).

3. METHODOLOGY

3.1 Background of Study Area

The proposed study area, Jalan Kuantan-Gambang was chosen as the study area after an intensive investigation on the road features were made in every possible aspect particularly on the level of safety of the route. Jalan Kuantan- Gambang that is forming part of Federal Route 2, falls under the category of rural and primary road when it was first constructed. As a primary road it functions as a major road, forms the backbone of the road network for the State of Pahang.

Before the opening of the East Coast Expressway, Jalan Kuantan-Gambang was the only route to cater for Kuantan – Gambang corridor. It was upgraded from single lane road to dual 2-lane carriageway in mid 90's. Since then, many residential developments have taken place along with business, shops, schools and shopping zones. The traffic flow also increased gradually with the increase in population along the roadside. At present, with similar road features in terms of speed limits and geometric design, the road has now becoming less effective to cater for the current needs of the road users. Figure 1 below shows the location of study area within Kuantan District.

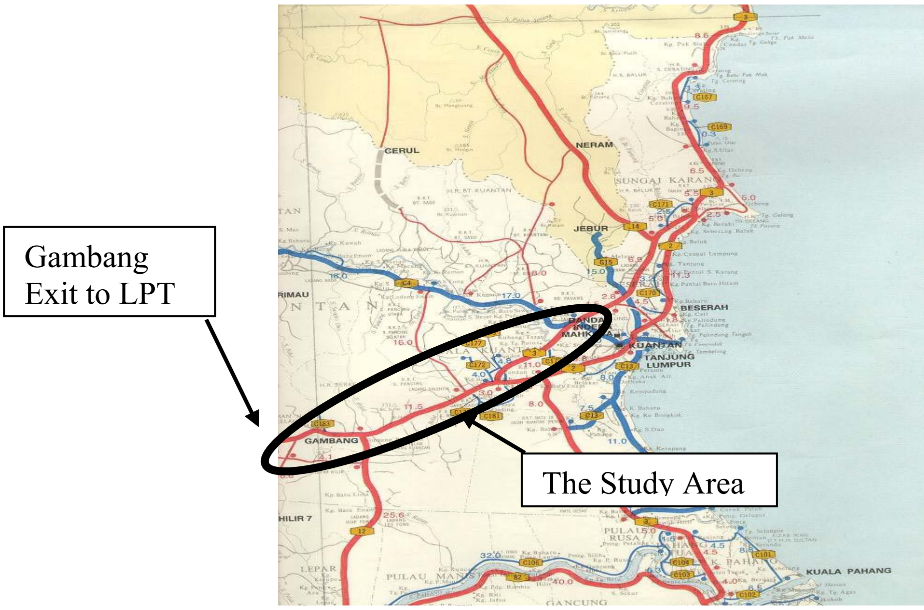

Figure 1. *Location of Study Area (Kuantan-Gambang road)*

As shown in Figure 1, the study area which is part of Jalan Kuantan-Gambang consists of 24 kilometre long 2-lane dual carriageway road connecting the 3-legged signalised junction at Jalan Wong Ah Jang and the 4-legged signalised junction at Jalan Muadzam Shah / access road to East Coast Expressway. This road is also connected to Jalan Kuantan-Pekan at KM 8 and Jalan Kuantan Kemaman at KM 14. As shows in figure 2, it is clearly explains that the traffic flow catered by Jalan Kuantan-Gambang is relatively heavy and becoming much heavier in festive seasons.

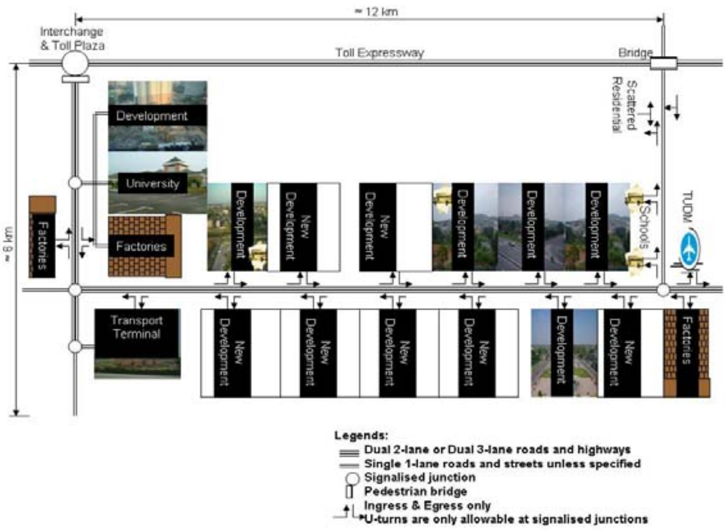

Figure 2. *Illustration of the existing Traffic Management along Federal Route 2 (Jalan Kuantan - Gambang)*

3.2 Road Accident Data

Road accident data consisted details on the number of traffic accidents as reported to PDRM who kept the state-wide databases on traffic accidents occurred at all major roads in Pahang. The police report contains information on the number of traffic accidents happened on these roads for the whole year.

Number of traffic accidents were defined as all type of accidents occurred along Jalan Kuantan-Gambang that caused death, injuries and damages (properties and facilities). In certain aspects, the number of traffic accidents was grouped in year to year basis and in other instance the number of traffic accidents was classified according to section by section basis. After all traffic accidents data were collected, the data were interpreted according to the needs and objectives of this study. For the purpose of this study, only relevant data that suited the needs of the study were extracted from the records.

The analysis considered only:-

a. Accidents that happened from KM 3 to KM 27 of the study route.

b. Accidents happened on the residential area streets, roundabout areas, flyovers, factory areas and at the alternative route were not counted as the valid number of traffic accidents.

c. Police records without specifying the accident location were discarded from the datasets.

3.3 Roadside Features Data

Roadside features data in this study was categorised as datasets that was indirectly related to the intensity of developments along the study route. The inventory of roadside facilities available within the roadside areas (edge of roadway to the beginning of adjacent land) was carried out to sort datasets for three roadside feature attributes mentioned above (rate of development, number of roadside facilities and number of access). The data were then enriched with Geographical Information System (GIS) data provided by Kuantan Municipal Council that comprised the essential digital maps.

Most roadside features data have a strong association with type of land use on the roadside areas and significantly had a great impact on the number of traffic accidents as pointed out by Eckhardt, N. and Thomas, I. (2004). The GIS data provides us with the information on the current population of the study area, location of the main road as well as the local roads.

3.4 Roadside Developments Data

The attribute by rate of development along the study route was referred to the index of development at the roadside area for every section. The index of development was presented in terms of percentage of developed land against the total land area. Higher percentage value meant to represent highly developed areas while low percentage value meant to represent less developed areas.

The attribute by roadside facilities along the study route was referred to the existence of various facilities such as guardrail, trees in groups, utility poles, street lighting and big advertisement boards within the roadside areas for every section. In each section, for each items exists it was marked as one, if all these five features exists in that section, the roadside facilities for that section were counted as 5. At the end of the observation, a table consisted of number of roadside facilities existed in every section were produced.

The attribute by number of access along the study route was referred to the number of paved unsignalized junctions i.e. access to/from residential area, shop lots and schools. Higher number of accesses and paralell roads would lead to heavy traffic flows and consequently higher risk of traffic accidents as suggested by Greibe, P. (2003).

Population was chosen as one of the roadside development attributes since it directly related to the development impact along the study routes. Roadside areas along the study route had experienced significant increase in population in particular attraction from business activities and generation from residential development. The estimation for population was made by focusing on the populated area or residential areas where their access roads were directly connected to the study route.

Traffic volume was chosen as one of the roadside development attributes since it also directly related to the development impact along the study routes in similar fashion as of population data. In some cases, traffic might not only generate from the study area land development but also from outside of the study route especially during festive seasons. For instance, Section 7 which consisted of undeveloped land was still recorded a high number of traffic accident contributed by the through traffic. Sources of data were obtained from Traffic Volume Malaysia by Highway Planning Units (HPU) at the following traffic counting station:-

a. Station CR 403 (KM 23.3 of the study route)

b. Station CR 404 (KM 8.1 of the study route)

Based on reported traffic volume at this two counting stations, traffic volume for year 2000 until year 2005 in every section were then estimated using the yearly traffic growth factor.

4. DATA ANALYSIS

4.1 Road Accident Analysis

Traffic accidents data for the 24 kilometres route of Jalan Kuantan-Gambang starting from KM 3 and ending at KM 27 were collected, interpreted and processed in eight similar sections (three kilometres interval).

Table 1. *Summary of Number of Traffic Accidents by Section (Year 2000 – 2005)*

	2000	2001	2002	2003	2004	2005	Total
Section 1	11	13	9	10	19	6	68
Section 2	9	3	6	6	9	6	39
Section 3	9	11	8	3	8	10	49
Section 4	6	10	10	13	9	5	53
Section 5	3	6	6	9	12	5	41
Section 6	4	5	3	5	13	5	35
Section 7	3	3	3	13	2	2	26
Section 8	3	2	8	8	13	8	42
Total	48	53	53	67	85	47	353

With reference to Table 1, an average of 14.7 numbers of traffic accidents per kilometer in six year period or 2.5 numbers of traffic accidents / kilometer / year was recorded for the study route. Many factors contributed to high number of traffic accidents and one of this was the effect of roadside development along the study route.

4.2 Relationship between Roadside Development and Number of Traffic Accidents

Equation 1 represented the linear multiple regression for measuring the effect of roadside development to number of traffic accidents.

$$\mathbf{Y_1 = \alpha + \beta_1 \mathit{x}_1 + \beta_2 \mathit{x}_2 + \beta_3 \mathit{x}_3} \qquad (1)$$

Where;

Y_1 = Traffic Accidents (Numbers)

ß, α_1 = Parameters determined through regression analysis

x_1 = Population (Persons)

x_2 = Traffic Volume (vehicle / day)

x_3 = Rate of Development (%)

The results of the multiple regression analysis were as per Table 2 below.

Table 2: *Results of Multiple Regression Analysis between Roadside Development and Number of Traffic Accidents*

Parameters	Results
Standard Deviation	10.303
Multiple Correlation, R^2	0.623

As shown in Table 2, the standard deviation of this model was lower than the value of the simple regression model obtained earlier with lower number of variables (standard deviation value: 15), indicating that the variations amongst data were minimized. More importantly, the R^2 value of 0.623 for this multiple regression was much higher than that of the simple linear model (R^2 value: 0.137) indicating that the model had improved with the inclusion of two more variables.

Multiple linear model derived to predict number of traffic accidents using coefficients obtained from the analysis was presented as below;

$$\mathbf{Y_1 = 24.952 - 0.003x_1 - 0.0002x_2 + 0.633x_3} \quad (2)$$

Where,

Y_1 = Traffic Accidents (Numbers)

x_1 = Population (persons)

x_2 = Traffic Volume (vehicle / day)

x_3 = Rate of Developments (%)

The model obtained could be used to measure the effect of roadside development on number of traffic accidents not only within the study area but also to other areas.

However, applications to other study areas required prior works to study the patterns of traffic accidents as pre-requisite for providing supporting evidences to the results of the model.

5. CONCLUSIONS

In the first phase of the study, the investigation of the traffic accidents data between year 2000 to 2005 showed that traffic accidents within those periods were able to be controlled since the number of traffic accidents for the study route in year 2000 and year 2005 was almost the same.

Main objective of this research are to determine the relationship between roadside developments and number of accident cases along the study route. With these findings, the investigation on the effect of roadside developments on the number of traffic accidents was narrowed down for the year 2005 only.

For a simple linear model, number of traffic accidents was best estimated by the population data. However, the R^2 that represent the reliability of the model is too low (R^2 = 0.137). As such, a multiple linear model is developed by combining the population, traffic volume and rate of development as the independent variables.

As a result, this model that gave R^2 of 0.657 improved the reliability of the estimated significantly. Therefore, it was recommended that the multiple model to be used for estimating number of traffic accident for the study route.

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THE ENGINEERING PERFORMANCE OF SUPERPLASTICIZED CONCRETE

Noor Faisal Abas[1] and Muhammad Naim Mahyuddin[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]nfaisal@usm.my

ABSTRACT: Superplasticizers are used for the improving workability and reducing the water to cement ratio. The presence of superplasticizers in a concrete mixture is quite advantageous, in that they assist in the effective dispersion of cement particles and hence improving the workability of concrete. The purpose of this research study is to examine and study the influence of engineering performance of superplasticized concrete with a different percentage added. The types of superplasticizer that are going to be tested are the Conplast SP1000 which consists of sulphonated naphthalene polymers. Conplast SP1000 disperses the fine particles in the concrete mix, enabling the water content of the concrete to perform more effectively. Three different percentage of superplasticizer used are 1.75%, 2.25% and 3%. Every each percentage will reduce the quantity of water in the design mixture. Every specimen was tested on the duration of 7, 28 and 56 day with 3 specimen cube and prism for every level.

Keywords: superplasticizer, cement, concrete

1. INTRODUCTION

A superplasticizer is one of the admixtures which are widely used in concrete in increasing the performance of a concrete. Superplasticizers are playing an important role in the present development of ultrahigh-performance concrete. The new cement-based materials have a very low porosity, which is obtained by the addition in the formulation of very small reactive particles whose role is to fill the interstitial space between larger particles. This addition can lead to very dense, high strength, hardened materials. Another positive point of this addition is that the use of superplasticizers requires a small amount of water inside concrete mixture because only a small quantity of hydrates is necessary to bind together the dense stack of solid particles.

In the nowadays of construction industry, it is imperative to have reinforced concrete structure made out of highly workable and durable concrete. The use of ordinary Portland cement systems alone in severe environmental conditions may not be effective and cannot provide the solution in trying to produce high strength performance reinforced concrete. With that, adding an admixture is one of the solutions which not only increasing the workability of the concrete, but also increasing the durability, consistency and strength of the engineering performance of a reinforced concrete.

Conplast SP1000 is a chloride free superplasticising admixture based on selected sulphonated naphthalene polymers. It is supplied as a brown solution which instantly disperses in water. Conplast SP1000 disperses the fine particles in the concrete mix, enabling the water content of the concrete to perform more effectively. The very high levels of water reduction possible allow major increases in strength to be obtained. The Conplast SP1000 standard complies with BS 5075 Part 3 and with ASTM C494 as Type F.

The Conplast SP1000 is the superplasticizers that are use in this research where a different dosage percentage of this superplasticizer is used to see the performance, workability, durability and its fast reaction with the concrete mix. The Conplast SP1000 provide excellent acceleration of strength gain at early ages and major increases in strength at all ages by significantly reducing water demand in a concrete mix. It's also improve the workability of site mixed and precast concrete without increasing water demand and also improved durability by increasing ultimate strengths and reducing concrete permeability.

For high strength, water reduced concrete the normal dosage range is from 0.80 to 2.20 liters/ 100kg of cementitious material, including PFA and microsilica. For high workability the normal dosage range is from0.80 to 1.50 liters/ 100kg of cementitious material.

Advantages:

i. Major increases in strength at early ages without increased cement contents are of particular benefit in precast concrete, allowing earlier stripping times.

ii. Makes possible major reductions in water: cement ratio which allow the production of high strength concrete without excessive cement contents.

iii. Use in production of flowing concrete permits easier construction with quicker placing and compaction and reduced labour costs without increasing water contents.

iv. Improved cohesion and particle dispersion minimizes segregation and bleeding and improves palpability.

2. TYPES OF TESTS AND DESIGN MIX

i. Types of Concrete Tests in This Research

- Slump test
- Density test
- Compressive strength test
- Flexural test
- Ultrasonic Pulse Velocity test
- Water absorption test
- Oxygen permeability test

ii. Design Mix Proposed

<u>Grade 50:</u>

- Target 50 + 10 = 60 $\frac{N}{mm}$
- Aggregate Size = 20 mm
- w/c ratio = 0.44
- Cement = 512 kg/m
- Water = 210 kg/m
- Density = 2400 $\frac{kg}{m}$
- Total Aggregate = 2400 – 512 – 210
  = 1678 kg/m

Fine Aggregate = 35%

35% x 1678 = 587 $\frac{kg}{m}$

Coarse Aggregate

1678 – 587 = 1091 $\frac{kg}{m}$

Table 1. *Design Mix*

Mix	Cement	Water	SP	Sand	Aggregate
Normal	512 kg/m	210 kg/m	-	587 kg/m	1091 kg/m
SP1	512 kg/m	210 kg/m	1.75%	587 kg/m	1091 kg/m
SP2	512 kg/m	210 kg/m	2.25%	587 kg/m	1091 kg/m
SP3	512 kg/m	210 kg/m	3%	587 kg/m	1091 kg/m

Preparation Mixing:

- Test for 7, 28 & 56 days
- (9 x 0.001m^3) + (9 x 0.005m^3) + (0.15 x 0.024m^3) = 0.0576m^3

Table 2. *Mix Proportion*

Mix	Cement	Water	SP	Sand	Aggregate
Normal	29 kg	12 kg	-	34 kg	63 kg
SP1	29 kg	12 kg	1.75%	34 kg	63 kg
SP2	29 kg	12 kg	2.25%	34 kg	63 kg
SP3	29 kg	12 kg	3%	34 kg	63 kg

Admixture:

Superplasticizer e.g:

Cement x percentage of admixture

- 29 kg x 1.75% = 500 gram
- 29 kg x 2.25% = 650 gram
- 29 kg x 3% = 870 gram

3. RESULT ANALYSIS AND DISCUSSION

3.1 Slump Test Result

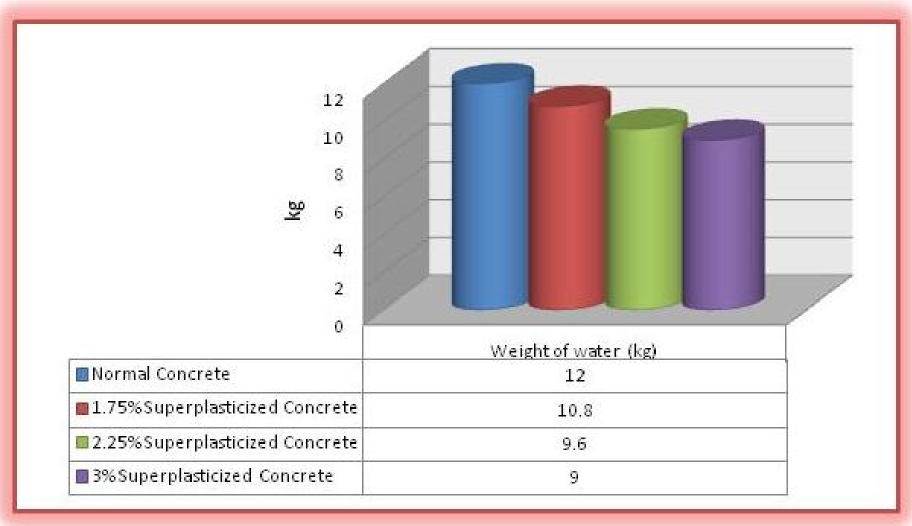

Figure 1. *Weight of Water Used between Four Sample Specimens after Slump Test is done*

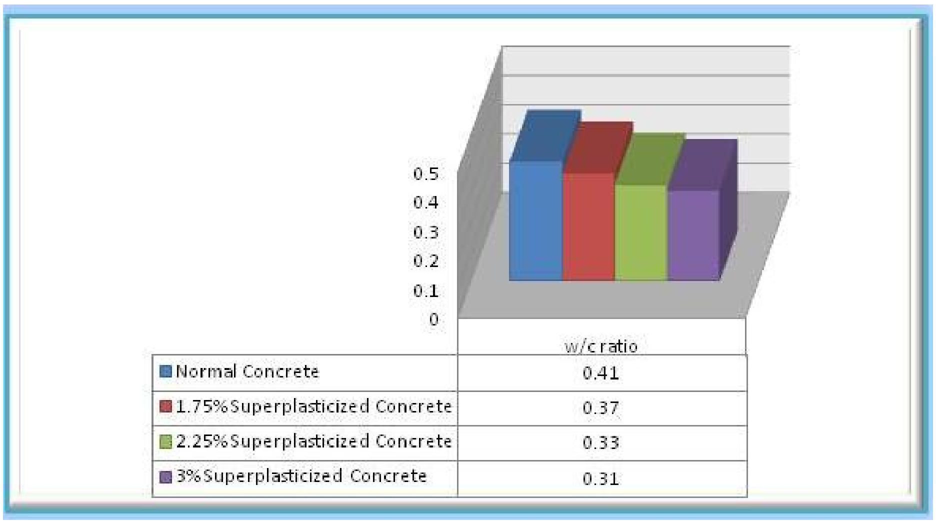

Figure 2. *Actual w/c Ratio to be used for Four Specimen Mix after the Weight of Water is divided by the Weight of Cement.*

From the result shown in the slump test analysis, 2.25% of superplaticizer used in concrete mix is one of the stable values which may reduce the use of water equally and at the same time increase the strength because of the workability which reacts in a great way in the concrete mixture. Although the 3% of superplasticizer in concrete mix has reduced more water than the 2.25% of superplasticizer, but the main worried concern is on the strength of the concrete when it is tested. From this research, researcher has identified that the 3% of superplasticizer used in concrete mix has made an uneven mix between particles of cement, water and aggregate. When placing into the mould, the problems occurred when too much superplasticizer makes the concrete mixture bleed with overflow water and drowning the aggregate further below the ground of the mould.

3.2 Cube Density Test Result

The bar chart concrete density of normal concrete, 1.75% and 3% superplasticized concrete shows the consistency of concrete density between 7 days, 28 days and 56 days even though there are a little amount of difference value between the age tested. For the 3% superplasticized concrete on the age of 7 days tested the density of the concrete slightly higher than the 28 and 56 days of tested.

But as for 2.25% superplasticized concrete, a huge difference of density of the concrete occurs when it reaches to 28 days. The density of this concrete seems to be decreasing by leaving a big gap of 1000 $\frac{kg^3}{m}$ between 7 days and 56 days. The difference value of this density might be occurs because of compaction or vibration has not well compacted causing the particles to be trapped inside the concrete thus reduced the density of the concrete.

This can be summarize as when increasing of the age of the specimen, the concrete density will be more increased even thought the increasing value is not so constant. This is because of the curing factor during the specimen is cured inside the water curing tank which cause the increasing weight of the specimen. The second factor is that the test specimen might not much be dry enough perfectly to obtain the dry weight. Therefore the inaccuracy actual value of concrete density specimen occurs for every concrete mix with the increase of the age.

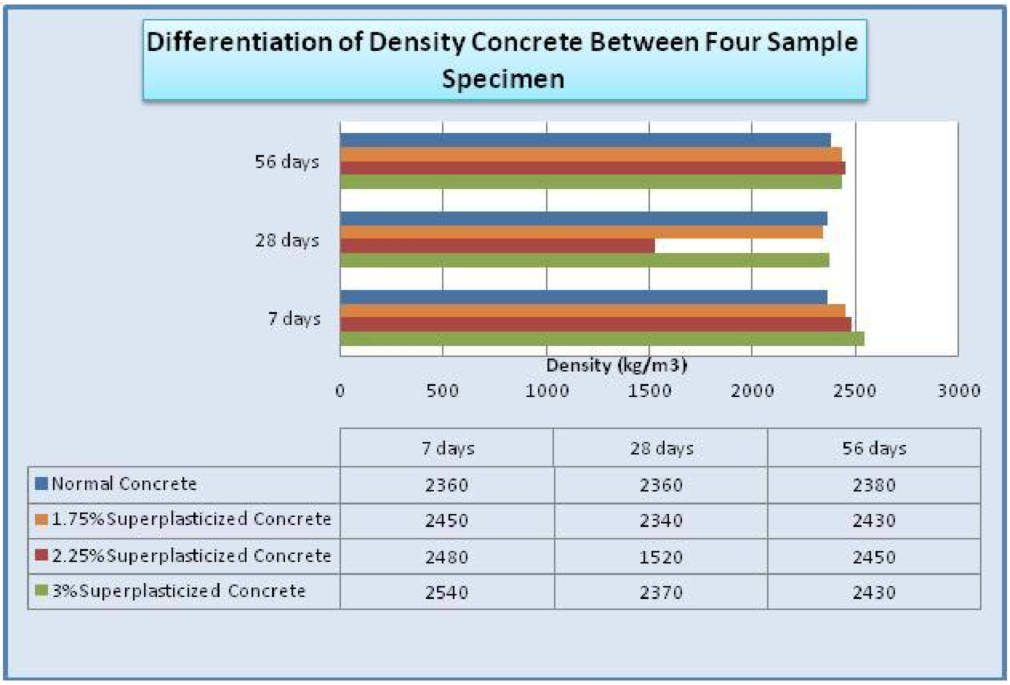

Figure 3. *Average result of density concrete between four sample specimens*

3.3 Ultrasonic Pulse Velocity Test Result

Table 3 Shows the total average difference of the pulse velocity value between normal concrete, 1.75% superplasticized concrete, 2.25% superplasticized concrete and 3% superplasticized concrete according to the age (days).

Table 3. *Pulse* Velocity

Types of Specimen	Age of sample (days)		
	7 days	28 days	56 days
Normal Concrete	3.81 (Good)	4.24 (Good)	5.09 (Excellent)
1.75% Superplasticized Concrete	3.81 (Good)	4.27 (Good)	6.18 (Excellent)
2.25% Superplasticized Concrete	3.82 (Good)	4.16 (Good)	7.10 (Excellent)
3% Superplasticized Concrete	4.09 (Good)	4.34 (Good)	7.22 (Excellent)

From the experiment and the data result shows that the value for all concrete specimens is in good and excellent rate. Starting from 7 days to 28 days shows the results of good rating, and by the age of 56 days of tests, it increase up going for an excellent rate. This shows that the more increase the density of the concrete, the higher will be for the pulse velocity of the concrete.

In ensuring an accurate reading is obtained, the surface of the concrete must free from dust and is applied grease. The ultrasonic pulse velocity equipment may occur in error and inconsistent if the equipment is not calibrated well every time the test is conducted. However, every grade and design shows the increase of the pulse velocity reading, so it shows that the concrete density is increased by time.

3.4 Water Absorption Test Result

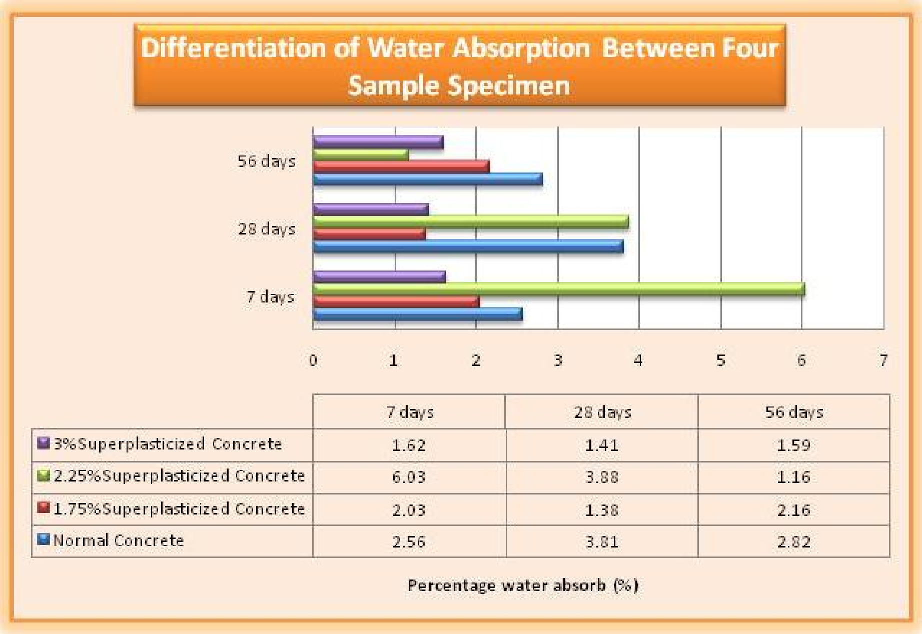

Figure 4. *Average value of water absorption between four types of specimen*

Figure 4 shown the most types of specimens that absorbs more water is the 2.25% superplasticized concrete which increase up to 6% when it reaches the age of 56 days. As for normal concrete, the increase of the water absorption is on 28 days but when it reaches the age of 56 days of tests, it decreased to 2.82%.

According to the bar chart above shown, the result shown for the four types of specimen is quietly inconsistent. This may be caused by the risk of cracking of vibration during the coring work is conducted. Another factor which may occurs the problem is that the specimen is put inside the oven to dry for too long which cause the specimen to shrink and lost its particles of strength connection.

3.5 Compressive Strength Test

The compressive strength test is one of the important tests for the concrete mix. This is because a concrete is really good at compressive strength and weak in flexural strength. This test will show the capability of the concrete strength towards the mix design in various types by using a different percentage of superplasticizers.

According to the figure 5 from the bar chart as shown, the value of compressive strength for normal concrete has increase in a normal way according to the age tested and that is on 7, 28 and 56 days. Compare to 1.75% superplasticized concrete, the value strength which actually increase more than the normal concrete on 7 and 28 days. But when it reaches the age 56 days it's constantly reach the strength of the same as the normal concrete.

According to the bar chart on result of 2.25% superplasticized concrete, the value of compressive strength has magnificently increase far beyond the expectation of the other three specimen. On the 7 days of age, this sample has increase the strength as nearly reach the strength of 28 days. By the age of 28 days, the value of compressive strength has increase passing the value strength of normal and 1.75% superplasticized concrete. When it comes to 56 days, the strength rise up more than what has been expected reaching to grade 80 $\frac{N}{mm}^2$. This may be occurred because of the curing process where the concrete reacts in hydrolysis and hydration process.

Eventually the 3% superplasticized concrete has not increase in a great strength value as the 2.25% superplasticized concrete. But when it reaches the age of 56 days, the value strength has slightly increased rather fast. From the result recorded, this may be concluded that the optimum percentage use of superplasticizer for the concrete design mix can only be limited to 2.25%. The fact is that when using more percentage of superplasticizer in concrete design mix, it increases the strength thus reducing the percentage of water use and increase the workability. But when too much superplasticizer is added, it can be over limit of the design where the mixture will not evenly distributed whether too much bleeding occurs or the mixture is not well mixed thoroughly when not much water can be use if the superplasticizer is too much added.

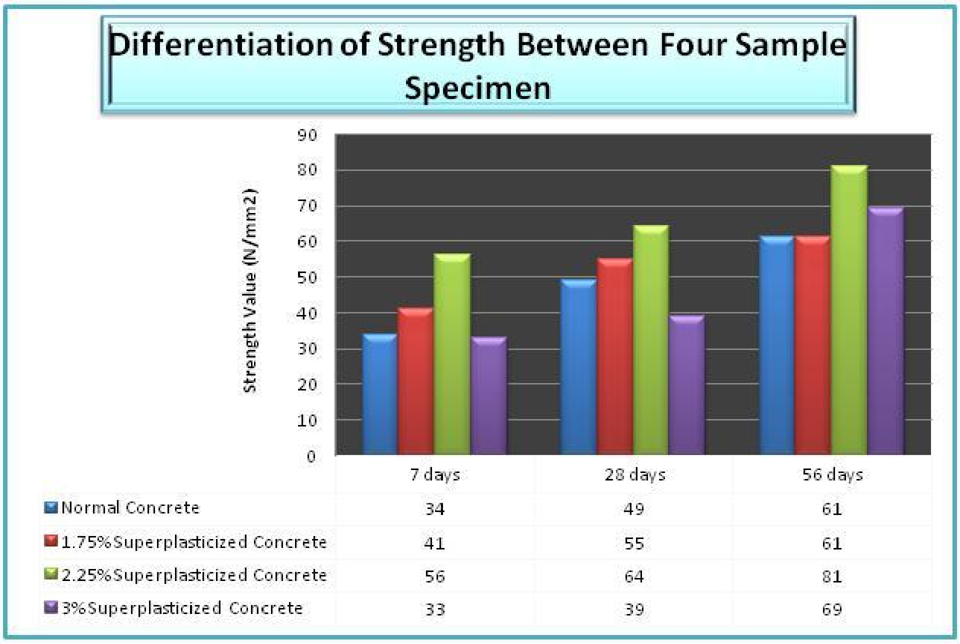

Figure 5. *Average value of Compressive strength between four sample specimens.*

3.6 Flexural Test Result

Even though that the flexural test is rarely done to determine strength of a concrete design nowadays but it is also one of the important aspect especially when it involves in slab structure. The flexural force of a slab is needed especially for the construction of airport to determine the flexural strength of slab constructed. The performance of flexural strength usually more depends on the density and the strength of the concrete where the obtaining weight before flexural test is run shows the tested concrete density is in good condition.

From the data analysis obtained, flexural strength too experiences the increase of strength process. According to the figure 6 from the bar chart shows that the equal increase of strength between 7 days, 28 days to 56 days is the normal concrete. Compared to 2.25% and 3% superplasticized concrete, on the beginning 7 days of tested the flexural strength increase more than the normal concrete. When it reaches 28 days of tests, the flexural strength of the both specimen increases more especially for the 3% superplasticized concrete where it rises over the 2.25% superplasticized concrete. But when it comes to the age of 56 days of tests, the 3% superplasticized concrete shows the decreased of the flexural strength around 1 $\frac{N}{mm^2}$ while 2.25% superplasticized concrete only increase about 0.007$\frac{N}{mm^2}$. From the 56 days of tests result shows that the stronger concrete in flexural strength is the normal concrete.
According to the bar chart, the 1.75% superplasticized concrete has not much shown its performance when it comes to flexural strength test. The result shows that this specimen has been proven to be the weakest concrete in flexural strength.

The increasing of flexural strength which is inconsistent has given a guide where if there is an existence of different quantity types of cement content in every specimen mix, then the features or the characteristics of the strength will be also different between one to another. A material of concrete mixture which plays the important role in concrete strength is the aggregate and cement. So, if there is a change in the quantity of cement content, then there will also be a change of the reading strength.

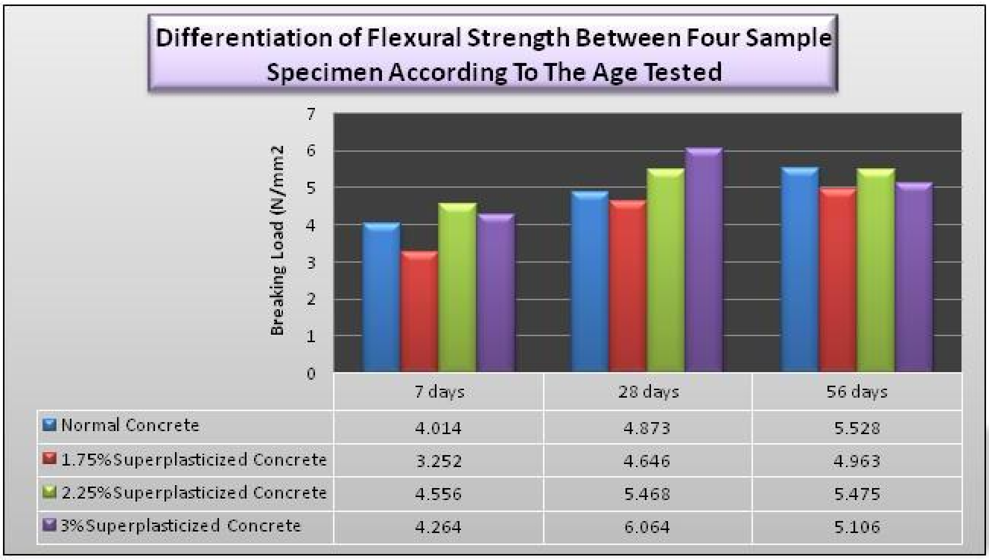

Figure 6. *Total average value of flexural strength for four types of specimen*

3.7 Oxygen Permeability Test

Oxygen permeability test is one of the important tests to determine the durability of the concrete whether the specimen is easily penetrated by small air or not. A concrete which has the fine particles of holes which finds it hard for the air flow through inside the concrete is one of the best chosen criteria that is needed for a quality concrete.

According to the figure 7 as shown from the bar chart, it shows that the highest value of oxygen permeability goes to the normal concrete. This is maybe because of the concrete which reacts in normal condition without the addition of superplasticizer. Comparing with adding a superplasticizer according to the percentage added, the superplasticizer helps increase the workability and the durability of the concrete by closing any entrapped air inside the concrete mix and reducing the possibilities of concrete bleeding according to the certain amount of superplasticizer. The 2.25% superplasticized concrete is the concrete which has the least amount of oxygen permeability. This shows that this type of concrete has a high durability.

	7 days	28 days	56 days
3%Superplasticized Concrete	2.04E-23	2.04E-23	1.95E-23
2.25%Superplasticized Concrete	1.87E-23	1.91E-23	1.89E-23
1.75%Superplasticized Concrete	2.15E-23	2.09E-23	2.06E-23
Normal Concrete	2.63E-23	2.58E-23	2.50E-23

Figure 7. *Average value of oxygen permeability between four types of specimen.*

4. CONCLUSIONS

The engineering performance of superplasticized concrete is one of the main topics of this research. By testing each different percentage of superplasticizer in concrete mix, the performance of each concrete mixture according to the percentage of superplasticizer added has shown magnificent performance and unbelievable result. The sample specimen used in this research are consists of four types of specimen and that is the normal concrete, 1.75% superplasticized concrete, 2.25% superplasticized concrete and 3% superplasticized concrete which is used to tests on the age of 7, 28 and 56 days. The design grade of the concrete in this research is grade 60.

From the test series that has been done towards the four types of design specimen, this can be concluded that the superplasticizer used in concrete mixture has made the following occurs:

- With the additional of 1.75%, 2.25% and 3% of superplasticizer inside the concrete mix, the density of the concrete increase more when comparing with the normal concrete. This can be concluded that when the more percentage of superplasticizer is added inside the concrete mixture, the more increase the density and its workability because the function of superplasticizer is to reduce the use of water inside the mixture thus reduced the existence of air bubble inside the concrete mix making the concrete more durable and strong.
- The concrete mix design which used different percentage of superplasticizer has showed an extremely increased of compressive strength than the normal design mix without superplasticizer. From chapter 5 table 5.6 (e), shows the total average of the four specimen which can identify the most perform concrete in this research. The 2.25% superplasticized concrete shows the most increase value strength than the other three specimens from the age tests of 7 days, 28 days to 56 days. However, the 3% superplasticized concrete does not perform well when tested in compressive strength. This can be concluded that the maximum limit of superplasticizer which can only be added is around 2.25% to 2.5%. This is because if superplasticizer is added within the percentage of an over limit of the requirement design which has been set, the concrete mix will fail and inconsistent mixture will occur between aggregate, sand, water and cement such as segregation.
- A concrete with no reinforcement usually cannot withstand with the strength of flexural. In flexural strength test, the 2.25% superplasticizer concrete shows the highest value of flexural strength among the three types of specimen. The performance of flexural strength usually more depends on the density and the strength of the concrete.

Overall, clearly shows that the used of superplasticizer in concrete mixture has influence the properties and the characteristics of concrete engineering. However, the actual percentage of superplasticizer that can be use only limited to 2.25% to 2.5% in gaining a high performance superplasticized concrete.

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T193

USE OF RAW CLAY OF LABU SAYONG AS A PARTIAL CEMENT REPLACEMENT IN CONCRETE: ENGINEERING PROPERTIES

Zulazril bin Idris[1], Noor Faisal bin Abas[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]zhuthyca_5535@yahoo.com.my, [2]nfaisal@usm.my

ABSTRACT: The use of pozzolanic materials as admixtures in partial replacement of Portland cement in mortars and concrete increased significantly in the last 20 years. Some pozzolanic materials, such as silica fume, fly ash, and rice husk ash, are by-products of industrial and agro industrial processes, which further contribute to reduce both environmental and economical problems that would be associated with their disposal. The study has been taken to investigate the engineering properties of concrete with partial cement replacement of raw clay of labu sayong. Five different concrete design mix by replacing the cement content with raw clay of labu sayong in the ratio 5%, 10%, 15% and 20%. The concrete grade 50 was designed in this study as a control mix. The study shows that, replacing cement in the range of 5% to 20% with raw clay result in increasing the long term development of compressive and flexural strength. The design mix of 5% replacement of cement with raw clay has the highest compressive strength at the all age of specimen compared with the other mix.

Keywords: Pozzolanic material, Environmental problems, Cement replacement, Compressive strength, Flexural strength.

1. INTRODUCTION

The world is now more concerned about the pollution occurred. Many agencies are increasingly concerned over the environmental quality in the world. This awareness is also felt in the construction industry. For example, an architect board has introduced the index of green building (GBI) as a way to reduce the greenhouse effect in the design and use of building materials for the building. This awareness also has led many researchers to carry out research to produce new building materials that's more environmentally friendly. One of building materials to receive serious attention in the research study is concrete.

The main ingredient in the production of concrete is cement. Production of cement by the cement factory plant has contributed many environmental pollutants. T. W. Bremner on his thesis of environmental aspects of concrete was stated that there is a numbers of environmental problems come from cement industries such as carbon dioxide (CO2) emissions, nitrous oxide emissions, noise pollution, water pollution and particulate air emissions. The carbon dioxide and nitrous oxide are among the greenhouse gases. The emission of these two gases to the atmosphere will cause the greenhouse effect to our earth.

Rationally, there are 2 simple solutions for the all environmental problem above. Firstly is by reducing the production of cement and secondly is by replacing the cement content in concrete with viable alternatives where is possible. However, it seems impossible for the cement manufacturers to reduce their total production of cement because the cement demand is increasing every year. This method may not be agreed upon by most of the cement as it will impact the amount of their income. Therefore, the method of replacement of cement with other substances that have characteristics similar or almost similar to the characteristics of cement is seen as the most effective solution for manufacturers of cement.

Many studies have been conducted by the researchers in the past 2 or 3 decade to utilize effectively wastes material and find a suitable alternative for partial cement replacement in concrete without decreasing the strength and durability of the conventional concrete. The use of waste material as partial replacement of cement in concrete also can preserve the environment and get to solve problems of waste disposal from industry.

The use of pozzolanic materials as admixtures in partial replacement of Portland cement in mortars and concrete increased significantly in the last 20 years. The name of pozzolan was taken in conjunction with the name of the village, Pozzuoli, near Napoli in Italy (Sersale, 1958 cited in Kitsopoulos, 1996, p. 576). Some pozzolanic materials, such as silica fume, fly ash, and rice husk ash, are by-products of industrial and agro industrial processes, which further contribute to reduce both environmental and economical problems that would be associated with their disposal (G. C. Cordeiro, 2008).

Metakaolin is one of the examples of pozzolanic material. It is obtained from calcining the kaolin (clay) within the certain temperature. Thermal activation of clay minerals between 600–900 °C by dehydroxylation leads to the breaking down or partial break down of the structure forming a transition phase with high reactivity (Ambroise et al., 1986 and Sayanam et al., 1989 cited in R. Siddique, 2009, p. 393). Metakaolin is poorly crystallized transition phase obtained by calcining kaolin in the temperature range 700 – 800 °C (M. Murat, 1983 and J. Ambroise, 1989 cited in Ambroise et al., 1994, p. 161)

The mechanical properties of concrete with partial cement replacement with metakaolin were improved compared to the normal concrete using 100% ordinary Portland cement. The previous research has reported, the compressive strength of concrete will increased by replacing 5% to 15% weight of cement with metakaolin. C. –S. Poon et al. 2001 was reported that the cement pastes containing 5% to 20% metakaolin had higher compressive strength than the control at all ages of test.

The use of metakaolin from different origin also will affect the performance of the concrete. The clay from the different resources had a different chemical content. It is one of the reasons why the concrete with partial cement replacement of cement with metakaolin from different origin had different compressive strength. F. Curcio et al., 1998 in the research on metakaolin as pozzolanic microfiller for high performance mortars was using the four different origin of metakaolin. He found that the use of metakaolin from the different origin at the similar percentage replacement will contribute the different compressive strength. Based on the above reason, this paper will discuss the mechanical properties of concrete with partial cement replacement with raw clay of labu sayong. The aims of this paper are to study the compressive strength and flexural strength of concrete containing partial cement replacement of raw clay of labu sayong.

2. METHODOLOGY

2.1 Material used

ASTM Type I ordinary Portland cement was used in all concrete mixtures. Mineral admixture that was used in this research is raw clay of labu sayong in the form of yellowish powder, collected from the area of Sayong, Perak. The chemical composition of ordinary Portland cement and raw clay of labu sayong is given in Table 1

Table 1. *Chemical composition of cement and mineral admixture*

Chemical composition %	Formula	Portland cement type 1	Raw clay
Silicon dioxide	SiO2	20.90	53.40
Aluminum oxide	Al2O3	4.76	27.78
Iron oxide	Fe2O3	3.41	2.16
Calcium oxide	CaO	65.41	0.11
Magnesium oxide	MgO	1.25	0.12
Sulphur trioxide	SO3	2.71	0.13
SiO2 + Al2O3 + Fe2O3		-	83.34

The coarse aggregate used in this study is the locally available aggregate with nominal size greater than 5.00mm and less than 20.00mm. The fine aggregate which is used in the concrete mix for this research is from river sand. The range size of the river sand particles used in concrete mix design is around 5 mm below. Sieve analysis test is done according to the British standard specification BS 882: 1982 and that is with the sieve size of between 5 mm to 0.15 mm.

2.2 Sample preparation

The concrete specimens were prepared in this study. The concrete grade 50 (the compressive strength at 28 days is >50 N/mm²) is used as the control specimen. Than the concrete with raw clay of labu sayong as a partial cement replacement in different ratio namely C2 (5%), C3 (10%), C4 (15%), and C5 (20%) were prepared. The mix proportions of the concrete are shown in Table 2 below.

Table 2. *Mix proportion of concrete*

Sample name	Concrete grade (N/mm²)	w/c ratio	w/b ratio	Mix proportion kg/m³					
				Portland cement	Aggregate		Admixture	Water (w/c + w/b)	Super plasticizer
					Course	Fine	Raw clay		
C1	50	0.44	-	477.00	959.28	753.72	-	210	4.77
C2	50	0.44	0.61	453.15	959.28	753.72	23.85	224.55	4.53
C3	50	0.44	0.61	429.30	959.28	753.72	47.70	239.10	4.29
C4	50	0.44	0.61	405.45	959.28	753.72	71.55	253.65	4.05
C5	50	0.44	0.61	381.60	959.28	753.72	95.40	268.19	3.82

2.3 Experimental program

a. *Compressive Strength*

Concrete cube with dimension 100 x 100 x 100mm was used to determine the compressive strength. The concrete were casting using Portland cement with partial cement replacement of raw clay at 5%, 10%, 15%, and 20%.the specimens were vibrated using vibrator table to assist the compaction. The concrete samples were demoulded after 24 hours (1 day) casting and cured in water until the testing age. The compressive strength of all concrete specimens was determined at ages of 3, 7, 14, 28 and 60 days. Three cubes were used for each test.

b. *Flexural Strength*

Concrete prism with dimension 100 x 100 x 500mm was used to determine the flexural strength. The concrete were casting using Portland cement with partial cement replacement of raw clay at 5%, 10%, 15%, and 20%.the specimens were vibrated using vibrator table to assist compaction. The concrete samples were demoulded after 24 hours (1 day) casting and cured in water until the testing age. The compressive strength of all concrete specimens was determined at ages of 3, 7, 14, 28 and 60 days. Three cubes were used for each test.

3. RESULT AND DISCUSSION

The result of compressive strength test of concrete with partial cement replacement of raw clay was shown in Figure 1. The value of compressive strength is the average value of three cubes for each age. Figures 1 show that the compressive strength of concrete with 5% replacement of cement with raw clay (C2) has highest result. The compressive strength increased 74.34%, increased 41.20%, increased 47.03%, increased 50.67% and increased 52.29% respectively at 3days, 7days, 14days, 28days and 60days of specimen ages.

The development strength of C3 (concrete with 10% replacement of raw clay) is slightly below the control mix for the early age of curing, but it is become higher than the control mix after 28 days of curing. At the age of 60 days, the compressive strength of C3 is 23.36% higher than the control mix. While the C4 and C5 compressive strength are below the compressive strength of control mix at every age of specimen. However, the different of strength are decreasing when the age of specimen increase. The C4 mix (concrete with 15% replacement of raw clay) has slightly the same strength with the control mix.

Based on the Figure 1 below, it looks like the concrete with replacement 5% to 20% of cement with raw clay will become same or maybe higher than the control mix strength when the age of specimen increased. The graph of C3 to C5 is still increased while the C1 was start decreased or remain constant after the age of 60 days.

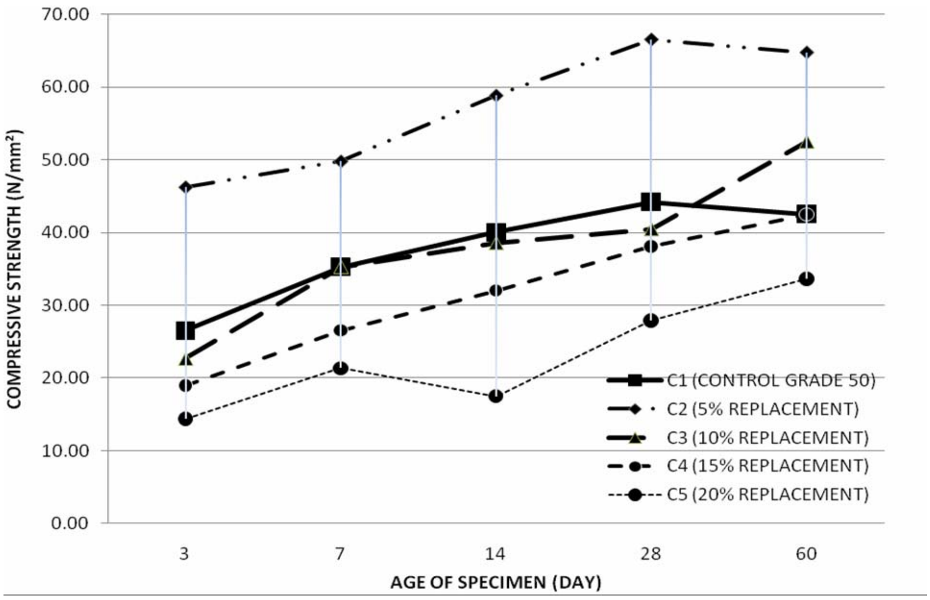

Figure 1. *Compressive strength of concrete grade 50 (N/mm^2)*

The result of flexural strength test of concrete with partial cement replacement of raw clay was shown in Figure 2. The value of flexural strength is the average value of two prisms for each age of specimen. From the result below, the average flexural strength varies from 2.20 to 5.47 N/mm². Figure 2 show that the distributions of flexural strength of concrete with replacement 5% to 20% of cement with raw clay are inconsistent at the early age of specimen compared the control mix. However, the flexural strength was more consistent at the age of 28days and above. C2 show the highest development of flexural strength compared to the other mixes. At the age of 60days, C2 has exceeding 21.56% of control mix strength.

The C3, C4, and C5 mixes however still have the lower flexural strength than the control mix. Even the flexural strength still lower than control, but the strength is indirectly perpendicular with the age of specimen increase. That means when the age of specimen increase, the different of strength between the C3, C4, and C5 with the control mix decrease. Besides, the C2, C3, C4, and C5 shows that their strength will still increase after the 60days curing period while the control mix does not show that it will be increased in strength after 60days.

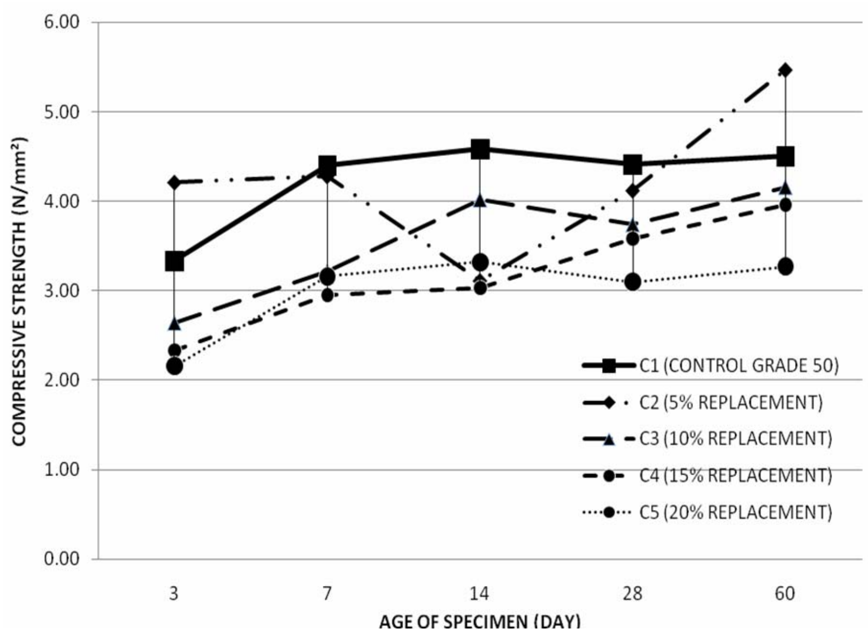

Figure 2. *Flexural strength of concrete grade 50 (N/mm^2)*

4. CONCLUSIONS

Based on the results indicate that, an increase of compressive strength and flexural strength of concrete is definitely noticed with the addition of mineral admixtures (raw clay of labu sayong). The result also show that, by replacing 5% t0 15% of cement with raw clay will increasing the long term strength development of compressive and flexural strength. Thus, it will improve the engineering properties of the concrete. The concrete with 5% replacement of cement with raw clay has the highest compressive strength at all the age of specimen.

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A REVIEW ON MECHANICAL AND THERMAL PROPERTIES OF LIGHTWEIGHT FOAMED CONCRETE AT AMBIENT TEMPERATURE

Md Azree Othuman Mydin[1] , Hanizam Awang[2]

[1,2]School of Housing, Building and Planning, Universiti Sains Malaysia, 11800, Penang

[1]azree@usm.my , [2]hanizam@usm.my

ABSTRACT: In recent years, the construction industry has shown significant interest in the use of lightweight foamed concrete (LFC) as a building material due to its many favourable characteristics such as lighter weight, easy to fabricate, durable and cost effective. LFC is a material consisting of Portland cement paste or cement filler matrix (mortar) with a homogeneous pore structure created by introducing air in the form of small bubbles. With a proper control in dosage of foam and methods of production, a wide range of densities (400 – 1600 kg/m^3) of LFC can be produced thus providing flexibility for application such as structural elements, partition, insulating materials and filling grades. LFC has so far been applied primarily as a filler material in civil engineering works. However, its good thermal and acoustic performance indicates its strong potential as a material in building construction. The focus of this paper is to classify literature on foam concrete in terms of its mechanical and thermal properties at ambient temperature.

Keywords: foamed concrete, thermal properties, mechanical properties

1. INTRODUCTION

LFC is defined as a cementitious material having a minimum of 20 per cent by volume of mechanically entrained foam in the mortar slurry (Van Deijk, 1992) in which air-pores are entrapped in the matrix by means of a suitable foaming agent. The air-pores are initiated by agitating air with a foaming agent diluted with water; the foam then carefully mixes together with the cement slurry to form LFC. Integrating the air-pores into the base matrix gives a low self-weight, high workability, but lower strength in contrast to normal weight concrete. LFC can be fabricated anywhere in any shape or building unit size. Table 1 shows the range of densities suitable for different applications.

2. CONSTITUENTS MATERIAL OF LFC

LFC with low density, i.e. having a dry density of up to about 600 kg/m^3, is frequently formed from cement (to which other binders could be added), water and stable foam whilst denser LFC will incorporate fine sand in the mix. The requirements of each constituent of LFC are explained below.

2.1 Cement

Portland cement SEM1 is typically used as the main binder for LFC. Additionally, rapid hardening Portland cement (Kearsley and Wainwright, 2001), calcium sulfoaluminate and high alumina cement (Turner, 2001) have also been used to reduce the setting time and to obtain better early strength of LFC. There was also an attempt to decrease the cost of production by using fly ash (Kearsley and Wainwright, 2001) as cement replacement to enhance consistency of the mix and to reduce heat of hydration while contributing for long term strength.

2.2 Fillers (sand)

Sach and Seifert (1999) suggested that only fine sands having particle sizes up to about 4mm and with an even distribution of sizes should be used for LFC. This is primarily because coarser aggregate might lead to collapse of the foam during the mixing process. Coarse pulverised fuel ash (PFA) also can be used as a partial or total replacement for sand to make LFC with a dry density below about 1400 kg/m^3.

2.3 Water

The amount of water to be added to the mix depends on the composition of the mix design. Generally for lighter densities, when the amount of foam is increased, the amount of water can be decreased. The water-cement ratio must be kept as low as possible in order to avoid unnecessary shrinkage in the moulds. However, if the amount of water added to cement and sand is too low, the necessary moisture to make a workable mix will have to be extracted from the foam after it is added, thereby destroying some of the foam in the mix. The range of water-cement ratio used in LFC is between 0.4 to 1.25 (Kearsley, 1996), the appropriate value will be depending on the amount of cement in the mix, use of chemical admixtures and consistence requirement.

2.4 Surfactants (foaming agent)

There is an extensive choice of surfactants (foaming agent) available in the market. Generally two types of surfactants can be used to produce foam: protein and synthetic based surfactants. Protein based surfactants are produced from refined animal products such as hoof, horn and skin whilst synthetic based surfactants are produced using man made chemicals such as the ones used in shampoos, soap powders and soaps (Md Azree, 2004). The surfactant solution typically consists of one part of surfactant and between 5 and 40 parts of water but the optimum value is a function of the type of surfactant and the technique of production. It is very important to store all surfactants accordingly because they are inclined to deterioration at low temperatures. According to McGovern (2000), foams formed from protein based surfactants have smaller bubble size, are more stable and have a stronger closed bubble structure compared to the foam produced using synthetic surfactants. Therefore, protein based surfactants would be best suited for the production of LFC of comparatively high density and high strength.

3. DESIGN PROCEDURE

At the moment, there is no standard method for designing LFC mix. For normal weight concrete, the user would signify a certain compressive strength and the water-cement ratio would be adjusted to meet the requirement. As far as LFC is concerned, not only the strength is specified, but also the density. It is not an easy task to achieve an accurate measurement of the density of LFC on site because of the hardened density of LFC depends on the saturation intensity in its pores. According to Jones and McCarthy (2005), it is difficult to achieve the design density of LFC because it has a tendency to lose between 50 and 200 kg/m^3 of the total mix water because it depends on the concrete fresh density, early curing regime and exposure conditions.

4. RELEVANT STUDIES ON PROPERTIES OF LFC

There is a lack of published information on LFC. Among the LFC related literature collected by the author, majority of these were published within the last 10 years and most of these previous studies on LFC were aimed at characterizing the ambient temperature properties of LFC. This section will review previous studies on properties of hardened LFC, including physical properties (density, air-void system and porosity), mechanical properties (compressive strength, tensile strength and modulus of elasticity), thermal properties and fire resistance performance.

4.1 Density of LFC

The relationship between dry density and casting density between 600 kg/m^3 and 1200 kg/m^3 can be calculated using the following linear equation (Kearsley and Mostert, 2005):

$$\rho_m = 1.034\rho_{dry} + 101.96 \quad .... (1)$$

where ρ_m is the target casting density (kg/m^3) and ρ_{dry} is the dry density (kg/m^3)

4.2 Air-void system and porosity of LFC

Porosity and the pore structure will have significant effects on thermal conductivity and mechanical properties of LFC. High porosity is highly detrimental to the strength of LFC, particularly if the pores are of large diameter. Large pores also result in high thermal conductivity. As a cement-based material, LFC consists of gel-pores (dimensions from 0.0005µm up to 0.01µm), capillary-pores (0.01µm to 10µm) and air-pores (air entrained and entrapped pores) (Visagie and Kearsely, 2002). The gel-pores occupy between 40 to 55% of total pore volume but they are not active in permeating water through cement paste and they do not influence the strength (Brandt, 1995). However, the water in the gel-pores is physically bonded to cement and directly controls shrinkage and creep properties of LFC.

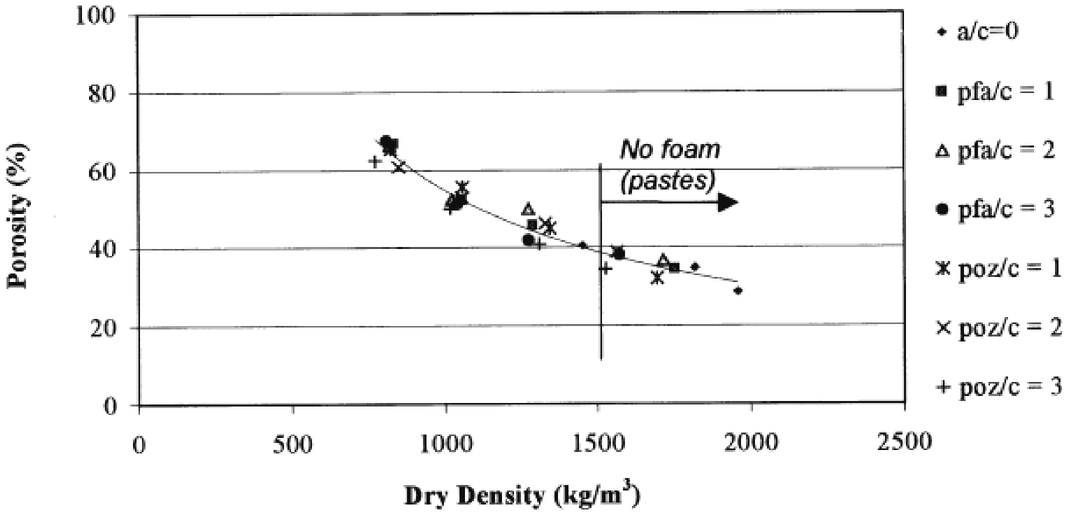

Figure 1. *Porosity of LFC as a function of dry density (Kearsley and Wainwright, 2001)*

Air-pores in hardened LFC can be entrained or entrapped. Entrapped air-pores occur inadvertently during the mixing and placing of concrete. As LFC is a self-flowing and self-compacting concrete and exclusive of any coarse aggregate, the possibility of entrapped air is insignificant. In contrast, entrained air-pores are introduced intentionally during production of LFC by using an air-entraining chemical admixture (surfactants). Entrained air-pores are discrete and individual bubbles of spherical shape. They are uniformly distributed throughout the cement paste and are not interconnected with each other and therefore do not affect the permeability of LFC (Kalliopi, 2006). The total volume of capillary-pores and air-pores affects the strength of LFC.

Kearsley and Wainwright (2001) carried out research to investigate the relationship between porosity and dry density of LFC. In this study, they utilized a large amount of both classified and unclassified fly ash (pulverised and pozz-fill) as a cement replacement up to 75% by weight. Figure 1 shows the relationship between porosity and dry density of LFC obtained from their research. It can be seen from Figure 1 that there is a strong relationship between porosity and dry density of LFC. They found that the porosity of LFC is the combination of entrained air-pores and the pores within the paste and the porosity was found to be dependant primarily on dry density of LFC and not on fly ash type and content. Kearsley and Wainwright proposed an equation to link the porosity and dry density of LFC (based on Figure 1) as follows:

$$\varepsilon = 18700\rho_{dry}^{-0.85} \quad ......(2)$$

where ε is the porosity (%) and ρ_{dry} is the dry density (kg/m^3)

4.3 Compressive strength of LFC

Table 1. *A review of LFC mixes, compressive strengths and density ranges (Ramamurthy et al., 2009)*

Authors	Proportion of cement (kg/m^3) or composition	Ratios			Density range kg/m^3	Compressive strength (28 days) N/mm^2
		S/C	W/C	F/C		
Van Deijk (1991)	Cement-sand/ fly ash	-	-	-	280-1200	0.6 -10.0 (91 days)
Durack and Weiqing (1998)	270–398	1.23-2.5	0.61-0.82	-	982-1185 (DD)	1.0-6.0
	137-380	-	0.48-0.70	1.48-2.50	541-1003 (DD)	3.0-15.0 (77 days)
Kearsley and Wainwright (2001)	Cement-fly ash replacement 193-577	-	0.6-1.17	-	1000-1500	2.0-18.0
Jones and McCarthy (2005)	500	1.5-2.3	0.3	-	1400-1800	10.0-26.0
	500	-	0.65-0.83	1.15-1.77	1400-1800	20.0-43.0
Nambiar and Ramamurthy (2006)	Cement-sand mix (coarse)	With filler-cement ratio varied from 1 to 3 and fly ash replacement for sand varied from 0% to 100%			800-1350 (DD)	1.0-7.0
	Cement-sand mix (fine)					2.0-11.0
	Cement-sand-fly ash mix				650-1200 (DD)	4.0-19.0

** S/C: sand–cement ratio; F/C: fly ash–cement ratio; W/C: water–cement ratio; DD: dry density*

The compressive strength of LFC reduces with decreasing density. Table 1 shows a summary of the range of compressive strength of LFC for various mixture composition and densities reported in literature. For mixes with similar constituents, the density-strength relations should be reasonably comparable. But, because the constituents in LFC mixtures can differ widely, density is not necessarily a dependable indicator of the compressive strength of LFC. The other main factors that influence the strength of LFC are cement-sand ratio, water-cement ratio, type of cement and content, pore size and distribution, type of surfactants (foaming agents) and curing regime (Aldridge, 2005), (Hamidah et al., 2005). Higher sand-cement ratios result in LFC with lower compressive strength. The strength of lower density LFC can be increased to equal that of higher density LFC by increasing the amount of cement content in the mix.

The effect of water-cement ratio on compressive strength of LFC is imprecise. Dransfield (2000) reported that the strength of LFC decreases with reduction in water-cement ratio. Whilst an other report indicates that the compressive strength of LFC reduces with increasing water-cement ratio up to 0.45, an opposite trend is noted above this value (between 0.5 and 1.0) (De Rose and Morris, 1999).

When cement is combined with silica fume (Kearsley, 1996) and fly ash (De Rose and Morris, 1999), higher compressive strength is achieved in the long term, owing to their pozzolanic reaction and filler characteristics, with a more marked effect at high LFC densities. Kearsley and Wainwright (2001) carried out a study on the effect of replacing large volumes of cement (up to 75% by weight) by both classified and unclassified fly ash on strength of LFC. They found that up to 67% of the cement could be replaced with ungraded and graded fly ash without any significant reduction in compressive strength. The results signify that the compressive strength of LFC is principally a function of dry density, and LFC mixes with high fly ash content needed a longer time to reach their maximum strength which was observed to be higher than that attained using only cement.

In terms of the influence of fillers on strength of LFC, better strength is obtained when finer sand is used. For a given density, the mix with fine sand results in higher strength than the mix with coarse sand and the variation is higher at higher density. This higher strength-density ratio is credited to the moderately uniform distribution of pore in LFC with fine sand, while the pores were larger and irregular for mixes with coarse sand (Nambiar and Ramamurthy, 2006). Similar behaviour was observed when sand was replaced by fine fly ash.

Jones and McCarthy (2005) performed an extensive experimental exploration into the effect of utilization of unprocessed, run-of-station, low-lime fly ash in LFC, as a substitution for sand on the rheological, strength development and permeation/durability properties for LFC with plastic densities ranging between 1000 and 1400 kg/m^3. They found that the use of fly ash in LFC considerably benefited the compressive strength growth, mainly after 28 days. At a known age, the fly ash coarse concretes were up to 6 times stronger than equivalent sand concretes.

The enhancement of strength with fly ash as filler is not pronounced at lower density range especially at earlier ages. This is due to the fact that at lower density range, the foam volume controls the strength rather than the material properties (Nambiar and Ramamurthy, 2006). The utilization of lime, demolition fines, recycled glass as fine aggregate has slight or no effect on compressive strength of LFC, while some decrease in strength was reported when crumb rubber, used foundry sand, china clay sand and quarry fines were used (De Rose and Morris, 1999).

The compressive strength of LFC decreases with an increase in pore diameter for dry density of LFC between 500 and 1000 kg/m^3. Nevertheless for densities higher than 1000 kg/m^3, as the air-pores are far apart to have an influence on the compressive strength, the composition of the paste determines the compressive strength (Visagie and Kearsely, 2002). The type of surfactant (foaming agent) also has major effect on the compressive strength of LFC. An increased of strength up to 70% was found with the used of protein based foaming agent rather than synthetic foaming agent (Dransfield, 2000).

In terms of curing regime, autoclaving increases the compressive strength. Hamidah et al. (2005) carried out an investigation to produce cost-effective mix for LFC by optimising the amount of sand in LFC mix by using different sand-cement ratio and curing conditions. In this study, a series of LFC of four different densities ranging from 1300 to 1600 kg/m^3 was fabricated using the appropriate mix proportions and a series of sand-cement ratios varying from zero to 2.0 for each series of density was attempted. Hamidah et al. found that water cured samples of LFC attained higher strength than those cured in air.

4.4 Flexural and tensile strength of LFC

The ratio of flexural strength to compressive strength of LFC is in the range of 0.06–0.10 and this ratio was also found to reduce with increasing water-cement ratios and decreasing densities (Van Deijk, 1991). The splitting tensile strengths of LFC mixes are higher for mixes with sand than those with fly ash. This is attributed to the improved shear capacity between sand particles and the paste phase (Jones and McCarthy, 2005). The introduction of polypropylene fibers in LFC has been reported to improve the tensile and flexural strength of LFC, provided this does not affect the fresh concrete behavior and self-compaction (Kearsely and Mostert, 1997).

4.5 Modulus of elasticity of LFC

As a porous material, the static modulus of elasticity of LFC is expected to be considerably lower than that of normal weight concrete for dry densities between 500 and 1500 kg/m^3 with values typically varying from 1.0 to 8.0 kN/mm^2, respectively (Jones and McCarthy, 2005). LFC mix containing fly ash as fine aggregate is reported to show lower modulus of elasticity value than that of LFC with sand. Jones and McCarthy (2005) reported that the utilization of polypropylene fibers in LFC mix could enhance the value of modulus of elasticity of LFC between two and four times. They proposed two relationships to predict the modulus of elasticity of LFC as follows:

Sand as fine aggregate $E_c = 0.42\, f_c^{\,1.18}$ …. (3)

Fly ash as fine aggregate $E_c = 0.99\, f_c^{\,0.67}$ …. (4)

where E_c is the modulus of elasticity (kN/mm^2) and f_c is the compressive strength (N/mm^2).

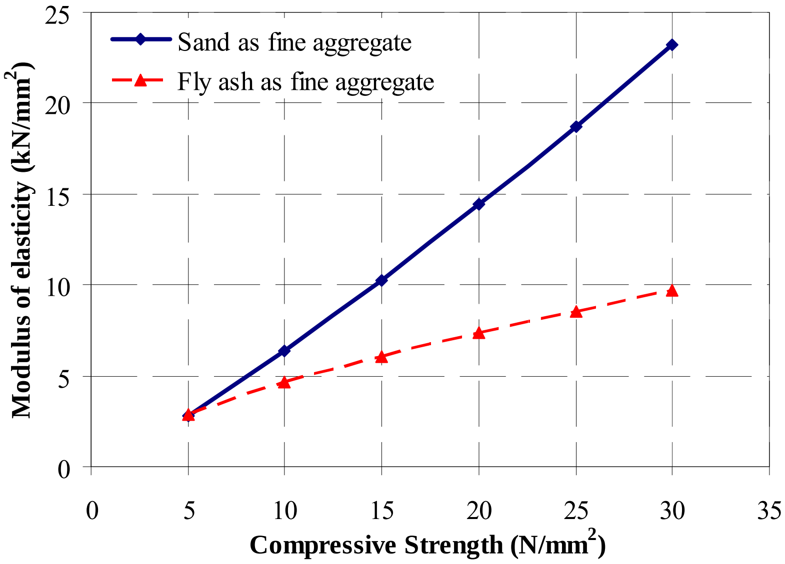

Figure 2. *Relationship between modulus of elasticity and compressive strength of LFC*

Figure 2 shows a plot of modulus of elasticity against compressive strength based on Equation 3 and Equation 4. From Figure 2, it can be seen that for the same compressive strength (f_c), sand as aggregate gives higher modulus of elasticity values compared to fly ash aggregate. This difference is attributed to the high amount of fine aggregate in sand mix compared to fly ash mix, which contains completely paste with no aggregate (Jones, 2001).

4.6 Thermal properties of LFC

Thermal properties are required in two design considerations: to provide thermal insulation at ambient temperature and to provide fire resistance.

4.6.1 Ambient temperature thermal properties

The cellular microstructure of LFC provides it with low thermal conductivity. According to BCA (1994) and Jones and McCarthy (2005), the thermal conductivity of LFC typically is 5 to 30% of that of normal weight concrete and range from between 0.1 and 0.7 W/mK for dry density values of 600 to 1600 kg/m^3 respectively. As pointed out by Kessler (1998), in practical terms normal weight concrete would have to be 5 times thicker than LFC ones to achieve similar thermal insulation. The thermal conductivity of LFC with 1000 kg/m^3 density is reported to be one-sixth the value of typical cement-sand mortar (Aldridge and Ansell, 2001). Since LFC is made by injecting air into a cement based mixture, the density of LFC is directly a function of the air inside LFC. Expectedly, the density of LFC should play an important role in determining its thermal properties. According to Weigler and Karl (1980), a reduction in LFC density by 100 kg/m^3 results in a lessening in its thermal conductivity by 0.04 W/mK.

In addition to use more air to reduce the thermal conductivity of LFC, it is possible to reduce the thermal conductivity of LFC by using pulverized fuel ash. For example, as reported by Giannakou and Jones (2002), a reduction in thermal conductivity by 12-38% was attained with the introduction of 30% pulverized fuel ash in the mix compared to the LFC with only Portland cement SEM1 as binder material. This was attributed to the lower density of fly ash particles.

5. CONCLUSIONS

The review presented in paper clearly indicates that most of the investigations on LFC so far have focused on its ambient temperature properties only. Among these, the majority are about mechanical properties of LFC with only a very few on its thermal properties. Quantitative information on fire resistance performance is extremely sparse. Nevertheless the reviewed literature does give some useful data of LFC mechanical and thermal properties at ambient temperature which can be used as the basis for further research.

ACKNOWLEDGMENTS

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DEFECT MANAGEMENT FOR THE HIGH RISE OFFICE TOWER

Idris Othman[1] and Nadia Azman[2]

[1,2]Universiti Teknologi Petronas, Malaysia

[1]idrisothman@yahoo.com

ABSTRACT: Construction defects had addresses as big issues in construction industries. It has been proved many big projects have a history of failure with non-completion, massive delay and cost overruns. Too excessive defects in the building incurred high cost for maintenance. Commonly major defects which keep happening must be managed correctly or else it will become worse. Many factors actually contributed to the occurrence of defects, and these factors need to be well managed in order to reduce defective works. Moreover, the issues and problem faced by building professional when they deal with defects become one of the obstacles in managing defects. Thus, good construction defect management must be adopted by all the building professional because excessive cost of defects may be reduced or eliminated by only putting some effort and investing small amount of money at initial stage.

Keywords: construction defects, maintenance, building professional

1. INTRODUCTION

Major construction works are often crucial to the success of the individual enterprises and to society as a whole. Such society often embraces the building, civil and heavy engineering sectors. Many major construction works have a history of failure with non-completion, or massive delay and cost overruns. Some completed projects have too many defects and raise the construction cost because defects were continuous. Ignorance in construction defects management from the beginning cause big losses in the project. Excessive major defects happened in the building will incurred high maintenance cost for rectification works and directly raise the construction cost. In fact, this high maintenance cost may be eliminated by only investing small amount of money at the initial stage if the designers really understand the root cause of this matter.Moreover, there were many problem faced by the developer, contractor and other building professional when they deal with defects. This problem includes those from management side and construction side. If these problems left without any solution, it will make the defects problem become worse. Too many defects in the building will lower down the aesthetic value of the building and the occupier in the building will feel uncomfortable. Hence, construction defects management is very vital and essential in every project to ensure the success completion of the building.

2. BACKGROUND

2.1 Construction Defects

Robert (2007) stated that the trial courts have recognized that construction defects are tangible and can typically be grouped into the following three major categories:

a) Design Deficiencies

Poor performance of the design professionals, such as architects or engineers. The motivation for the design may be form, function, aesthetics, or cost considerations, but the completed design could result into a defect. Typical problem identified are related to roofing which due to their design complexity, pitched or flat, which make them prone to leaks. A majority of roofing problems are a direct result of the improper specification of building materials, which can result in water penetration, intrusion or other problems.

b) Material Deficiencies

Inferior building material used might cause significant problems and fail to perform even when it had installed properly. Leaking windows are a common defect and prevention requires good workmanship. Window leaks can result from many things including, rough framing not being flush with outside at openings, improperly flashed windows, improperly applied building paper, window frame racked during storage or moving, lack of sheet metal drip edge above window header and other causes. Common manufacturer problems with building materials can include deteriorating flashing, building paper, waterproofing membranes, asphalt roofing shingles, particle board, inferior drywall and other wall products used in wet and/or damp areas, such as bathrooms and laundry rooms.

c) Construction Deficiencies (Poor Quality or Substandard Workmanship)

Poor quality of workmanship often results in water intrusion through some portion of the building structure. Cracks in foundations, floor slabs, walls, dry rotting of wood or other building materials, termite or other pest infestations, electrical and mechanical problems, and plumbing leaks.

d) Subsurface / Geotechnical Problems

Soil investigation has to be done properly to determine the type of soil before any development can begin. This important to ensure the stable and strong foundation is provided. Moreover, problems such as subsidence, expansion, slope failure, flooding and landslide will take place if the subsurface condition is not well determined. These types of conditions typically lead to cracked foundations, floor slabs, and other damage to a building. A worst-case scenario in some instances could render a building uninhabitable, as well as uninsurable.

2.2 Major Construction Defects

There are many types of existed construction defects which include structural, architectural, and mechanical and electrical defects. Some of the defects are occurred in less number, however some of them occur so frequently. This type of defect known as major defects. Web dictionary also defined major defect as defects which cause serious malfunctioning of a product. Additionally, defects are common in construction projects and it will definitely happen no matter how much care design professional has taken. But, what matter the most is when the defects occurs so excessively. These might lead to severe problems as the result of major defects. One of defects that listed under major defects was water issues related problems such as water intrusion and water leaking.

Chew et al. (1993) describe many defects have escalated with the growing demands of buildings. The fragmented nature of the building and the construction process was found to be the major contributing factor.Building defects was an issue if the defects are too excessive which include rain penetration, condensation, cracking and detachment. Wet area of high rise building in Singapore has recorded escalating numbers of defects and has become the bane of the effective building maintenance within the limited budget set by the building's owners. Average of 35%-50% of annual maintenance cost needed for building maintenance from the total maintenance cost. This matter indicates that excessive defects problem may lead to the increase in project cost.According to research done by Chew et al, (1993) on maintainability of wet area of non residential building (National University of Singapore). From their study, they discussed on some important findings which are the implication of key factors of building maintainability and the occurrence of 14 most common defects found in wet area. Problems area evaluated includes water leakage from ceiling, staining or discoloration, paint defects, fungi and algae growth, pipe leakage and corrosion, and cracking of concrete. From this study, it can be concluded that water issues problem was listed as one of the major defects which normally occurred in the building. This can be proven by the figure below which shown the fourteen (14) major construction defects that has been identified.

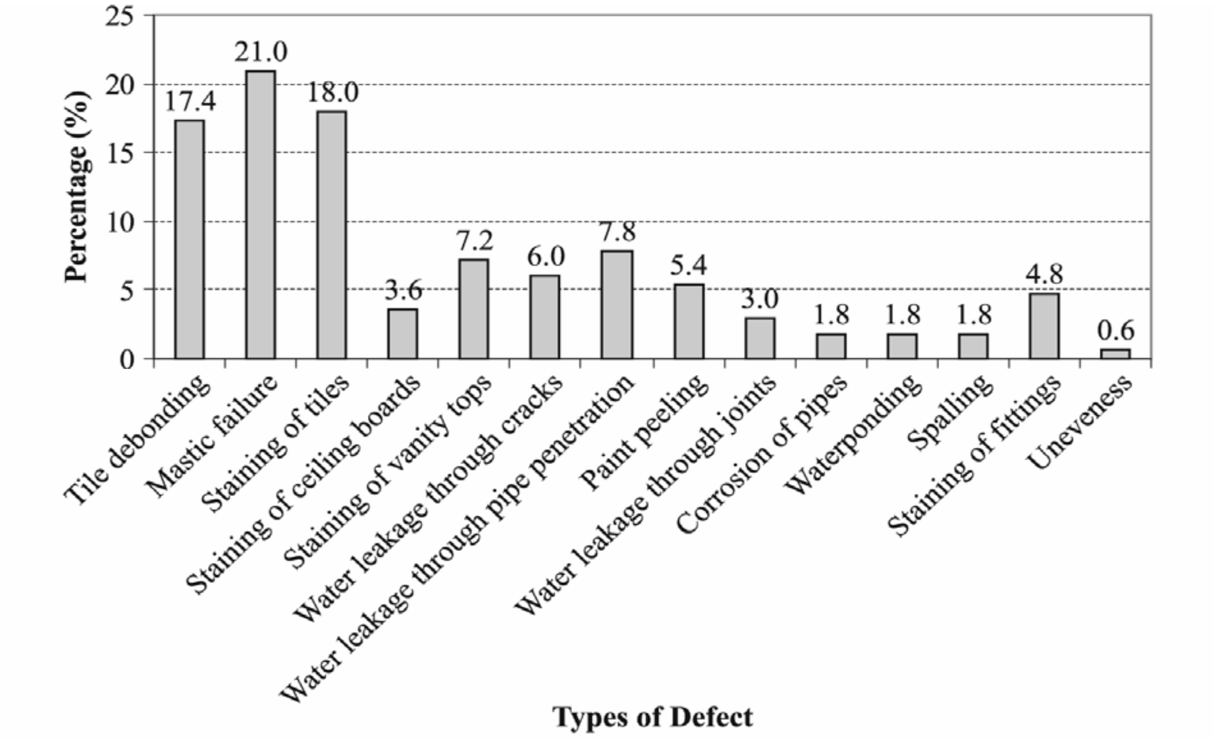

Figure 1. *Frequency of the major construction defects identified*.

Figure 1 shows the frequency of the major construction defects identified in wet area. Water leakage defects are listed in 3 categories out of 14 most major defects identified in wet area, these includes water defects through cracks, pipe penetration, and joints.They also did an analysis of variance (ANOVA) to test the equality of different population means and to examine the extend of defects in different types of building. This statistical analysis was conducted with the help of Statistical Package for Social Sciences (SPSS).

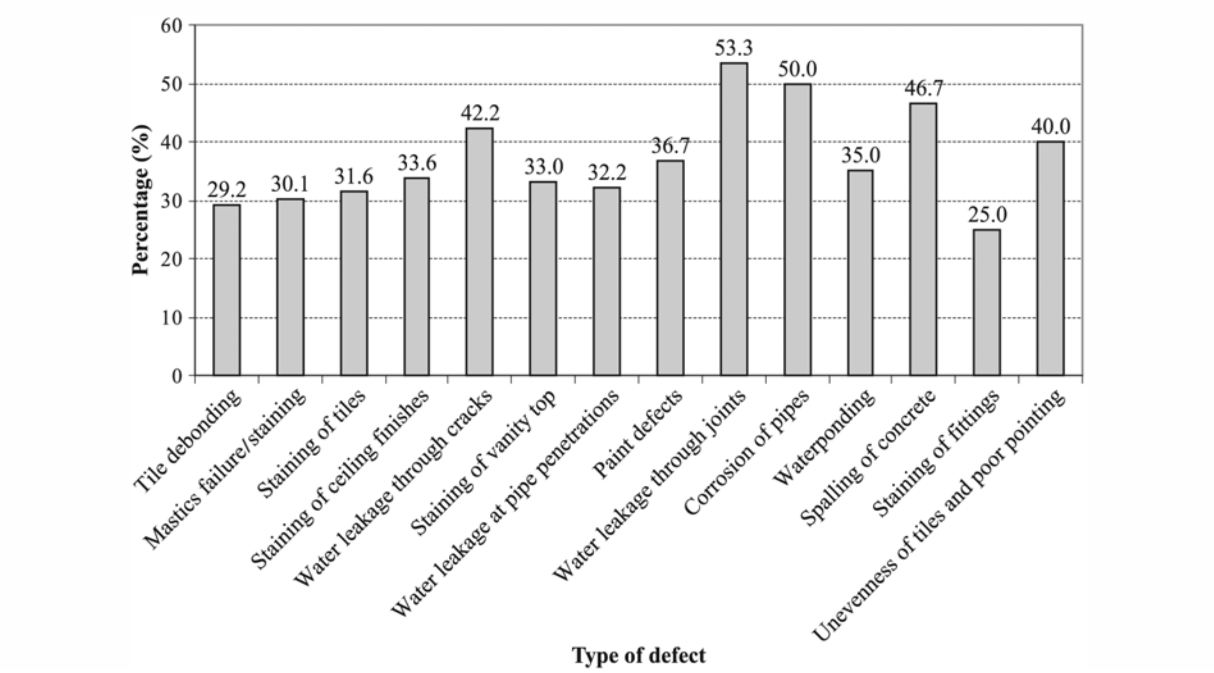

Figure 2. *The extend of defects*

From the Figure 2 above regarding the analysis of the extend of defects, water issues related defects did recorded high value of extend which means these defects may grow severe if no correct action taken by the owner. This study has shown that water leakage extend of defects has ranked the highest, followed by corrosion of pipes and spalling of concrete. By these findings, it may serves as benchmark for building professional to understand the defects at the initial stage to prevent and minimize its occurrence later and hence reduce wastage on the necessary maintenance work.

Wong et al.(2005) had done a research on water seepage in multi storey buildings had addressed the major issues and suggests the solution to solve the water seepage and health related problems in high rise building. From this study, they concluded that water seepage complaint was difficult to be handled by the government. There was a consent that causes of the defects are very difficult to be traced and lack of cooperation among individual owners lead to building neglect. Moreover, effective research on "Performance of Constructed Facilities" done by Chong (2006) had looked on the failure mechanism that caused design related-defects and the design parameters that could prevent these defects. It was found that three most design related failure causes were load and moisture from wet area, weather impact and impact from occupants after a 9-month building survey was conducted on 74 buildings. They also found that insufficient consideration for these failures causes to be the key in preventing these defects. The design strategies that could successfully prevent triggering these defects include aligning material performance against adverse weather condition, preventing impacts from occupant and loads and also, preventing water leakage, improve specification and improved design clarity. From this research, they conclude that defects continue to be repeated as designers failed to obtain important feedback from property managers. Designer also should encourage themselves to explore new ideas and keep the database whereby defects could be traces and mistakes avoided in their next design.

3. METHODOLOGY

3.1 Research Methodology

The research methodology adopted for this projects are shown in Figure 3. The methodology was divided into three phase which are preliminary phase, data collection phase, and data analysis phase. In preliminary phase, the related problem will be identified by go through the literature review and research paper of previous study. Next, the primary and secondary data will be collected to be analyzed in the data collection phase. Finally, all the data collected will be analyzed to achieve the objective and the required results.

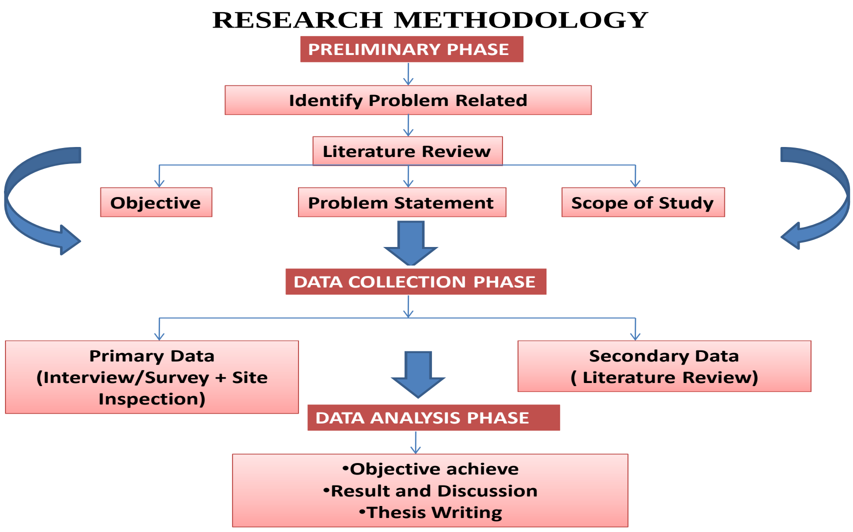

Figure 3. *Research Methodology*

3.2 Interview and Survey Questions

The secondary data was obtained by conduct the case study is by interviewing, distributing survey question, and site inspection. An informal interview has been arranged with the project manager, contractors and parties which involved in this project during the construction phase to identify in detail about the construction defect, how the defect occurs and solution taken. Project Managers also will be interviewed to obtain his opinion regarding management approaches of overcoming construction defects problems. On top of that, architect, interior designer, and other professional firm that involved in the construction project will also be interviewed to gain a broad understanding on construction defects regarding on their respective fields. Among the contractor, developer, architect firm, quantity surveyor and other firm that involved in the said project. From the survey, the issues, problem faced and factor that related to defects may be analyzed. The analysis will adopted qualitative measurement by the Average Index rating where respondent has been asked to answers the surveys according to the level of agreement which rating from:

1	Strongly disagree
2	Disagree
3	Moderately
4	Agree
5	Strongly agree

From the survey, the analysis of the defects problem will be completed by using the Average Index formula which is shown in Equation 1:

$$\text{Average Index (AI)} = \frac{\beta * n}{N}$$

Equation 1

Where :- β is the weighing given to each factor by respondent

n is the frequency of the respondents

N is the total number of respondents

With the rating scale as below (Majid & McCaffer, 1997)

1 = Never/Totally disagree (1.00 $\leq$ Average Index < 1.50)

2 = rarely/disagree (1.50 $\leq$ Average Index < 2.50)

3 = Sometimes/Neutral (2.50 $\leq$ Average Index < 3.50)

4 = Often/agree (3.50 $\leq$ Average Index < 4.50)

5 = Very often/strongly agree (4.50 $\leq$ Average Index < 5.00)

3.3 Site Inspection

The author participates in site inspection held by the developer. The defect inspection involves the building owner, contractor, architect, and consultants. From the defect inspection, all parties involved analyzed and discussing the existing defect occurs in the building. The argument happens on how the rectification method will be and when it can be taken. The inspection was done for each floor levels. When defects were detected, it will be noted on the floor plan drawings for the reporting purpose.

4. Result and Discussion

4.1 Site Defect Inspection

From the case study of the said project which the author had analyzed, the Author had found that there were many commonly major construction defects which usually occurred in new building. The construction defects include defects in structural, utilities and appliances, building codes and construction, internal and external building, and facade.

As stated in the scope of study, the author identifies the commonly major construction defects. The elaborations below indicate the certain construction defects exist at the case study area during the inspection of defect work attended by the client and developer.

- RCC Construction Defects

It is necessary to achieve the perfect quality of RCC after castig.RCC can fail due to inadequate strength or reduction of durability due to improper casting, and some of the structures are not properly watertight. These lead to the intrusion of water. All structural members as beams, slabs, columns, retaining walls must retained their full strength and need to be water tight.

- Windows

Defects associated with windows most often have to do with leaks, which can be a result of poor installation. This means the windows were not flashed properly, or the window product itself is defective and leaking. In many homes windows are difficult to operate. This can be a sign of soil settlement, structural deficiencies or just a bad product. Building inspector investigates the windows by performing several different tests to determine whether or not a particular window is leaking. If it leaks, we then remove a small area of stucco or siding to better understand the cause of the leak

- Brick walls-substandard

Quality of masonary work, poor workmanship and lack of supervision will cause the cracks in masonary work,give dampness to the structure, create an unleveled surface as shown in Figure 4.

Figure 4. *Brick wall cracks*

- Plumbing and pump defects

Plumbing defects are commonly found include dripping faucets, leaking fixtures, slow drains. There are several pressure inlets and outlets in sewer and water lines consisting of several plumbing fixtures, T bends, Lbends, several traps, and so on. All this sources and junctions are to be properly sealed, ass otherwise there is every possibility of corrosion as shown in Figure 5 and Figure 6.

Figure 5. *Pump in water pump room leaks.(Image courtesy from MRCB Eng Sdn Bhd)*

Figure 6. *The sprinkler point at office area broke and flooded the whole office levels.(Image courtesy of MRCB Eng Sdn Bhd)*

- Floor trap clogging
- Ceiling access opening off position
- Water seep through the slap(pipe sleeve)
- Ceiling stains

Caused by past or present leaks, ceiling stains are very common. It can be difficult to tell whether the stains are from leaks still present, or were caused by leaks which have since been repaired.

- Electrical hazards

Most common in older homes, but often found in newer homes as well. Electrical hazards come in many forms, from ungrounded outlets, substandard use of materials, incorrect wiring by the electrician

  - Power surges. Causes flickering of lights.
  - Breakers tripped.
  - Bulb burn out on power up.
  - GFCI breakers are not wired. Incorrect wiring.
  - Receptacles do not work.
  - Switches do not work
  - Inadequate loading over the entire building.
  - Burning smell

- Rotted wood caused by being wet for extended periods of time, most commonly found around tubs, showers and toilets inside.
- These are the set of miscellaneous defects.
  - Mold growth. Growth throughout the home from various causes.
  - House built with defective products (Masonite). Siding, plumbing, roofing materials many with recalls or lawsuits against them.

From the Inspection of Defects Work form collected, all major and minor defects are summarized in this form. Since the author study only limited to major construction defects which involved water issues related problem, hence the defects list are simplified into major construction defects occur at the case study area.For the month of April 2010, there were 26 major defects reported and 12 cases out of 26 are related to water issues problem. This give the percentage of water issued related defects is 46.15%, approaching nearly half of the total defects occurred.(refer to the appendices).The result obtained indicates the percentage of water issues related defects which exist at the case study area. Even though the results are quite apparent, it is proven that water issues related defects are creating major problem in the building. Thus, detail survey need to be done to obtained more accurate and competitive result.From the survey question distributed to all the respondent, the outcomes are summarized in Table 1.

***Table 1**. Problems and Issues Related to Defects*

No.	Problems and issues related to defects	Average Index	Rank
1	Clogging Problem(Floor Traps, Sanitary Fittings and others)	3.9	4
2	Waterproofing Failure	4	3
3	Mechanical and Electrical Equipment Breakdown	3.3	7
4	Water seeps through structures(wall, slab, floor and others)	4.6	1
5	Leakage(Sprinkler Pipe, Pipe, Plumbing, interior and others)	4.1	2
6	Construction Joint failure	3.4	6
7	Structural Cracks	3.8	5
8	Unevenness(wall, column, slab and others)	3.1	9
9	Stained mark(bubbling plaster/water stain and others)	3	10
10	Chipped or Spalling	3.2	8

The problems and issues related to defects are presented by the Figure 4.4 below to give clear view of the Average Index value

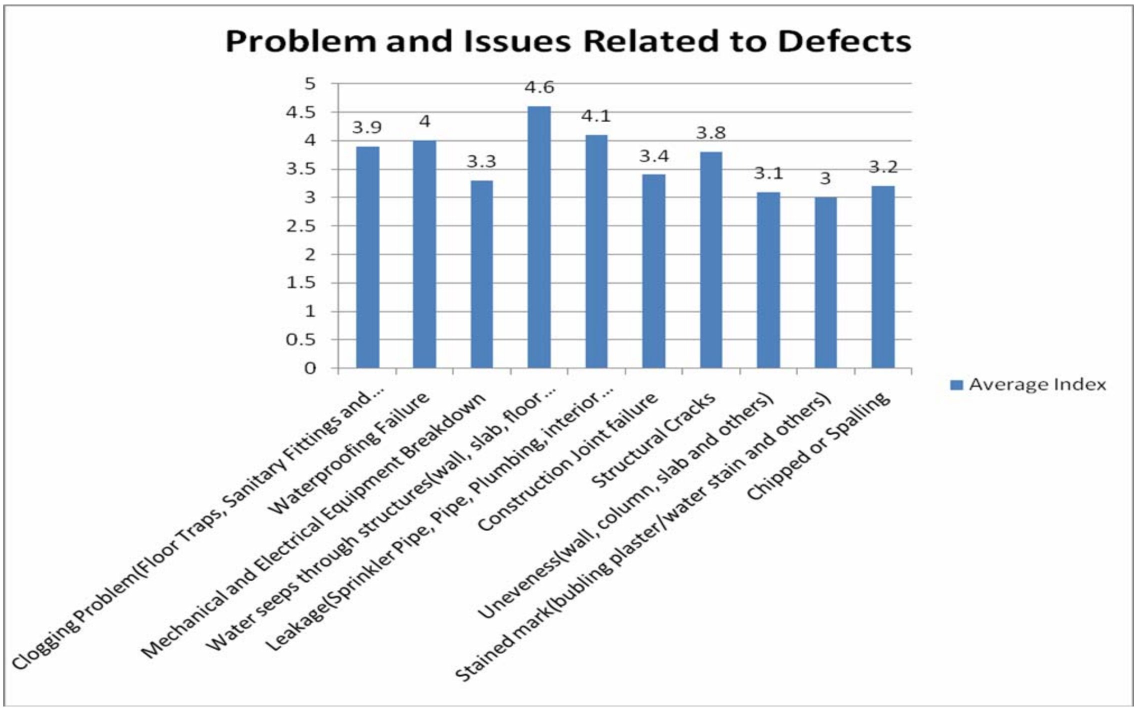

***Figure 6.** Average Index on Problem and Issues Related to Defects*

According to Table 1 and Figure 6, all the respondents agrees that the entire problems and issues related to defects which listed are the major defects that happen at the site, where average index from 3 to 4.6 was achieved. Noted also that water related defects are the most commonly major defects that happened at the site where it achieved Average Index (AI) from 3.9 to 4.6 which includes under:

- Water seeps through the structures(wall, slab floor)
- Leakage problem(Pipe, plumbing, interior leakage)
- Water proofing failure
- Clogging problems(Floor Traps, Sanitary Fittings and others)

The occurrence commonly major defects in the building bring many problems to the building professionals. Thus well defects management have to be adopted by these parties.

5. CONCLUSIONS

As the conclusion, this study has achieved all the objectives that have been stated earlier. Most of the respondents agree that the entire problems listed are the major defects that occurred at the building. Out of that, water defect related issues rated as the major defects which mostly happened with Average Index value from 3.9 to 4.6 was obtained. Moreover, this study also achieves the objective to determine the problems faced by the building professional. All the respondents agreed that most of the problems listed are really the problems which they faced except for unsuitable rectification works adopted. These problems have to be solved and well manage in order to reduce defects.

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ESTABLISHMENT AND DEVELOPMENT OF IBS IN MALAYSIA

Mohammad Abedi[1], Mohamad Syazli Fathi[2] and Abdul Karim Mirasa[3]

[1,2,3] UTM RAZAK School of Engineering & Advanced Technology, Universti Teknologi Malaysia, International Campus, Kuala Lumpur, Malaysia

[1]abedm1385@yahoo.com, [2]syazli@ic.utm.my, [3]abdkarim@ic.utm.my

ABSTRACT: Government of Malaysia has done a lot of effort to promote the usage of Industrialised Building System (IBS) as an alternative construction methods compare to conventional building system which is a more labour intensive. This paper describes the experiences of Malaysia in the adoption of industrialised building system (IBS) from the first level of establishment until its development along with various IBS definitions and categories. A comprehensive literature review from various sources was made to carry out the study. The idea of using IBS in Malaysia was first developed during the early sixties when the Minister of Housing and Local Government visited several European countries and evaluated their building systems performance. Then, in 1964, the government took a significant decision to implement one pilot projects using IBS concept which was launched on 1966. The findings of this research illustrated the timeline of IBS establishment and development in Malaysia along with the significant consequences of IBS implementation in construction industry. Therefore, it is hoped that the finding of this research could assist professional parties in construction industry in providing a better knowledge ground for improving decisions making to achieve the success of IBS construction projects implementation that could be along with attaining the project objectives in terms of predetermined objectives that are mostly within the time, specified budget and standard quality.

Keywords: Industrialised Building System, Malaysia, IBS Roadmap, Project Objectives, CIDB.

1. DEFINITION OF IBS

Government of Malaysia has done a lot of effort to promote the usage of Industrialised Building System (IBS) as an alternative construction method. Since the first project of IBS in year 1966, there has not been one absolute definition on Industrialised Building System (IBS) that could be describes the entire building construction system. However, there are several definitions by researchers who studied into building construction emphasizing on the concept on off-site construction (Pan, 2008) off-site production (Blismas, 2006), industrialized and automated construction (Warszawski, 1999), off-site manufacturing, prefabricated building, pre-assembled building (Gibb, 2003). Furthermore, IBS could be more elaborated by other definitions such as pre-cast building, pre-cast construction, non-traditional building, innovative building solutions and a Modern Method of Construction (MMC) (Goodier et al., 2007). Different definitions of IBS have been established by many researchers as illustrated in Table 1.

Table 1. *Definitions of IBS*

Author	IBS Definition
Chung (2007)	Mass production of building components either in a factory or at site according to specifications with standard shapes and dimensions and which are then transported to the construction site to be re-arranged with certain standards to form a building.
Rahman and Omar (2006)	A construction system that is built using pre-fabricated components. The manufacturing of the components is systematically done using machine, formworks and other forms of mechanical equipment.
Lessing et al., (2005)	An integrated manufacturing and construction process with well planned organization for efficient management, preparation and control over resources used, activities and results supported by the used of highly developed components.
Gibb (1999)	Preassembly for a given piece of work; the organization and completion of a substantial proportion of its final assembly work before installation in its final position including forming any temporary work or pre-assembly and it can also be carried out on or offsite which would involve the standard coordination.
Trikha (1999)	Two definitions have been created: 1) A system of construction that has been made to be mainly industrialized in its manner and process, such as the manufacturing of automotive components and furniture. 2) A system in which concrete components are prefabricated at site or in a factory and are assembled to form the structure with minimum on-site construction.
Warszawski (1999)	A set of interrelated elements that act together to enable the designated performance of the building.
Esa and Nurudin (1998)	A continuum beginning from utilizing craftsmen for every aspect of construction to a system that make use of manufacturing production in order to minimize resource wastage and enhance value for end users.
Parid Wardi (1997)	A system which uses industrialised production techniques either in the production of components or assembly of the building or both.
Junid (1986)	A process, by which components of a building are conceived, planned and fabricated, transported and erected at site. The system includes a balanced combination between software and hardware component. The software element includes system design, which is a complex process of studying the requirement of the end user, market analysis and the development of standardized components.
Dietz, A.G.H (1971)	Total integration of all subsystem and components into overall process fully utilizing industrialized production, transportation and assembly technique.

The most significant, feasible and practical definition of IBS in Malaysia was introduced by the Construction Industry Development Board in Malaysia's Roadmap of IBS (CIDB, 2003). CIDB defines IBS as a construction technique in which components are manufactured in a controlled environment (on or off site), transported, positioned and assembled into a structure with minimal additional site works. This research will use the definition by CIDB in categorising the building construction system. However, Industrialised Building System (IBS) is not new to the construction industry because it has just again become feasible and visible in Malaysia as a valuable solution to improve the construction industry mainly in terms of efficiency and productivity. In a larger view and open minded perception, IBS is further than implementing the construction project through prefabricated methods. According to Kamar et al., (2009), IBS is about the changing of conventional mindset, enhancing the capability, competency and value of human capital, developing better cooperation, team working and trust among the parties in construction industry, promoting intelligibility, innovation, transparency and most significantly the high integrity that will eventually enhance the productivity and efficiency within the construction industry. IBS has been identified as a potential method to improve overall construction performance in Malaysia in term of quality, cost effectiveness, safety and health, waste reduction, efficiency and productivity.

There are many benefits of Industrialised Building System (IBS) as a modern method of construction such as reduction in construction time, less site materials, better site management, minimal wastage, cleaner and neater environment, controlled quality, reduction of labour intensity, construction standardization, quality improvement and lower total construction costs that will eventually produce better products for the parties such as clients and contractors in construction industry (Kamar et al., 2009). The Malaysian government have implemented a lot of various significant efforts in to bringing the IBS for all professionals involved in the construction industry. One of these significant efforts is the establishment of IBS Roadmap 2003 -2010 that has been endorsed by the Government to be the blueprint document for the industrialisation of the Malaysian construction sector. The effort to promote the usage of IBS as a valuable (efficient and effective) alternative compare to conventional and labour intensive construction method should be more emphasized by the CIDB of Malaysia. Also, Kamar et al., (2009) asserted that although members of the industry are open to the idea, a major portion of the industry stakeholders are indifferent, perhaps due to resistance towards change, insufficient information and lack of technology transfer methods to support the feasibility of change to IBS. Successful IBS projects are Sekisui Home (Japan), Living Solution (United Kingdom), Open House (Sweden) and Wenswonen (Netherlands) (Oostra & Joonson, 2007).

According to Astrand (2002), the existing process of selecting innovative building systems or IBS can be divided into two main stages which are pre-occupancy and occupancy. Abdullah and Egbu (2010) stated that the decisions in pre-occupancy stage, (pre-design stage and design stage) are significantly valuable in terms of efficiency, effectiveness and productivity that will be made by stakeholders. These valuable decisions can be considered as combined, shared, united and consensus decision-making, because there is an efficient and well organized cooperation between the design teams and clients that will eventually discuss the best alternative for the building system to be implemented in the construction project. Furthermore, providing efficient, proper and adequate information beside the project knowledge will lead us to create and develop the best decision-making process. Knowledge based decision-making can reduce uncertainty for successful implementation of risk management that will facilitate the construction project parties to achieve the project objectives in terms of time, cost and quality to eventually enhance the success of construction projects.

2. ESTABLISHMENT AND DEVELOPMENT OF IBS IN MALAYSIA

IBS in Malaysia has been established, introduced and applied in construction industry in order to deal with a growing demand of affordable housing, solving issues associated with foreign workers and improving quality, efficiency and productivity of construction industry. Nowadays, IBS has evolved and implemented in hybrid construction projects to build national landmark as significant valuable national assets. From the comprehensive literature review, establishment and development of IBS have been illustrated through the timeline as shown in Figure 1.

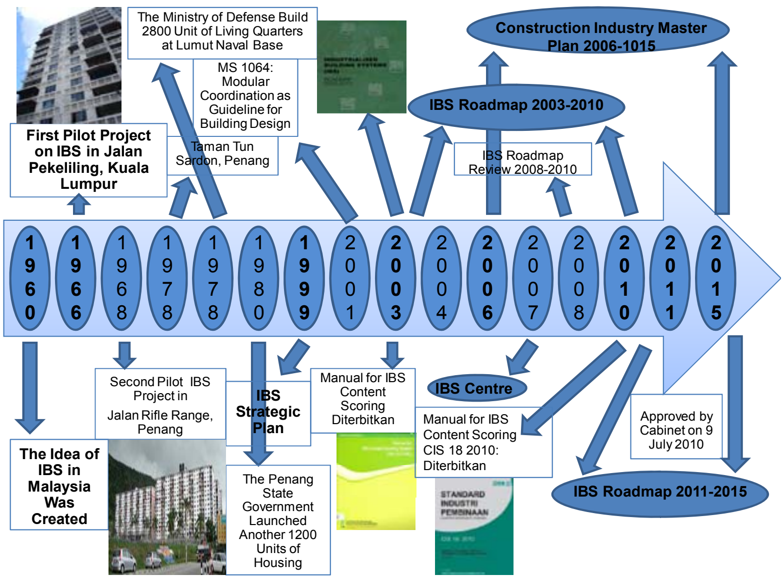

***Figure 1**. Establishment and Development of IBS Timeline in Malaysia*

According to figure 1, various efforts have been adopted since the establishment of IBS in Malaysia on the early 1960's. Furthermore, these efforts has been reviewed and improved for enhancing the efficiency, effectiveness and successful implementation of IBS until 2015.

The usage of IBS in Malaysia has begun since early 1960's in a low cost housing scheme when Ministry of Housing and Local Government of Malaysia visited several European countries and evaluated their housing development program (Thanoon et al., 2003). After their visit in 1964, the Government had launched first pilot project on IBS to speed up the delivery time and built affordable and quality houses. Almost, 22.7 acres of land along Jalan Pekeliling, Kuala Lumpur was dedicated to the project comprising seven blocks of 17-storey flats (3,000 units of low-cost flat and 40 shops lot). This project was awarded to JV Gammon & Larsen and Nielsen using Danish System of large panel pre-cast concrete wall and plank slabs. The project was completed within 27 months from 1966 to 1968 including the time taken in the construction of the RM 2.5 million casting yard at Jalan Damansara (CIDB, 2006; CIDB, 2003 and Thanoon et al., 2003). In 1968, the Government of Malaysia launched a second housing project which was a six block of 17-storey flats and three blocks of 18-storey flats at Jalan Rifle Range, Penang. The project was awarded to Hochtief/ Chee Seng using French Estoit System (CIDB, 2006; CIDB, 2003 and Din, 1984).

According to the two pilot projects, a performance comparison between the IBS system and conventional system has been carried by Thanoon et al., (2003) in terms of cost, productivity, and quality. It was discovered that the first pilot project incurred 8.1% higher cost than a similar building using conventional construction method, while the second project was 2.6% lower than conventional construction method. In term of construction speed, both projects required 27 months to complete, inclusive of time required to set up the recasting factories. Also, it was found that the quality of building finishes was better than the conventional construction method.

Precast concrete is one of the main IBS category built in Malaysia since 1960s. However, according to Rahman and Omar (2006) these buildings were normally associated with pre-fabricated mass construction method, low quality buildings, leakages, abandoned projects, unpleasant architectural appearances and other drawbacks. Among the earliest housing development project using IBS was at Taman Tun Sardon, Penang in 1978, (1,000 units of five-storey walk up flat). IBS pre-cast component and system in the project was designed by British Research Establishment for low cost housing in tropical countries (BRECAST system). A similar system was constructed almost at the same time at Edmonton, North London and about 20,000 BRECAST dwellings were constructed through out UK from 1964 to 1974 (CIDB, 2006).

Rahman and Omar (2006) also indicated that due to the poor architectural design, the old pre-fabricated buildings have given the public, bad impression about precast concrete. There have been quite a number of cases where the use of IBS had lead to such drawbacks such as, in the case of Pekeliling Flats in Kuala Lumpur and Taman Tun Sardon, Gelugor, Penang. These two early pre-fabricated flats were constructed in mass to produce low cost accommodation for lower income groups. In the case of Taman Tun Sardon the design was very basic and not considering the aspect of serviceability such as the need for wet toilets and bathrooms. Lacking in this design consideration leads to problems of leakage that becomes the common issue with precast buildings. In addition, in many cases the low cost housing was not maintained properly, thus contributing further to the poor image of IBS buildings. Other important aspect is the quality inspection which in housing construction is one of the major factors that general contractors have to consider for improving customer satisfaction and raising their intake in the industry (Kim, Oha, et al., 2008).

In 1978, the Penang State Government launched another 1200 units of housing using prefabrication technology. Two years later, the Ministry of Defense adopted large prefabricated panel construction system to build 2800 unit of living quarters at Lumut Naval Base (Trikha and Ali, 2004).

Nevertheless, the industrialisation of construction at its beginning stage on the early 1960's was never sustained. This issue could be due to failure of early closed fabricated system which made the professional parties in construction industry afraid of changing their construction method. Furthermore, other overseas systems that were introduced during 60s and 70s were found unsuitable for the Malaysia climate and social practices. Hence, newer technologies were constantly being introduced to the market such as wet joint systems that were identified to be more suitable choice for Malaysia tropical climate. For instance, it was better to utilise the bathroom types which were comparatively wetter than those types in the European countries (CIDB, 2005). In the period of 1980 till 1994 there is a huge gap of discontinuous and decreasing the IBS utilization towards developing and implementing the IBS. This could be because of problems in the leakage of IBS design that eventually contributing to poor image of IBS buildings for construction industry. Also, this gap could be due to lack of clients and contractors trust and interest to these IBS systems. IBS was not popular in 1980 to 1994. However, IBs become more popular in 1990's when many Malaysian infrastructure and mega projects were build especially for the Commonwealth games in 1998. As a result in Malaysian construction industry during 1994-1997, pre-cast, steel frame and other IBS were used as hybrid construction method to build national landmarks such as Bukit Jalil Sport Complex, Kuala Lumpur Convention Centre, Lightweight Railway Train (LRT) and Petronas Twin Towers (Kamar et al., 2009 and CIDB, 2006).

According to Construction Industry Development Board (CIDB, 2005) based on the resolution which was made during the Colloquium of Industrialised Construction System in 1998, the CIDB eventually formed the IBS Steering Committee on 1999 for the effort to bring to the forefront all the IBS related issues in a framework to promote the greater usage of IBS in the construction industry and to drive the industry forward. The IBS Strategic Plan in 1999 was published as a result for establishment of this committee. After this significant effort in 2001, one early initiative was the introduction of the Modular Coordination Guideline for Building Designs ("MS1064") which is essential for the adoption of IBS in the industry. This guideline was a public-private joint effort towards standardisation through modular coordination. However, this guideline was not made compulsory. The Construction Industry Master Plan 2006-2015 (CIMP 2006-2015) has been published in December 2006 as means to chart the future direction of the Malaysian Construction Industry. The importance and effort to promote IBS is highlighted under Strategic Thrust 5: Innovate through R&D to adopt new construction methods such as IBS in the Construction Industry Master Plan 2006-2015 (CIDB, 2007).

These various significant efforts by the government to promote the usage of IBS as an alternative compare to the conventional method did not made significant progress. Therefore, the IBS Roadmap 2003-2010 was developed and published to steer the direction of IBS implementation and promotion activities, guide the practitioners and policy makers on IBS related issues in ensuring the global competitiveness of Malaysian construction players through the efficient and effective usage of IBS. It is a systematic and coordinated blueprint which is mainly focused towards achieving totally an industrialized construction industry for Malaysia and achieving open building systems concept by the year 2010. The formulated roadmap is a comprehensive document that divided the IBS programme into a 5-M strategy (five main focus areas) that reflects the inputs needed to drive the programme which are Manpower, Materials, Management, Monetary and Marketing (CIDB, 2003).

The various efforts by the government to promote the usage of IBS such as IBS Roadmap 2003-2010 as a significant valuable alternative for construction industry on the end of 2010 did not achieve the goals of this roadmap. According to CIDB in Malaysia (CIDB, 2011), only one KPI (Monetary) out of the 5 measurable KPI's was achieved. The KPI's was categorized to Manpower (Reducing the percentage of foreign workers), Monetary (Enhancing the percentage of governmental construction projects using IBS), Materials (Increasing the numbers of IBS Manufacturers), Bumiputera Development (Increasing the numbers of Bumiputera IBS Manufacturers), Bumiputera Development (Increasing the numbers of Bumiputera IBS contractors or Installers). Hence, the new IBS Roadmap 2011-2015 is developed by CIDB under consultation with the industry players to chart the way forward for IBS industry. On the other hand, Construction Industry Development Board (CIDB, 2011) have emphasized on four policy objectives including quality, efficiency, competency and sustainability leading to a sustainable IBS industry that will eventually contribute to the competitiveness of construction industry in Malaysia.

The main goals for this IBS Roadmap 2011-2015 were encapsulated below:

i. To sustain the existing momentum of 70% IBS content for public sector building projects through to 2015.

ii. To increase the existing IBS content to 50% for private sector building projects by 2015.

3. CLASSIFICATION OF IBS SYSTEM IN MALAYSIA

In Malaysia, Construction Industry Development Board (CIDB, 2003) has classified the IBS system into 5 categories as follows:
i). Precast concrete framed buildings,
ii). Precast concrete wall system,
iii). Reinforced concrete buildings with precast concrete slab,
iv). Steel formwork system and v). Steel framed buildings and roof trusses.

4. ADVANTAGES AND DISADVANTAGES OF IBS

Despite its plausible advantages and systematic implementation plan established trough the IBS Roadmap, numbers of barriers were identified as being potential hurdles to the implementation. One of the most important barriers to IBS Implementation in Malaysia is Awareness and Knowledge (Kamar et al., 2009). According to IBS Roadmap Review (CIDB, 2007) report, the adoption of IBS in Malaysia is a client driven. Client with a good knowledge and awareness of IBS benefit will surely encourage appointed designers to design building according to IBS. However, lack of awareness program to understand client needs and giving correct information on IBS has contributing to a lack of interest from the client and decision makers (Rahman & Omar, 2006). Relatively, the low labor cost in Malaysia is the main root cause of the construction industry failing to reform and being complacent with the current systems of IBS, which if implemented will eventually enhance the level of productivity, efficiency, quality and safety (CIDB; IBS Roadmap Review, 2007). However, to be competitive at the international level and become globalized, it is important for the Malaysia construction industry to evolve, be well organized and prepared. This aim will be facilitated and achieved through enhancing the productivity, efficiency, quality and safety of construction industry. The advantages and disadvantages of IBS can be also compared and justified with the conventional system, whereas, conventional construction methods that have proven to be more time consuming , wasteful, dangerous and messy are illustrated in table 2.

Table 2. *Advantages and Disadvantages of IBS*

Advantages of Industrialized Building Systems	1. Achieving Higher Quality through careful selection of material, use of advanced technology and strict quality assurance control (Thanoon et al., 2003).
	2. Faster Construction Time: Wisam (2005) asserted that faster completion of projects will be achieved due to advance off-site preparations and simplified installation process.
	3. High Cost Saving: Control in using materials, such as: steel, sand, and timber will contribute in savings in material cost and savings in labor cost that will ultimately result in substantial savings on the overall cost of the project (Bing et al., 2001).
	4. Reduction of Unskilled Workers and Fewer Site Workers: Warszawski, (1999) indicated that IBS and particularly prefabrication takes place at a centralised factory, thus reducing labour requirement at site.
	5. Enhancing the Social Benefits: Reducing the dependency on foreign workers and reducing the money outflow and their social problems, low quality works, delays, and diseases (CIDB, 2009).
	6. Increased Environmental and Construction Site Cleanliness: According to Wisam (2005), this could be achieved through reduction of construction material at site and reduction of waste materials at site duo to casting in factory.
	7. Increasing the Safety: Implementing IBS will promote safe and systematic factory working environment as minimal workers, materials and construction waste is required on-site (CIDB, 2003).
Dis-advantages of Industrialized Building Systems	1. Higher Initial Investment Cost: IBS requires high initial investment capital for pre-casters to purchase new machinery, mould, importing foreign technology and wages of skilled workers (IBS Steering Committee, 2006; Thanoon et al., 2003; Rahman & Omar, 2006).
	2. The Industry is Uncompetitive Due to Lack of Open Collaboration: Contractors in Malaysia are obligate to close system and getting supply from the same manufacture throughout the construction (Chung & Kadir, 2007).
	3. Specialized Skills which Require more Time and Investment: This is due to intensive training and apprenticeship such as system integrator or assemblers (Thanoon et al., 2003; IBS Steering Committee, 2006; Rahman & Omar, 2006).

5. CONCLUSIONS

Industrialised Building Systems (IBS) is a construction process that utilizes techniques, products, components, or building systems which involves mainly the prefabricated components and onsite installation. Successful and effective implementation of IBS in Malaysia construction industry can offer various benefits compare to conventional in-situ systems. Those are: the speed of construction, less wastage of materials which means cost savings, reduction of unskilled workers, better quality control of construction, increased site cleanliness and safety in construction projects. These are very important aspects in achieving the efficient and effective construction industry which will enhance the market share of construction industry as well as contributing to the Malaysian economy. The government has done a lot of efforts to enhance the current conventional, labor-intensive activities to a more technologically advanced method of construction such as by developing the Industrialised Building Systems (IBS) through the Construction Industry Development Board (CIDB). The government had efforts in creating the 1st and 2nd IBS Roadmap to put Malaysia construction industry in producing fast, cost effective and high quality construction products and able to compete with the global construction market.

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PROPERTIES OF GYPSUM PLASTERBOARDS AT ELEVATED TEMPERATURES

Hanizam Awang[1], Md Azree Othuman Mydin[2]

[1,2]School of Housing, Building and Planning, Universiti Sains Malaysia, 11800, Penang

[1]hanizam@usm.my , [2]azree@usm.my

ABSTRACT: Gypsum plasterboard based systems are among those now extensively used, as walls or ceilings. Gypsum plasterboards are principally employed as lining material in light-weight construction, which is a competent and cost effective technique of providing flexible partitioning assemblies in commercial and residential buildings. The thickness of the gypsum plasterboard lining and the configuration of the framing can be flexibly changed to meet specified fire performance requirements. The use of such systems is increasing every day and there demands to be more research on their properties and behaviour. This paper will presents the properties of gypsum board which will includes the assemblies and standard fire tests and the thermal properties of gypsum in general and includes suggested properties of gypsum by different researchers.

Keywords: gypsum board, thermal properties, elevated temperatures, fire

1. INTRODUCTION

Gypsum plasterboards are mainly used as sheet material lining in light-weight constructions, namely Light Steel Framing (LSF) and Light Timber Framing (LTF). A typical wall assembly of drywall construction is shown in Figure 1. Such walling systems consist of steel studs or wood studs with one or two gypsum plasterboards fixed to each side of the studs. The cavity between the boards are filled with insulation layers or left empty. The insulation materials commonly used in the cavity are glass fibre, rock wool and cellulose fibre.

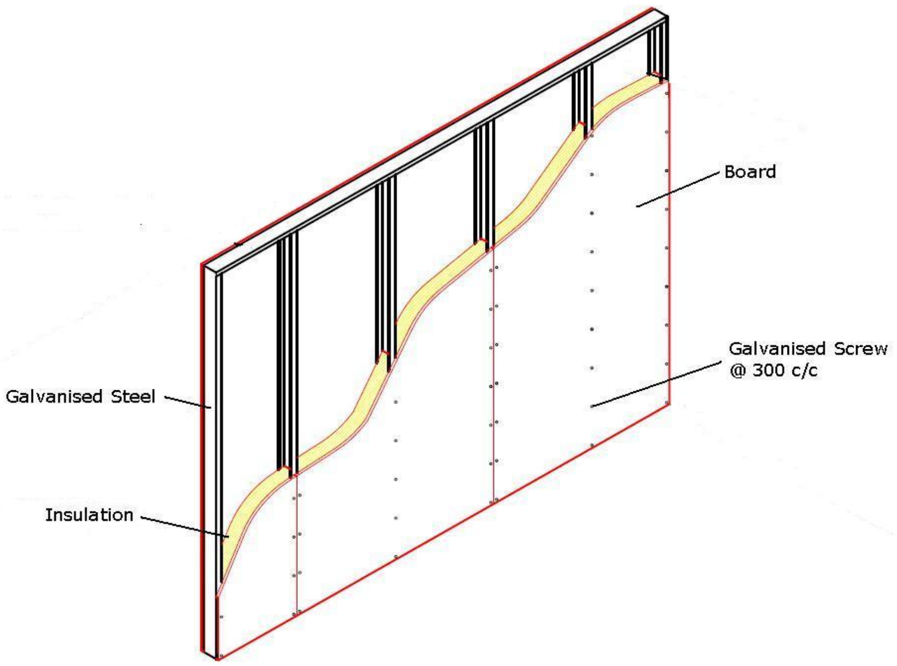

Figure 1. *A typical gypsum plasterboards wall assembly*

Each board is composed of a non-combustible gypsum core with paper-laminated surfaces which provide tensile strength to the lining. Gypsum contains chemically bound water and a small amount of free water, which play a key role in the performance of the assembly at elevated temperatures.

During a fire, when gypsum plasterboards is heated up to about 100°C, a great amount of heat is absorbed to drive off water. This process therefore delays the development of temperature rise through gypsum until the entire board is dehydrated. The consequent temperature plateau is the basis of gypsum plasterboards systems fire resistance as shown in Figure 2.

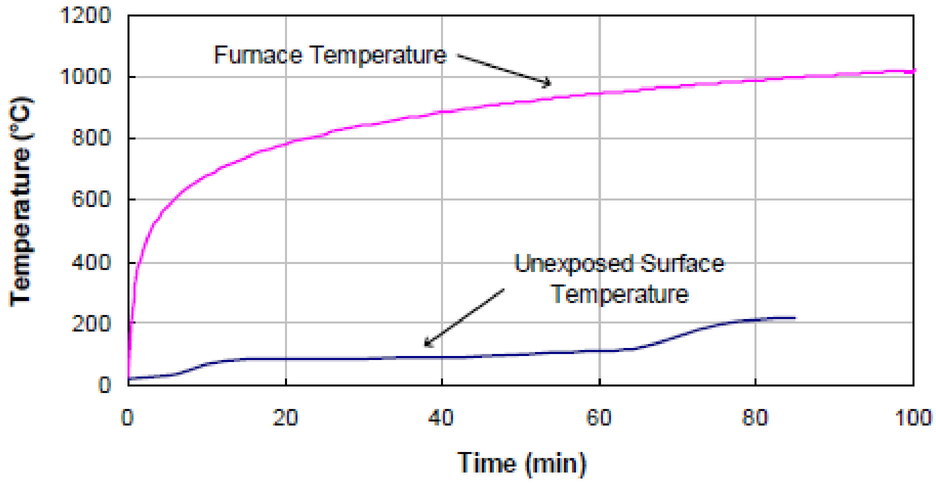

Figure 2. *Temperature development on the unexposed side of a gypsum plasterboard with 25mm thickness subjected to a standard cellulosic fire.*

2. FIRE RESISTANCE

To identify fire resistance of a system, building codes and regulations rely on standard fire test procedures. Fire resistance is then defined as the duration for which a fire protection system can endure a standard fire test until it reaches failure criteria. Failure is considered as loss of either fire separating function or load bearing function and categorized as insulation (excessive temperature rise on the unexposed surface), integrity (fire spread through fissures and openings) and stability (structural collapse) criteria.

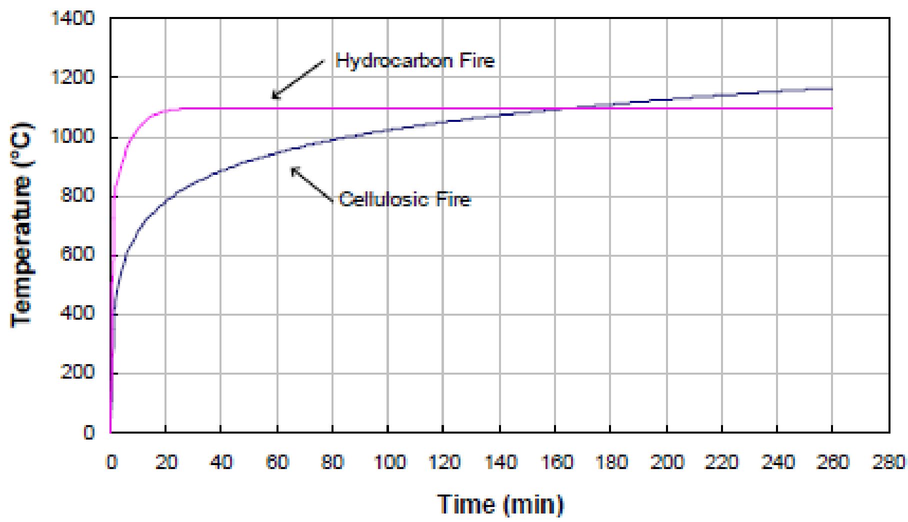

Figure 3. *Time-temperature curves for standard fires according to BS476*

In a standard fire test, the elements of interest are subjected to increasing temperatures governed by a specified temperature-time relationship (BS476, 1987). It should be noted that standard fire curves are attempts to classify construction elements, although they might not represent fire scenarios in real world. In general building uses, the cellulosic fire condition is applied. Hydrocarbon fires are more likely to occur in petrochemical industry. The two fire curves are plotted in Figure 3. As can be seen from the plots, hydrocarbon fire has a steep initial temperature rise to 1100°C, simulating fast reaction of hydrocarbons.

3. HEAT TRANSFER

Heat transfer through drywall assembly is a combination of all three modes of conduction, convection and radiation (Figure 4) as below:

- Path 1: Heat transfer from the furnace (fire) to the exposed side of gypsum is by convection and radiation, with radiation being the more dominating.
- Path 2: Heat transfer through gypsum is by conduction. However, as gypsum is a porous material, heat transfer through gypsum is a combination of all three modes: conduction through the solid and convection and radiation through the pores. Therefore thermal conductivity of porous materials like gypsum is an empirical factor that helps to describe the combined heat transfer based on Fourier law (conduction), and it might be called the effective thermal conductivity. This effective thermal conductivity can be affected by many factors, such as temperature, density, moisture content and porosity of the material. Such sensitivity then describes the diverse data reported by several research studies directed towards the determination of the effective conductivity of these materials. Developing a method to quantify the effective thermal conductivity of gypsum is one of the main objectives of this research.
- Path 3: Heat transfer from the cavity side of the exposed gypsum to the cavity gas is by convection and radiation.
- Path 4: Heat transfer from the cavity gas to the cavity side of the unexposed gypsum is again by convection and radiation.
- Path 4: Heat transfer through the unexposed gypsum is by conduction.
- Path 5: Heat transfer from the unexposed gypsum to ambient environment is by convection and radiation, with convection dominating at lower temperatures.

  It should be noted that in insulated assemblies, the insulation materials also contribute to heat transfer through the system. However, their change in volume should be considered in the analysis, since they burn away easily after a certain temperature is reached (depending on the material used).

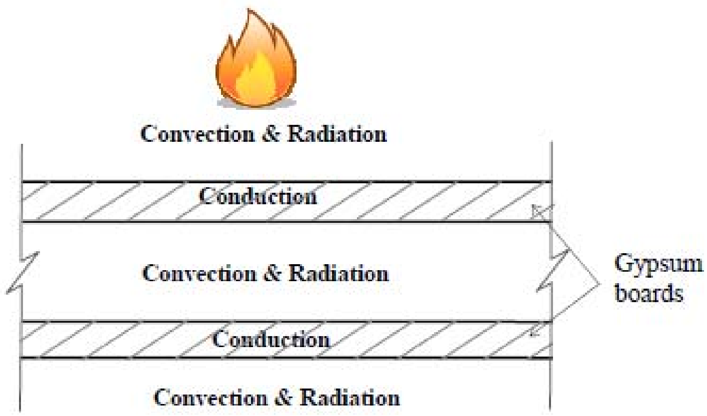

Figure 4. *Heat transfer modes through gypsum plasterboard assemblies.*

4. PROPERTIES OF GYPSUM PLASTERBOARDS AT ELEVATED TEMPERATURES

The key material in drywall construction which provides the fire resistance is gypsum. Therefore it is important to study thermal and material properties of gypsum. This section first describes some of gypsum's main thermal properties as studied by various researchers.

4.1 SPECIFIC HEAT

The specific heat of gypsum at different temperatures has been investigated by several researchers (Figure 5). Having known water dissociates from gypsum in two phases, it is not surprising that specific heat experiences two peaks. The results from different studies agree well on the first peak (corresponding to the first reaction) to occur at about 100°C. However, there is inconsistency on the temperature at which the second reaction takes place as well as the value of the peaks.

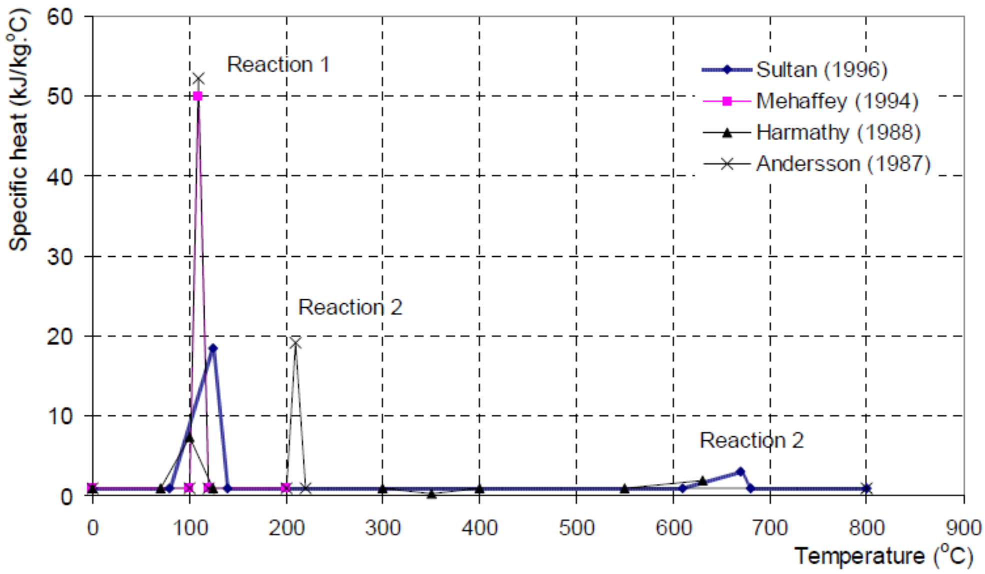

Figure 5. *Specific heat of gypsum plasterboards according to various researchers (Thomas, 2002)*

Harmathy (1998) reports a peak of 7.32 kJ/kg.°C at 100°C and although he does not give measurements over 630°C, his results shows a peak of 2 kJ/kg.°C at this temperature which indicates the second reaction. Andersson and Jansson (1987) provide peak values of 52.2 kJ/kg.°C and 19.2 kJ/kg.°C at 110°C and 210°C, respectively. Mehaffey *et al* (1994) first conditioned the specimens at 40°C for 24 hours in an attempt to drive off free moisture and then used a differential scanning calorimeter at two scanning rates of 2°C/min and 20°C/min. The results showed a peak of 29 kJ/kg.°C at 95°C when the slower scanning rate was used, and a peak of 14 kJ/kg.°C at 140°C when the faster scanning rate was employed, while the area under both peaks was about 500 kJ/kg, corresponding closely to the values in section 2.5.1 (100+356=456 kJ per kg of gypsum). Mehaffy *et al* measured specific heat up to 200°C, thus no second peak was observed. Sultan reports the first peak of 18.5 kJ/kg.°C occurring at 125°C and the second peak of 3.07 kJ/kg.°C at 670°C. The specific heat at ambient temperature is the base value when no reaction occurs. This base value is reported to be 0.88 kJ/kg.°C by Harmathy, 0.95 kJ/kg.°C by Mehaffey *et al* and taken as 0.7 kJ/kg.°C by Andersson and Jansson. Since specific heat of gypsum shows sharp peaks, in some finite element modelling, the use of enthalpy is preferred to separate values for specific heat and density to avoid numerical instability (Thomas, 2002).

4.2 DENSITY

Mehaffey *et al* (1994) used thermogravimetric analysis (TGA) at a scanning rate of was 20°C/min to determine the changes in mass of 10-30 mg specimens of gypsum (Type X) with temperature. The result is demonstrated in Figure 6. As can be seen, between 100°C to 160°C about 17.5% of the initial mass is lost, which indicates the first dehydration reaction and the release of water of crystallization (0.75x21%=15.75%) as well as the evaporation of the free water (less than 3%). They also noticed a mass loss at 650°C which corresponds to the second dehydration reaction.

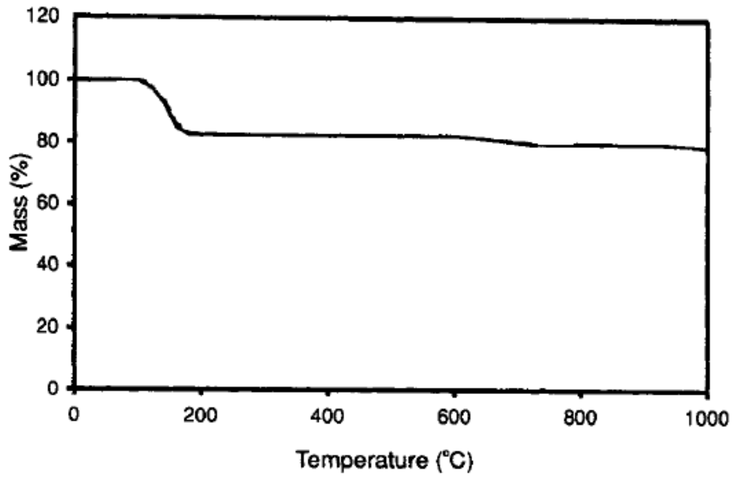

Figure 6. *Mass loss in gypsum with rising temperatures (Mehaffey et al, 1994)*

Mehaffey *et al* (1994) report the initial density of the 15.9mm Type X gypsum plasterboard as 648 kg/m^3. However, the density of the gypsum core of different gypsum plasterboards at ambient condition varies from type to type and also from different manufacturers. A study by Thomas *at al* (2005) on a large number of Type X gypsum plasterboard samples shows that the density of 12.5mm and 15.8mm boards varies largely both within and between manufacturers. The mean value of the densities ranges from 687 kg/m^3 to 811 kg/m^3.

4.3 THERMAL CONDUCTIVITY

The determination of the thermal conductivity of gypsum is quite complicated due to the effects of moisture and radiation in the pores. Gypsum from different sources or manufacturers varies in microstructure, and methods employed to measure its thermal conductivity also differ. As a result the values reported by different studied vary widely (especially at temperatures above 500°C); nevertheless follow a similar trend. Figure 7 shows thermal conductivity of gypsum versus temperature given by a few studies. The symbols represent measured values and the lines represent modified curves used in models to provide good calibration between numerical and experimental results.

Andersson and Jansson (1987) used the Transient Hot Strip (THS) method which measures the resistance of a metal strip embedded in the material and derives the thermal conductivity of the material. Their results for thermal conductivity of gypsum is quite isolated compared to others. Harmathy (1988) used a variable state scanning technique with relatively small temperature gradients. His results are very much in agreement with Mehaffey *at al*'s who used the TC-31 thermal conductivity meter (Mehaffey *et al,* 1994).

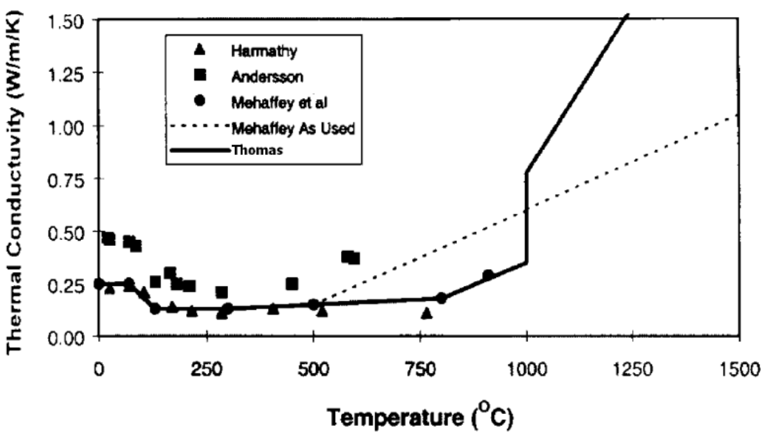

Figure 7. *Thermal conductivity of gypsum versus temperature (Thomas, 2002)*

Thomas (2002) mentions that the significant increase in thermal conductivity of gypsum at temperatures above 800°C is to allow for the opening of cracks and ablation of gypsum, since the testing method used by Mehaffey *at al* prevents the cracks from opening up in the board. It can also cover accelerated radiations in the voids at high temperatures: As mentioned before, heat transfers through the pores of gypsum is by convection and radiation. At high temperatures water is migrated from pores and the radiation through pores becomes significant (since it is proportional to temperature to the power three), which highly improves the heat transfer.

4.4 EXPANSION (SHRINKAGE)

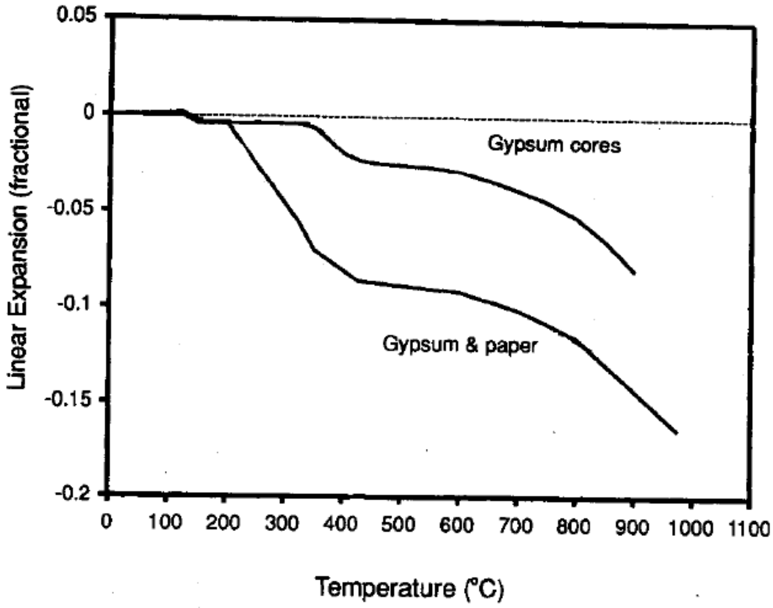

Figure 8. *Contraction of gypsum as a function of temperature (Mehaffey et al, 1994)*

Figure 8 shows the shrinkage of gypsum plasterboard and gypsum core against temperature according to NRCC (National Research Council of Canada). When subjected to high temperatures, gypsum core experiences significant shrinkage. In addition, at temperature range of about 200-350°C the paper laminates on the sides of gypsum core burn off. Thus the thickness of the board (gypsum and paper) is also reduced. This significant reduction needs to be considered when modelling the structure, since it will eventually cause the formation of cracks as well as the opening of the joints. Both these effects might hugely influence the heat transfer through the system.

Vermiculite, a natural mineral, is commonly used as an additive to the gypsum core to mitigate the effect of shrinkage. Vermiculite expands with the application of heat, partly offsets the contraction of gypsum and therefore enhances the performance of the system in fire. Glass fibre is also a reinforcing agent which bridges shrinkage cracks and attempts to sustain the integrity of gypsum plasterboard during calcinations (Gerlich, 1995).

4.5 ABLATION

Given sufficient time under heat or exposure to fire, some materials undergo physical and chemical changes, which results in bonding reduction of the material and removal of successive thin layers from its surface. This process is referred to as ablation. With rising temperatures the exposed surface of gypsum loses water and turns into calcium sulphate anhydrite, which falls off the unaltered substrate. As heat penetrates through the thickness, more material transforms to anhydrite powder and consecutive layers are shed. Using glass fibre reinforcements delays the ablation of the exposed surface. Thomas (2002) observed ablation at about 700°C for normal gypsum plasterboard and 1000°C for fibre reinforced board. It is also worth noting that ablation is of greater importance for thin boards compared to thick ones, as a larger proportion of the material is shed off. To include ablation in numerical analysis, one can simulate the reduction in thickness of the material, however, it is more convenient to modify thermal conductivity value (increase the value at high temperatures) to allow for the effects of ablation. Nevertheless, since ablation occurs at high temperatures, its effect would be small.

5. CONCLUSIONS

This chapter has provided a review of relevant literature on fire resistance of gypsum systems, focusing on the determination of gypsum thermal properties. There exist quite a large number of studies on this subject, each considering some aspects of the problem, which verifies the breadth of the matter and ongoing demand for more accurate and efficient approaches.

It is clear that there are large discrepancies in results of thermal properties of gypsum from different investigators and there is a need to develop a method to help manufacturers to extract relevant specific thermal properties of their specific gypsum products. These thermal properties can then be implemented in numerical models to generate results to evaluate the effectiveness of different new products before committing great resources to expensive full scale fire testing, which is the current practice. Since these properties are mainly to be used in numerical modelling, they do not need to represent the precise actual values. Provided the numerical analysis procedure is correct, the thermal property values used in numerical modelling should be those which give calculation results in agreement with experimental results of temperature developments in gypsum. This will often require an iterative process, therefore, the closer the thermal properties of gypsum to their actual values are used in numerical analysis, the better the agreement between numerical and experimental results.

ACKNOWLEDGMENTS

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INVESTIGATING THE STRENGTH OF SELF COMPACTING CONCRETE INCORPORATING LOW AND MODERATE VOLUME CEMENT REPLACEMENT USING RAW RICE HUSK ASH

Md Nor Atan[1] and Hanizam Awang[2]

[1,2]Universiti Sains Malaysia, Pulau Pinang, Malaysia

[1]mdnor_atan@yahoo.com , [2]hanizam@usm.my

ABSTRACT: This paper investigates the use of ‘raw’ rice husk ash in binary and ternary self-compacting concrete mixtures. Raw ash is preferred in this study, as against treated ash used in most other studies, for its economic advantage as well as for ensuring the robustness of self-compacting concrete mixtures. It is found that replacing 15% by mass of ordinary Portland cement with raw rice husk ash produces comparable strengths with normal concrete. It is also found that replacing 30% by mass of ordinary Portland cement with blends of raw rice husk ash-limestone powder and raw rice husk ash-pulverized fuel ash produce almost similar strengths as the 15% replacement with raw rice husk ash. It is concluded that rice husk ash may be employed in its raw form to replace ordinary Portland cement in low and medium volume cement replacement.

Keywords: self-compacting concrete, ternary blend, mineral additives

1. INTRODUCTION

Concrete is the most widely used material in the construction industry because it is mouldable and versatility; it bears excellent fire and weather resistant property, it is good in compression and although weak in tension but can be strengthened with reinforcements and above all, it is relatively cheap. But, when faced with issues of productivity, economy, quality and environment, concrete must meet new and higher demands. One direction in this evolution is towards self-compacting concrete (SCC), a modified product that without any additional compaction energy, flows and consolidate under the influence of its own weight. SCC offers many advantages for the precast and pre-stressed concrete industry as well as cast in-situ construction such as; low noise-level in the plants and construction sites, elimination of problems associated with vibration, less labour involved, faster construction duration, improved quality and durability and higher strength (Ouchi et al., 2003).

However, fresh SCC is a sensitive mix strongly dependent on the composition and characteristics of its constituents. It has to possess the incompatible properties between high flow-ability and high segregation resistance. This balance is made possible by the dispersing effect of high-range water-reducing admixture combined with the cohesiveness produced by a high concentration of fine particles (Esping, 2007). The latter is achievable by incorporating inert mineral filler and/or supplementary cementitious materials (SCM), collectively termed as mineral additives.

Incorporating mineral additive in SCC production allows the cement content to be regulated, which helps protect and preserve the environment from greenhouse gases associated with cement production process. In terms of fresh SCC rheology; regulating the cement content reduces heat of hydration and thermal shrinkage (The European Project Group, 2005). Furthermore, some additives are found in by-products of other processes and their use in concrete production keeps these materials from land fill (Neuwald, 2010).

Fine limestone powder (LP) is most commonly used as mineral filler material due to its ability in enhancing many aspects of cement-based system through its physical or chemical characteristics. The physical characteristic is associated with the fineness of its individual particles which enhances packing density and reduces interstitial voids. The chemical aspect includes supplying ions into the phase solution which modifies the kinetics of hydration and the morphology of the hydration products (Turkel and Altuntas, 2009). An investigation has shown that partial replacement of cement by equal volume of LP varying from 5% to 20% resulted in enhanced fluidity and reduced yield stress (Yahia et al. 1999 cited in Turkel and Altuntas, 2009).

On the hand, SCM is a class of mineral based material which possess pozzolanic reactivity and/or latent hydraulic reactivity (Kurtis, 2002). Hydraulic cement reacts chemically with water to form hydrate compounds (C-S-H) that have cementing property and calcium hydroxide (lime) that does not contribute to cementing property. Pozzolanic reaction is a chemical reaction with calcium hydroxide (lime) and water that leads to the formation of cementitious product, C-S-H. Many of the beneficial effects of SCM are the effects they have on the pore structure through micro-filler effect (increased packing of cementitious particles), increased C-S-H (replacing porous lime with C-S-H) and wall effect (densifying the interfacial transition zone at the cement-aggregate interface and pore blocking (which occurs because of the combination of the former two factors).

There are two types of SCM; processed/manufactured (e.g. fly ash, silica fume and slag) and natural (produced from natural mineral deposits and e.g. volcanic ash, shales etc or requires heat treatment e.g. rice husk ash, metakaolin, calcined clay etc). Fly ash (FA), also called pulverized-fuel ash, is a by-product of coal burning power plants. FA has a slow rate of hydration thus provides very little cementing values at early age but at later ages cementing activity becomes apparent and considerable. Because of this FA is sometimes used to reduce heat of hydration and to lengthen setting time. The spherical particle shape produces ball bearing effect which increases fluidity and reduces water requirement (Kurtis 2002).

Rice husk ash (RHA) is by-product of rice husk used as bio-fuel for the parboiling process in rice milling plants. It is a highly reactive SCM classified as super pozzolan (silica content > 85%). The way in which RHA improves workability is as follows; if well distributed in the cement paste, RHA particles segment the bleed water channels and consequently prevent bleeding and segregation. The physical effect, follows the chemical effect involving the pozzolanic reaction, fills up empty spaces and causes densification (pore refinement) and strengthening of the microstructure, particularly in high porous and least cracking resistant interfacial zone in the vicinity of coarse aggregate particles (Sinhania, 2004). It was reported that replacement up to 20% of cement with RHA improves the quality of concrete at age above 60 days (Ahmadi et al., 2007).

However, the use of single additive component (binary mix) could also pose some issues with workability and strength development (Kurtis, 2002). In order to hinder the disadvantages of single additive component (especially in high volume cement replacement), one or more mineral additives can also be added forming ternary or quaternary mixtures (Sahmaran et al., 2006). The main purpose for using multiple additives, especially in high volume cement replacement (HVCR) is to enhance particles packing. This is because HVCR could result in strength reduction because even when SCM is used to replace cement, its pozzolanic reaction alone may not be sufficient to offset for the strength loss (Kurtis, 2002). For example, FA particles are not much smaller than cement particles; by adding particles smaller than those of cement and FA (such as silica fume or limestone powder), enhances the particle packing because the intermediate spaces between cement and FA particles could only be filled by smaller particles. Enhanced particle packing could provide significant effect on strength and durability performances of HVCR concrete.

The main objective of this study is to investigate the effects of replacing ordinary Portland cement (OPC) with rice husk ash (RHA) in binary and ternary mixtures. In binary mixtures, 15% by mass of OPC is replaced with RHA while in ternary mixtures, 30% by mass of OPC is replaced with RHA/LP and RHA/FA blends. In order to evaluate the effect of RHA addition (replacement), one control mix without mineral additive and two binary SCC mixes without RHA were also prepared.

The uniqueness of this study is in the use of ‘raw’ RHA instead of ‘ground’ RHA as used in most other studies. Raw RHA is obtained direct from milling plant and used in its original form without undergoing further treatment (grinding or sieving). The use of raw RHA is hoped to produce a robust SCC product i.e. a product which is not susceptible to minor variation in physical and chemical composition of its constituent materials.

Rice husk ash (raw or ground) is a renewable agro by-product. Its application in SCC production provides economical and environmental benefits in terms of cost effectiveness, waste management and reduction of atmospheric pollution. Hence the Author strongly suggests that stakeholders of cement and concrete industry should at least review with renewed interest, studies that have been done pertaining to the use of RHA in cement and concrete production processes.

2. METHODOLOGY

The basic materials for SCC are binder paste, aggregates and high-range water-reducing agent (superplasticizer). In this study, the binder paste comprises of ordinary Portland cement (OPC), mixing water, and three types of mineral additive i.e. inert mineral filler (LP), pozzolanic mineral byproduct (FA) and pozzolanalic natural byproduct (RHA). All materials were used as received from the respective local sources. The typical properties of OPC, LP, FA and RHA are as shown in Table 1. Crushed granite (G) was used as coarse aggregate with 4.75/12.75 grading and washed river sand (S) used as fine aggregate with 0/4.75 grading. Crushed granite content was fixed at 40% of the total aggregate content while the remainder 60% was washed river sand. Higher fine aggregate fraction as compared with coarse aggregate was employed so as to provide high volume of fine materials that would improve the stability SCC mixture. The chemical admixture used in this study was ADVA 181; a high range water-reducing polymer-based superplasticizer (SP) formulated in accordance with BS5075 Part 3:1985 specification.

Table 1.*The oxide compositions and the physical properties of ordinary Portland cement (OPC), fine limestone powder (LP), pulverized fuel ash (FA) and rice husk ash (RHA)*

	OPC	**LP**	**FA**	**RHA**
Oxide Composition (%)				
Silicone dioxide (SiO_2)	21.28	1.84	56.2	92.99
Aluminium oxide (Al_2O_3)	5.60	1.37	20.17	0.18
Ferric oxide (Fe_2O_3)	3.36	0.14	6.69	0.43
Calcium oxide (CaO)	64.64	52.98	4.24	1.03
Magnesium oxide (MgO)	2.06	0.42	1.92	0.35
Sulphur trioxide (SO_3)	2.1	0.08	0.49	0.10
Physical properties				
Specific gravity	3.15	2.65	2.2 – 2.4	2.13
Specific surface area (m^2/kg)	295	320	440	1620
Loss on ignition (%)	1.05	42.92	1.34	-
Particle size (µm)	5 - 45	10 - 1000	8.08 (mean)	25
Density (kg/m^3)	1506	1201	540 - 860	530

The principles for the selection and proportioning of SCC constituents were based on guidelines laid out by the European Project Group while mix design was based on the rational method (EPG, 2005 and Okamura et al., 2003). The mass of OPC for the control mix was fixed at 475 kg/m^3. Subsequently; 15% of OPC was replaced with LP, FA or RRHA in binary mixes and 30% of OPC was replaced with blends of two mineral additives from LP, FA and RHA. Water content and SP dosage were adjusted so as to achieve class SF1 slump-flow with acceptance criteria ≥ 520 mm and ≤ 700 (EPG, 2005). Once the paste volume was determined, the aggregate volume fraction was calculated. Control and SCC mixture compositions are as shown in Table 2.

***Table 2**. Mixture compositions for control mix, binary SCC mixes and ternary SCC mixes*

Mix No.	Mix Description	OPC	LP	FA	RHA	Sand	Granite
		Kg/m³					
CM	100C	475	-	-	-	1047	712
BM1	85C/15LP	403.75	71.25	-	-	1041	708
BM2	85C/15FA	403.75	-	71.25	-	1028	698
BM3	85C/15RHA	403.75	-	-	71.25	1027	698
TM1	70C/15LP/15FA	332.5	71.25	71.25	-	1022	695
TM2	70C/15LP/15RHA	332.5	71.25	-	71.25	1023	695
TM3	70C/15FA/15RHA	332.5	-	71.25	71.25	1007	686

* OPC = ordinary Portland cement; LP = fine limestone powder; FA = pulverized fuel ash; RHA = raw rice husk ash; Sand = washed river sand graded 0/4.75 mm; granite = crushed granite graded 4.75/12.0 mm

CM = Control mix; OPC content is 475 kg/m³

BM = SCC incorporating binary blends of OPC (85% mass/mass) with single mineral additive (15% mass/mass) comprising of LP, FA and RHA

TM = SCC mixes incorporating ternary blends of OPC (70% mass/mass) with two mineral additive components (15% mass/mass each) comprising of LP, FA and RHA

2.4 Experimental programmes

c. Testing fresh properties of the control mix and SCC mixes.

Immediately after mixing, slump flow tests were carried on each SCC mix in accordance with BS EN 12350-8:2009 (No slump-flow test was carried out on the control mix).

d. Testing the hardened properties of the control mix and SCC mixe

Fresh concrete mix was cast into six 100 mm cubic moulds and six 100 mm x 100 mm x 500 mm prismatic moulds and left to stand for 24 hours. Subsequently, the hardened specimens were demoulded and immersed in water for curing. Dry density, compressive strength, flexural strength and ultrasonic pulse velocity tests were performed after 7, 14, 28, 60 and 90 days of water curing.

3. RESULTS AND DISCUSSION

3.1 Fresh properties

Fresh property tests are presented in Table 3. Binary mix, BM3 (85C/15RHA) and ternary mixes TM1 (70C/15LP/15RHA) and TM3 (70C/15FA/15RHA) which incorporate 15% replacement of OPC with RHA and 30% replacement of OPC with LP/RHA and FA/RHA blends respectively, exhibit up to 38% higher water demand as compared with the control mix.

Table 3. *Fresh properties of the control mix and the binary & ternary SCC mixes*

Mix No.	Mix Description	Mixing Water (L/m³)	Superplasticizer (L/m³)	Slump-flow (mm)
CM	100C	185	10.5	*-
BM1	85C/15LP	175	3.75	610
BM2	85C/15FA	194	5.7	640
BM3	85C/15RHA	255	10.5	640
TM1	70C/15LP/15FA	185	5.6	580
TM2	70C/15LP/15RHA	255	11.3	640
TM3	70C/15FA/15RHA	245	11.0	600

* No slump-flow test was performed on the control mix

The cellular structure of RHA particles, as shown in Figure 1, is responsible for its high specific surface area even when the particles are not small in size (Mehta, P.Q., 1994, cited in Sampaio et al., n.d., p. 2). Since the ratio of surface area to volume increases exponentially with particles irregularity, this parameter has a predominant effect on fresh SCC. When particles provide large surface area, high quantity of water is absorbed on the particles' surfaces and less water will be available to lubricate and disperse the particles for flowability (Esping, 2007).

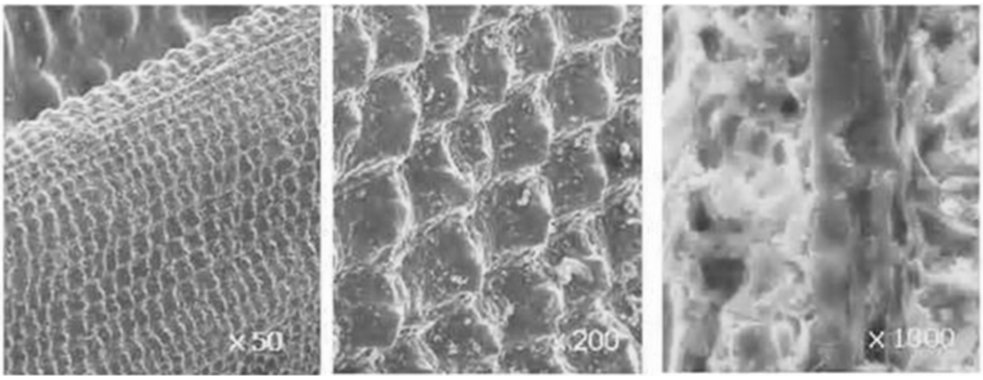

Figure 1. *Rice husk ash cellular structure responsible for high its surface area*

3.2 Compressive strength

Compressive strength tests were performed on hardened concrete specimens after 7, 14, 28, 60 and 90 days in accordance with BS EN 12390-3:2009. Compressive strength development for the control mix and SCC mixes are presented in Table 4. All SCC mixes which incorporate 15% replacement of OPC with RHA (BM3, TM2 and TM3) produce G40 class concrete, similar with the control mix. The best performance is shown by ternary mix TM3 (85C/15FA/15RHA) which is shown to produce a 90-day compressive of 43.4 MPa; 3% lower as compared with the control mix. Binary mix BM2 (85C/15FA) is the only SCC mix which does not incorporate RHA that obtains comparable compressive strength with mixes that incorporate RHA.

As mentioned in the literature, RHA is classified as super pozzolan and therefore it is highly reactive. High water content helps to disperse RHA particles hence maximizing RHA-lime pozzolanic reaction. As shown in Table 4, the effect of RHA-lime pozzolanic reaction on compressive strength development is noticeable after 14 days. The chemical reaction then generates a momentum of strength increase through physical densification and strengthening of microstructure effects which enhances compressive strength development up to the ultimate 90-day age (Sinhania, 2004).

Consider the following 90-day compressive strength tests:

Case 1;

Replacing 15% OPC with LP produces - 36.7 MPa

Replacing 15% OPC with RHA produces - 42.5 MPa

Replacing 30% OPC with LP/RHA produces - 42.4 MPa

It is shown that higher OPC replacement level is possible by incorporating LP/RHA blend, while still able to achieve equivalent compressive strength as compared lower level of replacement.

Case 2;

Replacing 15% OPC with FA produces - 39.4 MPa

Replacing 15% OPC with RHA produces - 42.5 MPa

Replacing 30% OPC with FA/RHA produces - 43.4 MPa

It is shown that higher compressive is obtained at higher OPC replacement level by incorporating two mineral additives components comprising LP and RHA as compared with lower replacement level incorporating either FA or RHA. As explained in the literature, the physical effect of LP particles and both physical and chemical effects of FA particles, improves the effects of RHA enabling higher level of OPC replacement while still maintaining comparable compressive strength as compared with concrete without OPC replacement.

Case 3;

Replacing 15% OPC with LP produces - 36.7 MPa

Replacing 15% OPC with FA produces - 39.4 MPa

Replacing 30% OPC with LP/FA produces - 28.5 MPa

Case 3 shows that higher OPC replacement level by incorporating two mineral additive components of LP/FA producer lower compressive strength as compared with lower replacement level using either LP or FA.

The above analyses suggest that raw RHA is the prime factor that generates enhanced compressive strength performance. The characteristic that differentiate RHA from LP and RHA is its high SiO^2 content making it highly reactive. The possible deduction which could be made is that when compressive strength is the concern in high volume cement replacement, chemical reactivity of the mineral additive is the prime factor that affects strength performance. This deduction is based on compressive strength tests that show all SCC mixes that involve replacement of OPC with raw RHA, either in single additive component system or in two additive component systems, are able to produce comparable results with concrete without OPC replacement.

Table 4. *Compressive strength development for the control mix, the binary and ternary SCC mixes*

Mix No.	Mix Description	Compressive strength (MPa)				
		7 days	14 days	28 days	60 days	90-day
CM	100C	36.5	37.6	37.8	45.4	44.7
BM1	85C/15LP	27.4	34.2	37.1	33.4	36.7
BM2	85C/15FA	33.3	37.0	37.9	38.9	39.4
BM3	85C/15RHA	22.7	29.6	39.8	41.9	42.5
TM1	70C/15LP/15FA	22.1	24.6	24.9	30.8	28.5
TM2	70C/15LP/15RHA	20.7	29.4	30.9	38.5	42.4
TM3	70C/15FA/15RHA	24.3	32.3	38.9	42.7	43.4

3.3 Flexural strength

Flexural strength tests were performed on 100 mm x 100 mm x 500 mm prismatic concrete specimens after 7, 14, 28, 60 and 90 days in accordance with BS EN 12390-5:2009. Flexural strength development for the control mix and SCC mixes are as shown in Table 5. Binary and ternary SCC mixes which incorporate RHA partially replacing OPC, exhibit improved flexural strength development as compared with the control mix.

Flexural strength tests performed on day 90 reveal that binary mix BM3 (85C/15RHA) produces 14% higher flexural strength as compared with the control mix. Similarly, ternary mixes TM2 (70C/15LP/15RHA) and TM3 (70C/15FA/15RHA) produce 9% and 2% higher flexural strength respectively, as compared with the control mix. These results indicate that RHA addition improves inter-particle cohesiveness which enhances SCC's flexural performance.

Table 5. *Flexural strength development for the control mix, the binary and ternary SCC mixes*

Mix No.	Mix Description	Flexural strength (MPa)				
		7-day	14-day	28-day	60-day	90-day
CM	100C	4.5	4.7	5.7	5.8	5.7
BM1	85C/15LP	4.0	4.1	4.7	4.5	4.9
BM2	85C/15FA	4.0	4.7	5.0	5.3	5.6
BM3	85C/15RHA	3.2	3.5	4.0	6.1	6.5
TM1	70C/15LP/15FA	2.9	3.4	3.4	4.2	3.9
TM2	70C/15LP/15RHA	3.4	3.8	4.1	5.3	6.2
TM3	70C/15FA/15RHA	2.7	3.8	3.7	6.0	5.8

3.4 Dry density and ultrasonic pulse velocity (UPV)

Dry density and ultrasonic pulse velocity tests are presented in Table 6. All SCC mixes are shown to possess lower density as compared with the control mix. This is because all additives used in the experiment possess lower bulk density than that of OPC. The bulk density of RHA is between 70 to 110 kg/m^3 whereas the bulk density of OPC is between 830 to 1050 kg/m^3, therefore when OPC is replaced with RHA it is bound to produce lighter concrete. In order to non-destructively assess the internal structure of concrete, UPV tests are performed on test specimens (Sahmaran et al., 2006). The grading for UPV values is as shown in Table 6. Since all SCC mixes which incorporate RHA addition obtain UPV values greater than 4.3 km/s, the internal structure may be classified as being close to excellent.

Table 6. *The average dry density and ultrasonic pulse velocity of the control mix, the binary and ternary SCC mixes and grading for ultrasonic pulse velocity*

Mix No.	Mix Description	Average Dry Density (kg/m³)	UPV (km/s)	UPV (km/s)	Grading
CM	100C	2287	4.53	> 4.5	Excellent
BM1	85C/15LP	2223	4.12	3.5 to 4.5	Good
BM2	85C/15FA	2290	4.40	3.0 to 3.5	Medium
BM3	85C/15RHA	2268	4.39	< 3.0	Doubtful
TM1	70C/15LP/15FA	2196	4.62		
TM2	70C/15LP/15RHA	2279	4.41		
TM3	70C/15FA/15RHA	2260	4.31		

4. CONCLUSIONS

Based on this study the following conclusions are made:

- replacement of OPC with RHA results in high requirement for mixing water in order to produce acceptable slump-flow,
- low volume cement replacement with RHA and moderate volume cement replacement with RHA/LP blend produce comparable compressive strength with that of normal concrete and improved flexural strength,
- moderate volume cement replacement with RHA/FA blend produces compressive and flexural strengths comparable with normal concrete and
- partially replacing OPC with RHA and with blends of LP/RHA and FA/RHA, produce better strength development than replacement with LP, FA or LP/FA blend.

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PROBLEMS OF PRIVATE RESIDENTIAL PROPERTY DEVELOPMENT IN MAKAMA JAHUN NEIGHBOURHOOD OF BAUCHI METROPOLIS, NIGERIA

Aliyu Ahmad Aliyu[1], Rozilah Kasim[2] and David Martin[3]

[1,2,3]Faculty of Technology Management, Business and Entrepreneurships, Universiti Tun Hussein Onn Malaysia, Johor, Malaysia

[1]aaaliyu1978@yahoo.com , [2]rozilah@uthm.edu.my , [3]martin@uthm.edu.my

ABSTRACT: This study was designed to investigate and examine the problems encountered by private residential property developers in developing their residential accommodation in Makama Jahun neighbourhood of Bauchi Metropolis.Two hundred respondents were selected through stratified random sampling. Data were presented using simple percentage distribution tables and complete theoretical analyses were used to analyze data that are not numerical in nature. The analysis revealed that high cost of building materials, non-use of local building materials, low income of the majority of the respondents and poor source finance amongst others were discovered to be the major constraints to residential property development in the study area. Others are problem of land acquisition and statutory regulation which were also among the obstacles faced by the respondents.In other to make a meaningful and adequate coverage the study is limited to Makama Jahun area of Bauchi metropolis.This paper represents the first work that analyse the problems of rivate residential property development in MakamaJjahun neighbourhood of Bauchi metropolis, Nigeria.To arrest this situation, the use of local building materials should be encouraged by the respondents. Financial institutions should scale down on their stringent lending requirements for a building loan. The chains of schedule in formal land acquisition should be reduced by the government and thereby quicken acquisition process. If these are followed, it will help to combat residential development problems identified in the study area and other similar blighter neighbourhoods.

Keywords: development, neighbourhood, Nigeria private residential property, problem

1. INTRODUCTION

One of the greatest problems in the world today is that of provision of shelter. Shelter must not be only adequate for the population but also habitable. This problem is more pronounced in the urban areas where population pressure is on the increase. The problem of looking for shelter leads to invasion of land for the purpose of building (Jinadu, 2008)

In Nigeria, the urban housing situation continues to deteriorate in the absence of an adequate arrangements to ensure that housing facilities expanded in line with the rapidly population growth. Despite the past efforts of the nations housing problem, it was evident that the combined effort of the public and private sectors over the past successive government plans had continued to fall far short of housing need (Agbola, 2007). Past governments had tended to leave this important sector almost entirely to private effort, concentrating itself on the provision of limited number of residential quarters for its deserving officers. Abdul (2008) noted that the major symptoms of urban housing problems include:-

(i) An absolute shortage of housing units.

(ii) The emergence and proliferation of slums and squatter settlement especially in large cities.

(iii) Rising house rents; and

(iv) A growing inability of citizens to buy or build their own houses.

Individual effort to build or own a residence becomes quits difficult by the majority of the people subject to number of constraints. As a matter of fact the quality of life in any given environment is greatly influenced by the nature and standard of built up structures particularly residential properties. Agbola (2009)A cursory look at past housing policies and programmes in Nigeria reveals that effective solutions to housing problems in general and low income housing in particular are yet to be found. The housing situation has deteriorated continuously due to rapid natural population growth, increasing rural-urban migration etc. It is now generally agreed that most Nigerian states are facing a housing "crisis" of major proportion and a bounteous number of statistics and studies are available supporting this view (Abiodun 2008).

It is evident in Nigeria that in most cases objectives designed for a particular project are hardly ever realised. This is especially so when a critical look is made of the various government housing policies. For instance, Liman (2007) assessed the effectiveness of public low income housing programme in metropolitan Kano, which showed that the overall contribution of the programme in terms of increasing the supply of new housing required in metropolitan Kano was 50% of the planned target of 77,852 housing units for the period between 1975 and 1980. In addition, the low income group for whom the programme was meant did not constitute a substantial proportion of the eventual beneficiaries.

2. RESEARCH METHODOLOGY

The entire layout (Makama Jahun area) was stratified into eighteen clusters by the Ministry of Land and Survey, Bauchi. The number of plots vary from one cluster to another, the largest being S: 13 with a total of 262 plots while cluster S: 14 has the least number with 13 plots. Each cluster represents a stratum that can be studied independently from others. Having stratified the layout, 10% of the plots i.e partially developed and undeveloped plots from each cluster were selected randomly using the table of random digits.(see table 1 below).

This is necessary to achieve a reasonable spread in the location of interviews and questionnaires to be able to obtain a cross-section of problems experienced in the development process in different parts of the study area. This is imperative because of local variations in problems.

Table 1. *Makama Jahun Layout*

S/NO	Layout No.	Total No. of plots	No. of Fully Developed Plots	No. of Partially Developed Plots	No. of Undeveloped Plots
1	S:1	116	15	65	36
2	S:2	80	7	40	33
3	S:3	83	9	44	30
4	S:4	104	12	61	31
5	S:5	144	14	82	48
6	S:6	114	11	74	29
7	S:7	283	21	185	77
8	S:8	15	2	8	5
9	S:9	90	9	47	34
10	S:10	55	5	28	22
11	S:11	198	18	118	62
12	S:12	132	15	67	50
13	S:13	120	12	62	46
14	S:14	159	16	87	56
15	S:15	238	28	153	57
16	S:16	155	18	77	60
17	S:17	28	3	15	10
18	S:18	157	17	87	53
Total		2271	232	1300	739

Source: Field Survey (2010)

Table 2. *Ten Percent Samples Drawn from Each Layout*

S/NO	Layout (Cluster) No.	Total No. of plots (partially & undeveloped plots)	Samples Drawn
1	S:1	101	10
2	S:2	73	7
3	S:3	74	7
4	S:4	92	9
5	S:5	130	13
6	S:6	103	10
7	S:7	262	26
8	S:8	13	1
9	S:9	81	8
10	S:10	50	5
11	S:11	180	18
12	S:12	117	11
13	S:13	108	10
14	S:14	143	14
15	S:15	210	21
16	S:16	137	13
17	S:17	25	3
18	S:18	140	14
Total		2039	200

Source: Field Survey (2010)

In all, a total of 200 plot allotees representing about 10% of the total number of plots (2033) on the area were selected and administered with the questionnaires. Two types of questionnaires were administered. The first one was administered to the property developers in the study area. The second one was administered to Federal Mortgage Bank of Nigeria Bauchi Branch. The content and scope of the questionnaires are broad. For the property developers, it touches on socio-economic characteristics of the respondents their source of finance, problems encountered during land acquisition etc. For the Mortgage Bank it touch on the procedures for the issuance of the loan, interest rate chargeable, class of people that the Bank gives loan for private residential property development and classes of people that benefited from the Bank loan scheme among others.

Interviews were held with the head of planning unit of Bauchi State Urban Development Board, practicing estate surveyors and valuers, land officers, private developers, builders etc on information about building regulations, planning standards, etc. To have sufficient ground of balancing these observations, legal practitioners were also contacted using the same unstructured interview pattern. The aim of this was to find out the frequency at which they handle land-related cases in the study area, especially as it relate residential land and how these cases are finally resolved.

3. RESULTS AND DISCUSSIONS

3.1 Duration of Plot Acquisition Process by the Respondents

Though there is no official guidelines as regard how long land acquisition for residential purposes should take; it is expected that a minimum delay is associated with land transaction, especially for residential needs. This is because acquisition of land is the basis upon which all land use rest. Table 3 below is an analysis of duration experienced by respondents before they could get their titles registered in case of the formal plot allotees.

Table 3. *Time taken to acquire registered Plots by the respondents*

Time Taken	Number of Respondents	% Frequency
1-3 Months	14	7
3-6 Months	48	24
6-12 Months	64	32
After one year	74	37
Total	200	100

Source: Field Survey (2010)

From table 3 above, it is evidenced that in the study area, the official plot allotees could not finally document their land acquisition formalities until after about one year (37%). This period is indeed frustrating to so many people wishing to acquire land for residential purposes. The duration that is considered by many respondents as reasonable is the period between one to three months and as it can be seen from the table, this is represented by a small percentage (14%). Greater respondents in the study area were not able to register their land between three to six months and six months to one year of application. Other findings revealed that those plot owners that were able to register their titles between 1-3 months had either used the influence of their position or wealth.

Observations have shown that because of the individual interest, sentiments and values attached to the land in the study area, acquisition for public and private purposes has always been difficult, protracted and frustrating affair which is worth mentioning here. It is because of this, in addition to the earlier reasons that land transaction in the study area become problematic and cumbersome. Furthermore it is because of these reasons that land transaction has to vary considerably from place to place. The attractive quality of a place where plot is situated could also influence the speed of registering such titles.

3.2 Difficulties Experienced by Respondents in the Study Area

Land acquisition process in the state start by filling the prescribed application form, processing the application and registering of the title as already observed in this write up. The process is not without its difficulties, and these are differently experienced by plot owners in the study area. These difficulties range from high fees to staff attitude.

Table 4. Difficulties Experienced by Respondents in the Study Area.

Difficulties	Number of Respondents	% Frequency
Deposit fee	19	9.5
Processing fee	11	5.5
Reproduction of building plan	53	26.5
Reproduction of deeds	7	3.5
Conversion process	29	14.5
Staff attitude	17	8.5
All of the above	64	32
Total	200	100

Source: Field Survey (2010)

Table 4 above showed that in Makama Jahun area, the entire respondents experienced problem at all levels; the greatest problem experienced by these respondents was production of building plans. Though, they had already acquired the land, they maintained that it was one of the stages that delayed the regulation of their plot. 5.5% of the respondents in the study area experienced the difficulty of processing fee; this is the fee that plot allotees have to settle in survey, town planning and land division before registering their titles. 8.5% however, suffered the problem that emanate from staff attitude. This is the attitude of staff in the three units of lands, survey and town planning.

This staff attitude manifest itself in the way and manners staffs deliberately delay acquisition of land in order to frustrate the efforts of applicants. This was discovered to be reasons why many respondents could not carry their land acquisition process to an end; in the contrary they often stopped at the level of issuing a grant right. In the same vein, 14.5% encountered difficulties of conversion process; this is the process of regularising lands that had earlier on, been given or acquired through the procedure of either ward head or local governments. 3.5% of the respondents in the study area faced problem of deed registration in land department.The nature of the building plan problem is in the number applicants are required to include in their application files. A minimum of eight building plans, according to many respondents are too much for residential purposes and consequently delay land acquisition process, more so that they are often rejected on the ground of non-conformity with the planning regulations.

3.4 Degree to which various procedures are exploited in Land Acquisition

Table 5. *Analysis of various Procedures for Land Acquisition*

S/N	Various Procedures of Land Acquisition	No. of Respondents	% Frequency
1	Through informal acquisition	63	31.5
2	Through local government	19	9.5
3	Through sale	51	25.5
4	Through wardhead	21	10.5
5	Through gift	7	3.5
6	Through inheritance	13	6.5
7	Through lease	8	4
8	Through sublease	12	6
9	Through pledge	6	3
	Total	200	100

Source: Field Survey (2010)

Above finding revealed that the most used procedure is through informal acquisition in the study area. This is followed by those who acquired their land through sale; that is, people acquired their land through purchase from hoarders. The least used procedure in land acquisition is that of gift. The procedure of sublease is fairly used too.Now that the procedure of informal acquisition is seen to be dominant among all the rest method, the degree to which this method is properly observed becomes problematic because of the high prices charged by the land speculators. Most of the respondents who acquired their land through sublease indicated that the official consent process has not been formalised. This is a negation of the section 21 and 22 of the 1978 land use decree.

3.5 General assessment of Land Acquisition Process by respondents

To assess a phenomenon of this kind, there is bound to be varying opinion since it is not possible to have the same acquisitioned experience. Parameters of simple, very simple, fair, difficult and very difficult is therefore adopted for easy reference point of assessment.

Table 6. *Assessment of Residential Land Acquisition Process by the Property Developers*

Assessment Formula	Number of Respondents	% Frequency
Very simple	2	1
Simple	3	1.5
Fair	21	10.5
Difficult	82	41
Very difficult	92	46
Total	200	100

Source: Field Survey (2010)

From the above table almost half of the respondents e assessed the general land acquisition process to be very difficult (92%). This is followed by those respondents who assessed the process as difficult (82%). Putting these together, there are many respondents in the study area whose experience in land acquisition is not favourable. On the whole, therefore, majority of respondents in the study area had assessed land acquisition process to be either difficult or very difficult. This is a great obstacle to residential property development in the study area.

Due to urbanisation, many people are moving from the rural to the urban areas where modern facilities are available; population pressure in cities and towns had made residential accommodation in particular a problem. Furthermore, the congested urban places are in need of expansion, but land where this expansion is to be made is scarce. Land has become of great marketable value and no longer the ordinary land known to African tradition as a gift of nature to mankind. This observation has probably acted as a catalyst to the promulgation of the 1978 Land Use Decree to have a unified system of land acquisition and most importantly to reduce the activities of land speculators. This, from policy view, should have taken care of all or at least some of the irregularities in urban land acquisition.The involvement of ward heads in land transaction in the study area have been established through this finding to be due to lack of adequate settling of the affected people through compensation, and even where compensation issues is adequately addressed, they are still found in land transactions in the study area.

3.6 Duration in Formal Land Acquisition

Table 7, *Time Taken in Formal Land Acquisition*

Period Taken	Number of Respondents	% Frequency
1-3 months	1	0.5
3-6 months	7	3.5
6-12 months	14	7
After one year	178	89
Total	200	100

Source: Field Survey (2010)

It can be seen here that most respondents generally acquired their land with difficulties in the formal method. This picture is more conspicuously revealed looking at the table critically. 89% of the respondents acquired their land after one year. Invariably, therefore, people will continue to prefer other method of land acquisition not mindings the high financial involvement but for its fast nature.

3.7 Reason for Acquiring Land Informally

The delay and uncertainties that is associated with government official procedures of land acquisition has been the general reason for people opting for other alternatives of land acquisition. Investigation had discovered that there are other specific reasons which range from being cheap, easy to costly depending on the source.

Table 8. *Reasons for Informal Land Acquisition*

Reasons	Number of respondents	% frequency
Easy, cheap and fast	11	5.5
Easy, costly but fast	40	20
Cheap and fast only	4	2
Easy and fast only	10	5
Easy and cheap only	9	4.5
Costly and fast only	126	63
Total	200	100

Source: Field Survey (2010)

The table showed that the respondents who gave reasons of costly and fast (i.e 63%) are the highest in the study area. Representation of other reasons is indeed small, and this as the findings showed is in respect of lands that are acquired through warhead, inheritance and alienation among others. In the same vein, those respondents who gave reasons of costly and fast are those plot owners who acquired them through either vendors or land speculators as revealed through this research.

This research had discovered that the amount involved in this informal land acquisition is several times higher than what is involved in the government official procedure. The only advantage which the formal acquisition procedure has over this is that of legality. This however is not a threat to the patronisers of informal process. It is in the light of the above observations that James (2001) was quick to conclude that accumulation of lands by speculators who are protected by the law indirectly hinders acquisition of lands by individuals and public agencies for execution of development plans.

3.8 Cost of Building Materials

Building materials normally are over half of the total cost of a building. The period since the mid 1990s has witnessed an increasing upsurge in the cost of construction materials. The property developers in the study area face high cost of building materials caused by the unprecedented inflation rates in the economy. Even as far back as 1987 when prices were relatively lower, unless one had a “five digit” savings, he could not think of embarking on a building project. This is purely because of the high cost of construction materials. For example, the rising cost of building materials based on market survey of some materials is on the table below.

***Table 9**. Prices of Some Building Materials from 2008-2009 in Muda Lawal Market Bauchi*

Materials/Item	**Brand**	**Quantity**	**Building Materials Prices as at January, 2008 (N)**	**Building Materials Price as at December, 2009 (N)**
CEMENT	Portland	Per 50kg bag	1,600	1,800
Sandcrete block	9 inch	Per 1	70	90
Sandcrete block	6 inch	Per 1	50	60
Gravel	Aggregate	Per tipper	1,200	1,500
Sharp	Sand	Per tipper	7,000	8,000
Laterite	Sand		6,000	6,000
REINFORCEMENT				
Rods				
16mm	Smooth	Per price	2000	2,500
12mm	Smooth	Per price	1,000	1,200
13mm	Smooth	Per price	1500	2000
DOORS				
Plush doors	Big/small	Per 1	2,600	3,000
Wood doors	Big	Per 1	9,000	12,000
Wood doors	Small	Per 1	7,000	10,000
Iron doors	Big	Per 1	20,000	22,000
Iron doors	Small	Per 1	12,000	13,000
TILES				
Ceramic tiles	Ceramic	72 pieces	600	800
	6 x 6 wall tiles			
	8x8per square		800	1,000
P.V.C. Tiles	Rubber tiles	Per carton	10,000	12,000

FITTINGS				
Wash hand basin	Abeokuta product England	Per 1	7,000	8,000
Wash hand basin	Aristone	Per 1	8,000	10,000
Bath tub	Aristone	Per 1	6,000	7,000
Water heater	Abeokuta	Per 1	16,500	18,000
Water closet	England	Per 1	7,000	8,000
Standard shower	With	Per 1	10,000	12,000
Shower tray	Mixer		4,000	5,000
	Deep type			
Shower tray	Fat type		6,000	7,000
			6,500	8,000
ROOFING SHEET	Swan Zinc	Per Bundle	8,500	10,000
	Star Zinc	“	9,000	10,500
	Hand Print	“	8,500	10,000
	Elephant	“	9,000	11,000
GLASS WORK				
Louvre Blades	4 blades	Per pair	400	500
	6 blades	Per pair	550	650
	8 blades	Per pair	750	900
	2 ft plain	Per pair	100	200
	2 ft tinted	Per pair	150	250
	3 ft plain		180	300
	3 ft tinted		200	350
PAINTINGS				
Emulsion paint	Saclux coloured	Per 4 lit tin	500	600
Gloss oil paint	Saclux coloured	Per 4 lit tin	2,000	2,300
WOODS				
Planks	Soft 2 x 3 x 12	Per 1	950	1,100
	Hard 2 x 3 x 12	Per 1	1,100	1,300

Source: Market Survey (2010)

The frequent increase in prices of building materials constitute a factor affecting housing development since developers cannot afford the high quality materials due to exorbitant rates. Therefore, the research work revealed that prices of building materials rise up continuously. The market survey as at 2009 on building materials shows the instability in prices of building materials.

It was also established through this study that masons charge ₦ 50 per single block laying and labourers were paid from ₦ 600 to ₦ 700 per day. The carpenters charge between ₦ 1,500- ₦ 1,700 for fixing windows and doors frames in addition to their actual prices of ₦ 1,200 and ₦ 1,300 and an averagely build cement toilet cost not less than ₦ 100,000. Electricity fittings, wall decoration (paints) and bugler proof fixtures will be necessary for a house to meet urban standard. This will not cost less than ₦ 300,000 for a 2 bedroom house. A conservative estimate by 2 building technology experts revealed that for a 2 bedroom core unit house to reach its occupancy stage, it will require the sum of ₦ 800,000 to ₦1,000,000,000.00.

3.9 State of Development and Rent in the study area

A larger proportion of the respondents (57.3%) are still constructing their residential accommodation 32.5% have not even started developing their plots. While 10.2% have completed their residential accommodation (See table 1). In the study area, monthly rent ranged between N 2,000 to 3,000 for a single room (normally very small- 6m^2). But in cases of self contained flats- a 3- modern bedroom flat cost between N 140,000 to N 200,000 per annum. These high rentals are reflective of the shortage of housing accommodation. In some cases a self contained room and parlor was being rented at N 230,000 per year while a 2-bedroom self- contained flat was N 250,000.

4. CONCLUSION AND RECOMMENDATIONS

This research work has attempted a description and explanation of the problems of private residential property development in Makama Jahun neighbourhood of Bauchi metropolis. Through the study carried out, it was realised that among the most important or prevailing problems affecting private residential property development in the study area include:- Problem of land acquisition, housing finance, building materials, statutory regulation among others.

Land virtually remains a factor negatively affecting housing development in the study area due to incessant failure of developers to get title to land. The long period of awaiting Right of Occupancy (R of O) and Certificate of Occupancy (C of O) is an outstanding obstacle to residential property development in the study area.The payment of numerous fees by developers at various stages of preparation of the legal document discourages developers to continue with the struggle for land acquisition. Eventually, some of the respondents resort to informal way of land acquisition which is financially cumbersome.

The activities of hoarders also discourage land development in the study area. The landowners that are not capable of developing them are not willing to sell their lands to the potential developers because they want to sell it at an exorbitant price. . The remaining available land for sale attracts high price. This excessive rate shorts up development costs. High cost of building materials has been identified as one of the major problems militating against adequate residential property development in the study area. Generally, the problems have included that of scarcity and high cost of imported building materials or those with foreign components.

In the study area, most developers do travel out to buy building materials in bulk to enable them realise some discounts as there is no building materials industry in Bauchi apart from asbestos roofing sheet industry. As earlier mentioned, this is due to the inflationary pressure in the economy which contributed excessively to the present rise in the cost of building materials, most especially when the building materials are imported.

Unless these problems are taken as challenges or are addressed promptly in the study area, the problem of residential property development in Makama area is likely to remain for sometime. Indeed, without the total commitment and involvement of both private and public sector, there will be no long lasting solution to the identified problems. Thus, while trying to provide a long lasting solution to the problems in the study area, it is advised and hope that those involved in the provision of residential accommodation would look at it as a challenge as well as race that is fast moving ahead of time in the study area. For the purpose of overcoming the problems of private residential property development in Makama Jahun area, the following recommendations and suggestions are made:-

1) There is a need to reduce the level of bureaucracy associated with land acquisition and thereby quicken acquisition process in the study area. It is necessary to reduce the chain of schedule in formal land acquisition. This will go a long way to facilitate efficient residential property development in the study area. This may not however be possible until when land officers are exposed to broad training on simple map interpretations in cadastral surveying. This can enable them to confirm space under application, instead of forwarding that to survey department. Survey department can however continue with its traditional function of charting the layout and monitoring of such charts too.

2). For effective and efficient land acquisition to be put in place in the study area, there is the need to revisit the issue of land fees charged currently and the basis of determination, especially in respect of residential need. The fixing of land fees by the state chief executive should rather be handled by a special committee that should be made up of professionals in land- related disciplines. This is necessary because most of the state chief executives who are mandated by the land use decree to fix land charges are not all professionals and therefore lack the professional skill with which to assess land values.

3). The use of local building materials should be encouraged by the government. The property developers should on their own use such building materials in the construction of their houses to demonstrate to the society that they are functional and durable. Also the government should grant subsidy to the industries manufacturing such local building materials until the society develops enough taste for such product to stimulate substantial demand.

4). Emphasis should be on functional design with economy of material uppermost by the property developers in the study area. Design type obviously has a direct effect on development cost, the realities of our time does not give any room for flamboyancy in design if we actually appreciate the meaning of residential property.

(5) Enforcing penalty for none development of allocated plots:- This measures will be taken in order to reduce the problem of land speculation which is one of the important factors explaining the delay in the development of residential plots in the study area. Government should enforce a strict compliance with the conditions stated in the allocation paper. Where allottee fails to effect development on his plot after five years, the title to such land should be revoked. This measure becomes necessary because the two years development, period is too short considering the present economic condition of the country, and the rate at which the plots are left undeveloped in different residential layouts within the metropolis.

(6). Delay in disposing urban land: - The Government should avoid unnecessary delay in disposing urban residential land to genuine developers. In order to implement this effectively, right measures have to be taken in allocating the residential plots to most deserving applicants not to those who use the avenue as opportunity for making fortune. In addition to this, government and other financial institutions, should introduce and give out soft loan to developers, to facilitate plot development, particularly, the low income these groups. To implement this successfully loan should be inform of some building materials, such as cements roofing sheet etc.

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FIRE DESIGN FOR HOLLOWCORE CONCRETE FLOORING

Md Azree Othuman Mydin

School of Housing, Building and Planning, Universiti Sains Malaysia, 11800, Penang

azree@usm.my

ABSTRACT: In Malaysia and many other countries, the use of precast hollowcore concrete flooring systems in multi-storey buildings are very common. This is due to low onsite labour cost and high quality control. Among different precast flooring systems, prestressed hollowcore concrete slabs are the most popular because of their lightweight nature and the economical utilization of concrete. Nevertheless the structural behaviour of such systems under fire exposure is not straightforward to be envisaged because of the multifaceted geometry, composite construction and a extensive range of possible support conditions. This paper will describe the characteristics of hollowcore slabs and the advantages and disadvantages of hollowcore flooring systems. In addition, assessment on the requirements and recommendations to the fire design of concrete slabs from different standards will also be presented. These requirements will form the framework of future research focuses on providing a new method for the fire design of structures with hollowcore concrete flooring systems.

Keywords: fire design, hollowcore slab, concrete flooring

1. INTRODUCTION

Multi-storey buildings with precast concrete flooring systems are becoming more popular and well accepted in Malaysia and in many other countries owing to low onsite labour cost and high quality control. Among diverse precast concrete flooring systems, prestressed hollowcore concrete slabs are among the popular because of their light weight and the economical use of concrete. Hollowcore concrete flooring systems are precast and pre-tensioned concrete slabs. Dimensions of every unit is 1.2m wide, the depths of available unit in Malaysia are 150mm, 200mm, 300mm or 400mm, and the span length can be up to 20m for 400mm deep units. The units span one-way, and the current practice in Malaysia and most of other countries requires a layer of cast in-situ topping concrete which increases the shear strength and connects the hollowcore concrete units together laterally. The topping concrete can provide the completed floor some two-way spanning capability.

There are two different processes that can be used to produce the voids within the hollowcore concrete units: extrusion or slip form. In this process, zero slump concrete is pierced by an extrusion machine which creates the voids (Matthews, 2004). Due to the nature of this process, it is not possible to include reinforcement in the hollowcore concrete units. The prestressing strands are placed in the casting bed and stressed before the concrete is cast. With pre-tensioned concrete, anchorages for the prestressing steel are not used and the prestressing force in the strands is transmitted to the concrete only by bond stresses (Fellinger, 2004).

It is very important to distinguish the fire resistance of the floor slabs in multi-storey building design. If the given fire resistance of the floor slabs is less than the worst predictable fire severity, the outcome can be disastrous once the fire happens. Whilst the trend in the world of fire engineering is changing towards performance-based design, the present method to establish the fire resistance of hollowcore concrete system is still using tabulated data from the concrete standards, which in many countries are based upon equivalent slab thickness and not actual experimental results. It is quick and simple for designers to use tabulated data for the fire design of hollowcore concrete slabs. Because tabulated data is not performance based, by using it the designers decide to pay no attention to the structural behaviour of hollowcore flooring systems under fire.

The structural behaviour of a hollowcore flooring system under fire is complex, and conducting fire experiments with large hollowcore floor slab sub-assemblies is almost impracticable. Accurate computer models for simulating the structural behaviour of hollowcore slabs under fires have been developed to perk up the understanding of this behaviour (Fellinger, 2004), though; such analyses are too time-consuming to be relevant in the day-to-day design process. At the other end of the spectrum, simplistic approaches using straightforward code rules are insufficient to capture the thermal expansion across the units, the effects of the support conditions, or two-way behaviour. Hence, it is essential to develop a straightforward yet satisfactorily precise means for designers to model the structural fire behaviour of hollowcore slabs. For that reason, this research will focus on the characteristics of the hollowcore slabs which include the advantages and disadvantages. Additionally, this particular research will also compares the requirements and recommendations to the fire design of concrete slabs from different countries and code of practice.

2. REQUIREMENTS AND RECOMMENDATIONS FOR FIRE DESIGN OF CONCRETE SLABS

2.1 REQUIREMENTS FOR FIRE DESIGN OF CONCRETE SLABS

All building codes requires that, in terms of the structural stability and safeguard against fire spread, the structural members exposed to fire shall allow sufficient time for people to evacuate securely and also allow fire service personnel to undertake rescue and fire fighting operations. This is achieved by providing structural members with a fire resistance rating in accordance with ISO834 (ISO834, 1999). Within the claimed duration of fire resistance under the ISO834 standard fire shown in Figure 1, the load capacity of the floor should be maintained, flames or hot gases should not penetrate the slabs, and the floor should prevent an average temperature rise of 140°C or a local maximum of 180°C on the unexposed face. These three requirements, in such order, are also known as the stability, integrity and insulation criteria for the structural members.

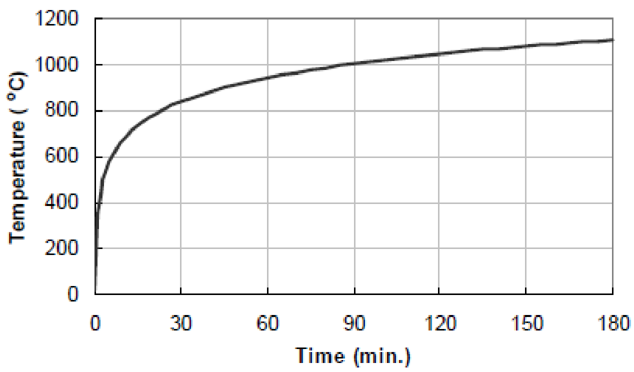

Figure 1. *ISO 834 standard temperature-time curve (ISO834, 1999)*

In Europe, the performance requirement of the structures in fire is defined in Eurocode 1 (EC1, 2002) which includes an adequate level of load bearing ability, as well as integrity and insulation of the structural members to allow the evacuation of the occupants and the rescue operation by the fire-fighters.

2.2 RECOMMENDATIONS FROM EUROCODES

Eurocode 2 (EC2, 2002) for the design of concrete structures re-states the idea of retaining stability, integrity and insulation of the structural members as stated in Eurocode 1, but with much more detail. Other than using engineering design, Eurocode 2 also provides information on concrete slabs to fulfil there criterions which are stability, integrity and insulation. Stability is the standard specifies a minimum concrete cover to the tendons to achieve a particular fire resistance rating. This is also affected by the rotational restraints at the supports. Hence the fire resistance of continuous slabs are considered separately, and the two-way supported slabs are considered separately from one-way supported slabs and have different fire resistance based on the aspect ratio. Integrity is the standard states that a slab has a fire resistance rating for integrity if it meets the requirements for both insulation and stability for that rating. Finally insulation is the criterion which depends on the effective thickness of the slabs and the type of aggregate of the concrete. The effective thickness of hollowcore slabs is the net cross-sectional area divided by the width of the cross section. In addition, Eurocode 2 also provides three alternative design methods for calculating the fire resistance:

Simplified calculation methods – Eurocode 2 suggests two simplified calculation methods for assessing the ultimate load-bearing capacity of heated concrete members: 500°C isotherm method and zone method. The first method disregards the part of the cross section with concrete heated above 500°C and full strength is used for the remaining cross-section. In the second method the cross section of the element is divided into zones, each with a different amount of strength.

Advanced calculation methods - Advanced calculation methods are based upon the fundamental physical behaviour aiming to provide a reliable approximation of the expected structural behaviour under fire. Both thermal and mechanical analysis should consider the temperature dependent properties of the materials. The result should be verified against relevant test results.

Tabulated data – This tabulated data gives standard fire resistance based on minimum slab thickness and cover to the reinforcement.

These design methods from Eurocode 2 are illustrated in Figure 2.

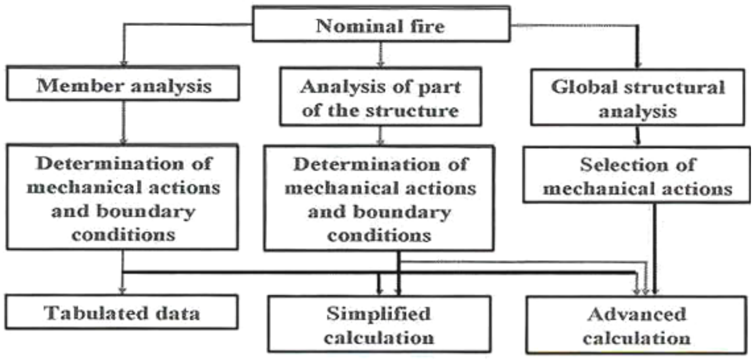

Figure 2. *Design methods for standard fire (Hietanen, 2005)*

2.3 RECOMMENDATIONS FROM ASTM E119

ASTM E119 (ASTM, 1999) specifies laboratory procedures and criteria for determining fire resistance ratings of different proprietary floor systems. Fire testing of concrete members or assemblies according to ASTM E119 is often decided on the following two criteria (Phan, 2005):

Heat transmission – This criterion limits the temperature rise of the unexposed surface. To meet this criterion, the walls, floors or roofs are required to have a sufficient thickness.

Load carrying ability – This criterion limits the thermally reduced yield strength of steel reinforcement be at least 50% of the yield strength at ambient temperature. This is achieved by providing sufficient thickness of concrete cover. ASTME119 also recognises the difference in the fire resistance of concrete floor slabs caused by the restraint conditions at the supports. For a precast flooring system to be qualified as restrained, ASTM119 states the must that the space between the ends of precast units and the vertical faces of the supports, or between the ends of solid or hollow-core slab units does not exceed 0.25 percent of the length for normal weight concrete members or 0.1 percent of the length for structural lightweight concrete members. This definition is schematically illustrated in Figure 3.

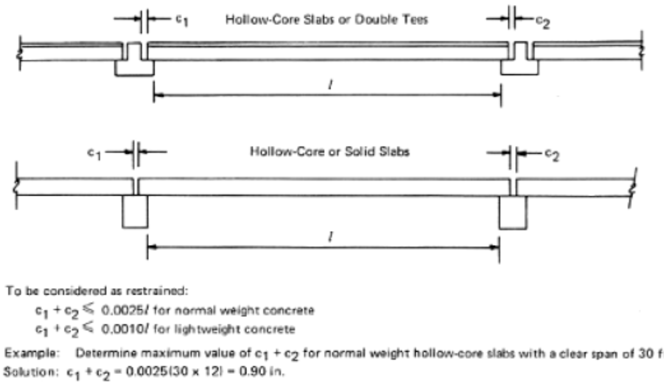

Figure 3. *Definition of restraint by ASTM E119 for precast systems*

2.4 RECOMMENDATIONS FROM PRESTRESSED CONCRETE INSTITUTE (PCI)

The 1989 design manual from PCI Fire Committee provided some calculation methods to evaluate the fire resistance of structures with precast and prestressed concrete but not specifically for hollowcore slabs (Gustaferro et al., 1989). The standard fire follows ASTM E119, in which the time-temperature curve is shown along with the ISO834 standard fire in Figure 4.

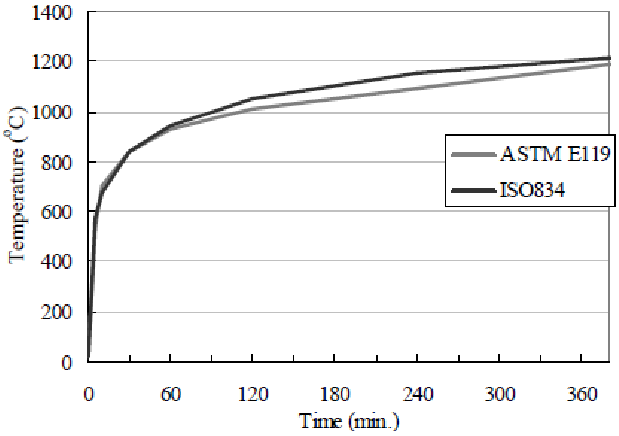

Figure 4. *Standard fire temperature curves*

The manual considers different restraint conditions on the slabs. In simply supported slabs the ends of the slab are free to rotate and move. The recommended calculation method focuses on the flexural failure mode; this is reflecting the fact that in all the fire tests none of the simply supported prestressed slabs designed according to ACI 318 has failed in shear. This flexure calculation method considers the loss of moment capacity during fire, resulting from the reduction of tensile strength in the prestressed strands and not by the loss of compressive strength in concrete. This is because the compression zone, which is near the top of the slab, remains at low temperatures during fires. The steel or concrete temperature with different cover thickness or depth after different time of exposure is obtained from experimental results and given as graphs in the manual. The continuous slab is taken as having an intermediate support in the simply supported slab, therefore both ends of the slab can still rotate and displace freely, but the bending moment diagram is different due to the presence of the intermediate support. The schematic drawing of a continuous slab, as well as the bending moment diagrams before and after the exposure of fire are shown in Figure 5. In the continuous slab the topping is used to provide continuity across the support, but it has been observed that the reinforcing bars in the topping around the support, which are the negative moment reinforcement, yield early during the fire tests and cause the redistribution of bending moments (Gustaferro, 1970). The behaviour of continuous beams and slabs in fire was tested on a series of 300mm by 350mm deep beams. At the support there are six no.6 (20mm diameter) reinforcement bars at the top and two at the bottom, at the midspan there are two bars at the top and four bars at the bottom.

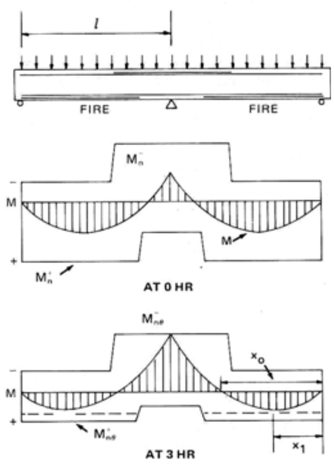

Figure 5. *Schematic drawing of a continuous slab; the bending moment diagram at cold condition and after 3 hours of the ASTM E119 standard fire*

During the fire test the bottom of the beams was hotter than the top, therefore it expanded more and created thermal bowing. The thermal bowing caused the ends of the beam to lift from their supports and consequently increased the reaction at the interior supports causing a redistribution of moments. In summary, thermal bowing increases the negative moment along the length of the beam.

When the negative moment reinforcement yields, the negative bending moment capacity is reached, hence the amount of moment redistribution depends upon the amount of negative moment reinforcement. It is important to limit the negative moment reinforcement so that compression failure in the negative moment region does not occur. Furthermore, the length of the negative moment reinforcement must be long enough to accommodate the complete redistributed moment and change in the inflection points. Therefore it is recommended that at least 20% of the maximum negative moment reinforcement is extended throughout the span.

This design method indicates that the positive bending moment capacity can be designed as for simply supported slab, and the negative bending moment capacity should be designed as the total bending moment minus the positive bending moment. The total moment is the maximum bending moment the beam or slab would encounter if it is simply supported, for uniformly distributed load that means $wL^2/8$, where w is the uniformly distributed load and L is the length of the span; the positive bending moment can be obtained from the figures in the appendix of PCI Design for Fire Resistance of Precast Prestressed Concrete (Gustaferro et al., 1989) or tables in Chapter 5 of PCI Design Handbook (Martin et al., 2004). Gustaferro et al. (1989) also provides the calculation method for the minimum length of the negative moment reinforcement. The manual also provides the calculation method for the minimum length of the negative moment reinforcement.

3. ENGINEERING DESIGN METHOD

All the standards mentioned above allow for using an engineering design method to calculate the required dimensions for structural members to meet the desired fire resistance. There are several computer programs available to analyse the behaviour of concrete structures under fire conditions, in both micro and macro scale. Due to the advancing of computational power, finite element programs such as Vulcan (Huang et al., 2006) and SAFIR (Franssen, 2002) are used not only for research or forensic analysis purposes but also for design by industry. Nevertheless, in terms of fire design of hollowcore concrete slabs, to the author's knowledge there is no ideal computer model for such purpose so far, hence one of the aims of this research is to create such a model.

4. PROPRIETARY FIRE RATING OF HOLLOWCORE UNITS

Proprietary fire rating is the fire resistance rating of proprietary products made by specific manufacturers. These ratings are obtained by either testing or by calculation depending on the manufacturers, and the engineer should consult the manufacturer to confirm the accuracy of the fire rating. In the US, Underwriters Laboratories has conducted more than 30 standard fire tests on hollowcore floor assemblies and published the results for more than 50 designs of hollowcore slabs which qualify for ratings of 1 to 4 hours. For hollowcore units not found in the Underwriters Laboratories ratings, the fire resistance rating can be obtained by conducting standard fire tests in accordance with ASTM E119 (ASTM, 1999) as mentioned before or by using the effective thickness method described in the PCI Manual (Gustaferro et al., 1989). Engineers should give different levels of caution to the prescribed fire ratings depending on the methods used.

5. CONCLUSIONS

This paper has discussed some important characteristics of the hollowcore slabs and recommendations and requirements to the fire design of from different countries and code of practice. It is vital to distinguish the fire resistance of the floor slabs in multi-storey building design. If the given fire resistance of the floor slabs is less than the worst expected fire sternness, the result can be devastating once the fire occurs. It is expected that the obligations and requirements discussed in this paper will shape the framework of potential and future research focuses on providing a new means for the fire design of structures with hollowcore concrete flooring systems.

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INNOVATIVE SEISMIC BASE ISOLATION TECHNIQUE

Shivam R. Mishra[1] and Sameer S. Dhuri[2]

[1,2]Civil Engineering, VJTI, Mumbai, India

[1]shivam18@gmail.com

ABSTRACT: Design of earthquake-resistant low to medium-rise structures is quite a challenging task. It is necessary and important to dissipate the energy which is generated from an earthquake. Base isolation technique is one of the advanced methods. By deploying this technique, the earthquake energy can be dissipated and the structure can be made earthquake resistant. Conventional energy dissipation techniques involve the use of Rubber Bearings. The disadvantages of currently available isolators are high cost of devices and their installation, short life span of bearings (high maintenance) and last but not the least, in case of Lead Rubber Bearings the toxic nature of lead makes it difficult to dispose-off, posing severe environmental issues and health hazards. In the present work, an experimental study has been carried out by conducting series of tests on multi-storeyed scaled building model by incorporating a viscous fluid and springs. This system basically comprises of two components viz. a highly viscous incompressible fluid present below the super-structure and a spring as stiffness element installed at the sides of the superstructure. This innovative design is an alternative to isolation system with Rubber Bearings. The building model is tested under seismic excitations generated by shake-table. Significant reductions in displacement and acceleration response quantities are found for the building with proposed isolation technique as compared to the non-isolated building. This isolation system has shown notable reduction in base displacement and inter-storey drifts as compared to non-isolated cases. The proposed technique provides good protection to both, the structure and its non-structural components over a wide range of earthquake ground motions.

Keywords: earthquake, energy dissipation, base isolation, viscous fluid, spring

1. INTRODUCTION

Seismic isolation has emerged as one of the most promising techniques for retrofitting strategies to improve the seismic performance of existing structures. It is also a practical solution for new construction when conventional design is not suitable or economical. In the seismic isolation approach, the superstructure mass is decoupled from seismic ground motions. It uses special types of bearings known as seismic isolation bearings which are placed below the superstructure and on top of the substructure piers and/or abutments. Under normal conditions, these bearings behave like conventional bearings. However, in the event of a strong earthquake, they add flexibility to the structure by elongating its period and dissipating input energy. This permits the superstructure to oscillate at a lower frequency than the piers resulting in large relative displacements across the isolator interface.

However, these large displacements can be controlled by incorporating damping elements in the bearing or by adding supplemental dampers. Seismic isolation provides two significant design features for a structure, viz. (1) it can reduce the seismic forces substantially, and (2) it can control the distribution of these reduced lateral forces among the substructures and foundations to further enhance the overall economy and effectiveness of new and retrofit designs. Seismic isolation is a proven design idea. This technique proposes to decouple the structure or part of it, or even the equipment placed in the structure from destructive effects of ground-motions. One of the aims of seismic isolation is to shift the fundamental frequency of the structure away from the dominant frequencies of earthquake induced ground-motion and the fundamental frequency of the fixed base superstructure. The other purpose of an isolation system is to provide an additional mean of energy dissipation, thereby reducing the transmitted acceleration into the superstructure. This innovative design approach is aimed mainly at the isolation of a structure from the supporting ground. This is commonly deployed in the horizontal direction so as to reduce the transmission of shear forces to the structure.

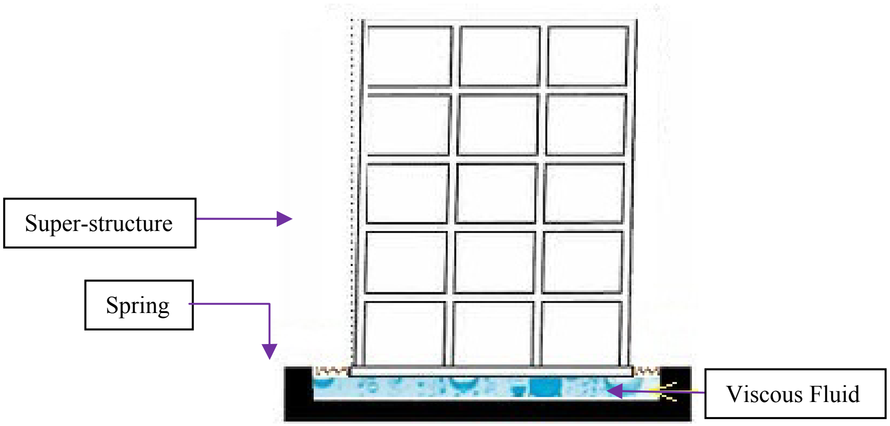

Figure 1. *Schematic Diagram*

1.1 Objective of current study

The protection of civil structures, including material content and human occupants is a worldwide priority. The extent of protection may range from reliable operation and occupant comfort to human and structural survivability. Civil structures, including the existing and future buildings, towers and bridges must be adequately protected from a variety of events. This includes earthquakes, wind, traffic and other dynamic loads. Use of passive, active and semi-active structural control devices to mitigate undesired responses to dynamic loads is the worldwide subject of research. However, even in controlled structures, it can be expected that large seismic events, such as the Northridge (1994) and the Kobe (1995) earthquakes, will cause structural members to exceed the elastic limit. Recent devastating earthquakes around the world have confirmed the importance of control devices in protection of civil structures. Structural control devices have been developed mainly in civil engineering structures to dissipate energy from earthquake and reduce vibrations in structures. This also reduces human and material losses. This technique helps in reducing the effective shear forces acting on the superstructure and hence minimising the chances of failure of a building during an earthquake. The main idea behind developing or proposing an isolation system is to propose a system which increases the time period of oscillation, reduces the frequency and shear force coefficient. The motivation behind developing this system is to make a highly economical and cost effective system. The key objective is to develop an intelligent damping system that not only provides damping in lateral but also in vertical direction in order to maintain stability. The primary idea of this technique is to reduce the collapse potential of a building and enhance its structural safety. This greatly improves the serviceability of the structures, resulting in higher returns on investments in case of mega-structures. Thus the objective of the new proposed system is to isolate the superstructure from the intense ground vibrations during an earthquake. This is basically done by dissipating the high energy seismic waves and reducing the intensity of vibrations for the structure during an earthquake. The structure is thus ensured of having lesser or negligible damage and higher bearing capacity.

1.2 Proposed innovative technique

Existing systems make use of bearings for isolations which have few disadvantages. The proposed system attempts to overcome these short-falls. The superstructure is fundamentally isolated by means of a thin layer of viscous fluid and springs. The springs are deployed at the sides, of which one end is fastened to the moat and the other end to the super structure. There exists a thin layer between the base slab of the isolated structure and the ground.

1.3 Concept and working of system

The seismic surface waves travel with high energies and are known be the most destructive waves. These high energy waves are responsible for most of the structural and non-structural damage. In order to minimize the destructive forces induced at the advent of the seismic waves, it is of paramount importance to dissipate this energy. This energy is dissipated by providing adequate isolation to the structure. The structure to be isolated is provided with a base slab. The base slabs are provided with calculated air columns/cavities as buoyant units. A layer of incompressible viscous fluid is provided between this slab and the raft of its foundation. The base slab is appended with number of heavy duty springs. The other end of the springs is secured on the retaining walls of the moat. At the advent of seismic waves, most of the energy of the surface waves are absorbed and dissipated to large extent with the help of combined damping action of the springs and the viscous fluid. This increases horizontal as well as vertical flexibility of the isolated structure. This considerably reduces the amount of shear forces transmitted to the superstructure. Thus, it helps in increasing the time period of structural vibration and reduces the drift acceleration with respect to the ground.

1.4 Advantage of proposed system

The Maintenance of this system is minimal as compared to the existing primitive systems. The existing systems do not isolate the structure from vertical vibrations. The proposed technique is capable of providing both lateral and vertical damping. The technique can allow economic design of structural members as the lateral forces for which the members have to be designed are relatively less. Reduced overall capital cost - the structure can be designed more economically. For example, in case of the St Francis Towers project, there was a net saving of at least US$ 4mn. For most of the tall building projects over 200m high that had been reviewed by structural engineers and experts, there had been a net cost saving when a isolation outrigger system or similar was used. Due to reduction in design forces, the columns can be designed in economical sizes. This results in a considerable increase in net floor areas. This typically leads to 0.5-2% more net floor area. This may sound small, but its value can often exceed the capital cost savings made. Unlike many existing systems, the proposed technique is effective for minor quakes and wind pressures.

1.5 Applications

As mentioned, the proposed technique has numerous advantages over the existing systems. The technique being economical and maintenance free can be applied to all important structures as well as other residential and commercial structures in seismically active zones. The system can be easily applied to both - existing structures for retrofitting and new construction.

2. MATHEMATICAL MODEL

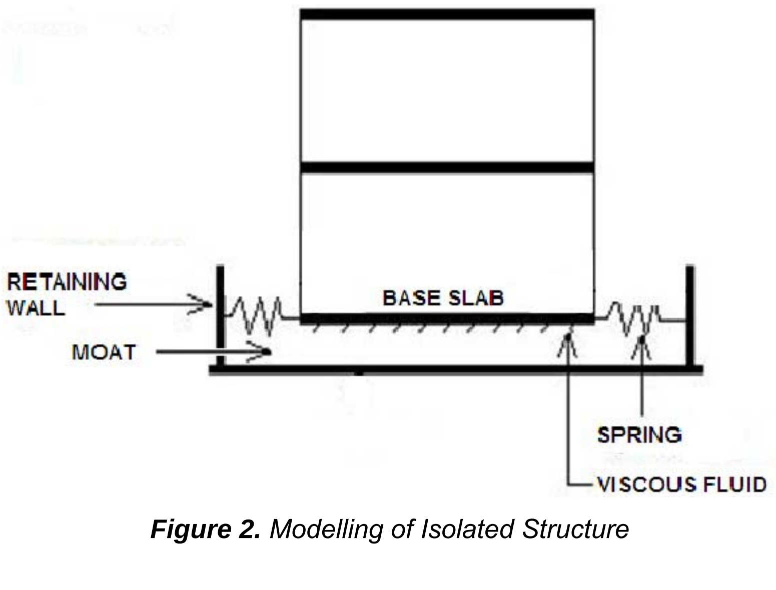

Figure 2. *Modelling of Isolated Structure*

Governing Equation for Isolation in Horizontal Plane:

$$M\ddot{x} + C\dot{x} + K_{eq}x = P(x)$$

Governing Equation for Isolation in Vertical Plane:

$$M\ddot{y} + C\dot{y} = P(y)$$

The equations can be written in matrix form as:

$$[M]\{\ddot{v}\} + [C]\{\dot{v}\} + [K]\{v\} = -[M]r\ddot{u}_g$$

3. SCALED PROTOTYPE DESCRIPTION

As mentioned, the proposed isolation technique mainly consists of two components viz. viscous fluid and the heavy duty helical springs at the sides. A 2 storeyed scaled model was prepared using mild steel plates and threaded rods. The slabs are represented by 3 mm thick 16x16 cm mild steel metal plates and the columns are represented by the 6 mm diameter threaded rods. Appropriate inter-storey height was chosen conforming to required structural stiffness and natural frequency

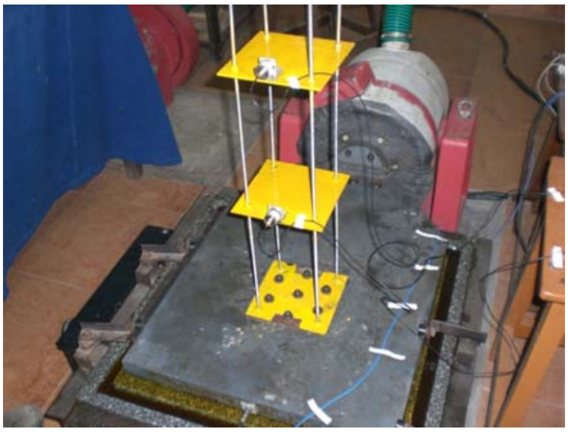

Figure 3. *Model fixed on Shake Table*

The substructure was also prepared using mild steel which was required to represent the moat. It was made leak-proof to avoid leakage of any viscous fluid which was to be used during experiments. A pair of spring was fixed with one of its ends on the moat wall and other end on the 2-storeyed structure. The superstructure was allowed to rest on a thin layer of viscous oil. The complete prototype assemblage was fastened on to the shake-table by means of bolts. The shake-table had a capacity of generating frequencies in the range of 5-50 Hz.

Figure 4. *Isolated Structure Model*

4. EXPERIMENTAL ANALYSIS

The 2-storey model was clamped and two accelerometers were attached in the middle and top storey. It was subjected to random excitation. The natural frequency of the structure was found out by free-vibration method. The response was analysed using FFT (Fast Fourier Transform). The two modal frequencies were found out to be 13 Hz and 38 Hz. Same model was then fixed on the shake table for testing under dynamic conditions. The model was subjected to a forced harmonic vibration of 0.1g. Accelerations of the middle and top slab were measured using accelerometers. The forced frequencies generated by the shake-table were increased from 5Hz to 45Hz and corresponding accelerations were noted for the slabs. The results were tabulated and a graph was plotted.

The scaled model was decoupled from shake table using the proposed isolation system. The assemblage was then subjected to same forced harmonic vibrations. Similar procedure was adopted for measuring and noting the behaviour of system for ascending frequencies of the shake table.

The natural frequency of isolation system was found out by free vibration response using FFT. This frequency was found to be 3 Hz.

5. EXPERIMENTAL RESULTS

The FFT analysis was carried out by placing accelerometers at the first and second storey. The readings for accelerations were compiled. 'Transmissibility Ratios' were calculated for corresponding 'Forcing Frequencies' for both fixed and isolated conditions. The frequency response for the two conditions was obtained graphically.

The experimental findings are depicted as below:

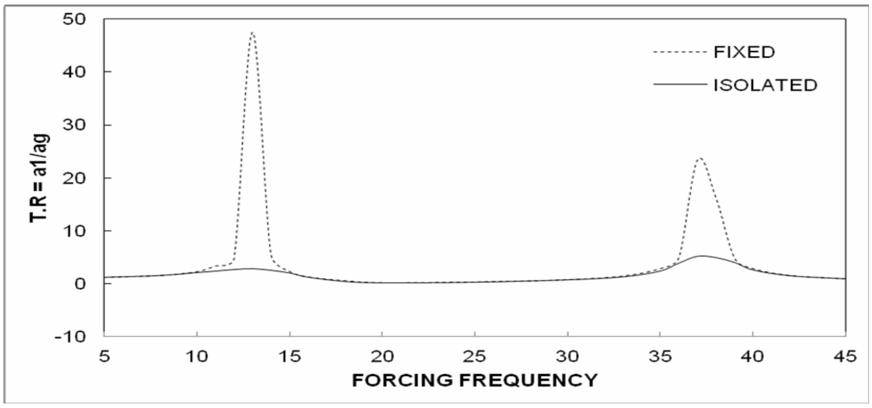

***Chart 1**. Frequency response for middle storey*

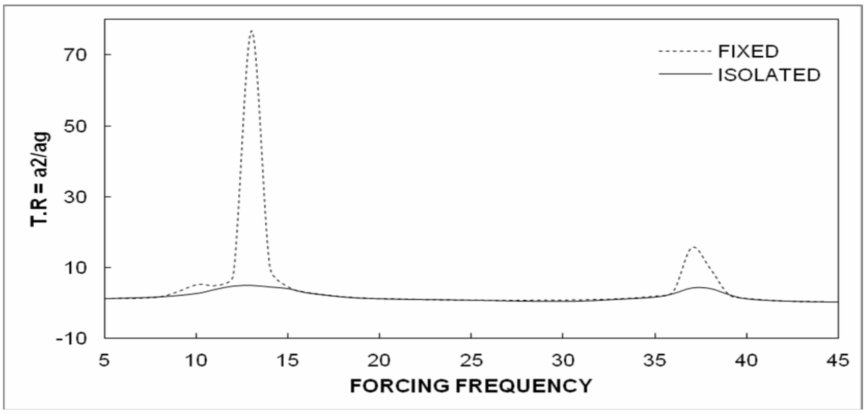

***Chart 2**. Frequency response for top storey*

The damping ratios (ξ) were calculated for both middle and top storey under fixed and isolated conditions. The damping ratios for fixed and isolated structures under different modes can be given as:

$$\xi = ((\omega_b - \omega_a))/(2\omega_n)$$

The damping ratios for middle storey in 1st mode were found as:

$$\xi(\text{fixed}) = 3.85\ \% \qquad \xi(\text{isolated}) = 20.6\ \%$$

The damping ratios for middle storey in 2nd mode were found as:

$\xi(\text{fixed}) = 2.63\ \%$ $\quad\quad$ $\xi(\text{isolated}) = 5\ \%$

The damping ratios for top storey in 1st mode were found as:

$\xi(\text{fixed}) = 3.08\ \%$ $\quad\quad$ $\xi(\text{isolated}) = 20.4\ \%$

The damping ratios for top storey in 2nd mode were found as:

$\xi(\text{fixed}) = 2.89\ \%$ $\quad\quad$ $\xi(\text{isolated}) = 4.92\ \%$

6. DISCUSSION

The effectiveness of the proposed isolation system is very much evident from the experimental results. The average damping ratio of (ξ) 20.5% in the first mode is more than that of LRB, N-Z systems and few other isolators.

There was an unanticipated downcast of damping ratio under 2nd mode i.e. 37 Hz. This could be explained by knowing the fact that; the springs used for experiments had a natural frequency of 37.1 Hz. (Spring Stiffness = 1418 N/m; Spring Mass = 26.1 gm.). During experimental tests, when the 'Forcing Frequency' equaled the natural frequency of the springs, (i.e. under resonance condition) the springs failed to dissipate the energy of vibration effectively.

This can easily be avoided by designing the springs accordingly. And hence, same damping ratio (20%) can be maintained under different dynamic and modal conditions.

Knowing that the technique is still in its primitive and crude stage, and the experiments were done under limited resources, the results are encouraging. The performance of the proposed technique can be ameliorated by further research and experiments with better resources.

7. CONCLUSIONS

Thus we conclude that the above mentioned system can lead to substantial saving of lives and cost incurred due to damage of structures when implemented on a large scale. The limitations of the existing systems being excessive operating and maintenance cost, which can be solved by application of the proposed system. The design of above system can be modified as per site conditions to meet the requirements. For its low cost, minimal maintenance, effective and eco-friendly behaviour, we conclude that the proposed system proves to be a better alternative.

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