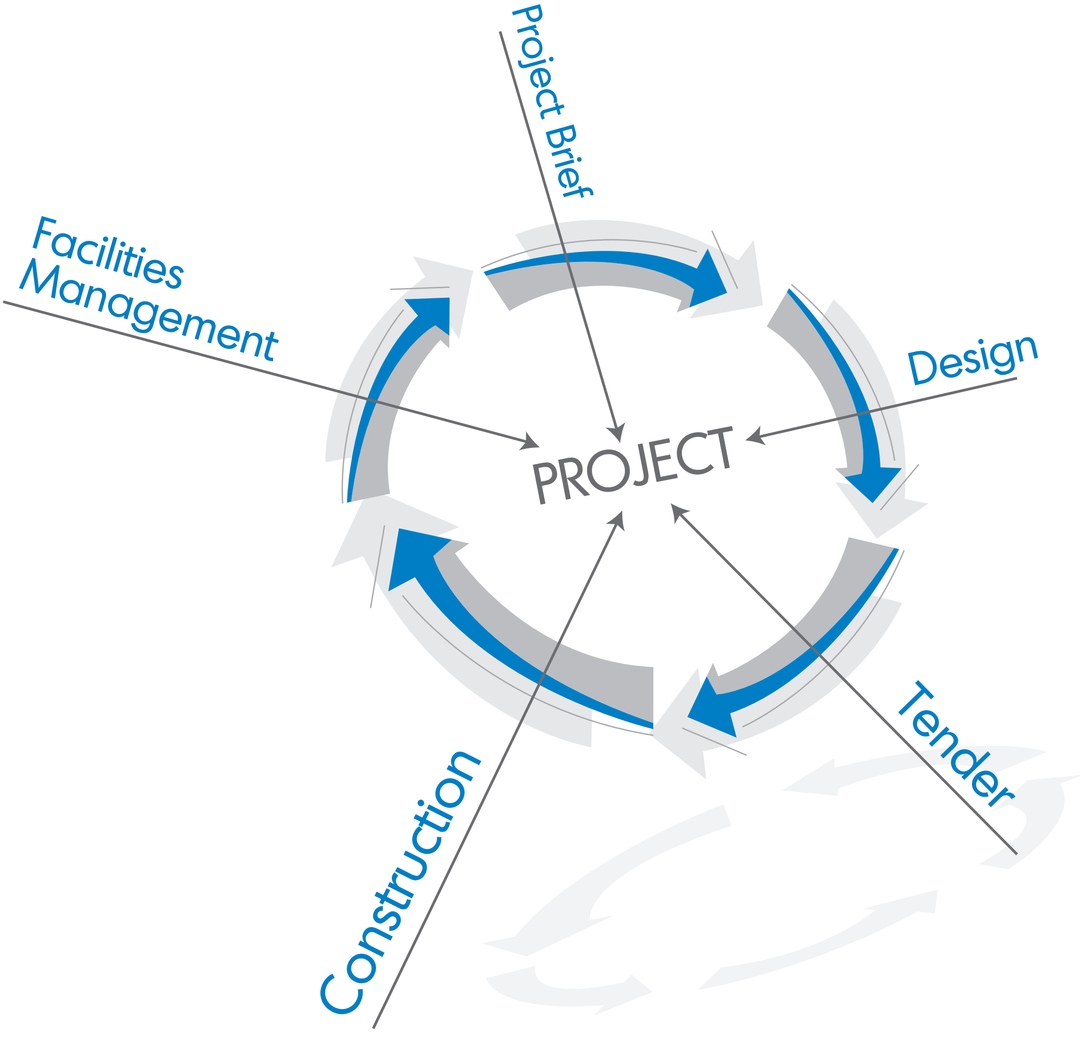
MCRJ Volume 30 | No.1 | 2020

ISSN No. : 1985 – 3807
eISSN No. : 2590 – 4140

Malaysian Construction Research Journal



CREAM
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MALAYSIA

MALAYSIAN CONSTRUCTION RESEARCH JOURNAL (MCRJ)

Volume 30 | No. 1 | 2020

The Malaysian Construction Research Journal is indexed in

Scopus Elsevier

ISSN No. : 1985 – 3807
eISSN No. : 2590 – 4140

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

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Contents

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Editorial

Welcome from the Editors

Welcome to the thirty (30th) issue of Malaysian Construction Research Journal (MCRJ). In this issue, we are pleased to include eight papers that cover a wide range of research areas in construction industry. The editorial team would like to express our sincere gratitude to all contributing authors and reviewers for their contributions, continuous support and comments.

In this issue:

Nor Khalisah Bidi et al., presenting the outcomes of quality cost for Life Cycle Cost (LCC) analysis of maintenance during the in-use phases of university building. Qualitative research strategy including literature review and semi-structured interview was used in this study. The outcome of the study shows the key quality of data inputs comprises of data availability, data accessibility and data currency. Few recommendations based on the key quality of data inputs has drawn such as accessibility of data for the good use of practitioner. The study has proposed there is a need to develop appropriate methodologies of data input requirements for LLC analysis of maintenance during the in-use phases of the university building.

Ponnada Markandeya Raju experimented the assessment of strength of epoxy bond between steel and roughened concrete interface. The study evaluates the effect of surface roughness and adhesion strength between steel and concrete surfaces in two different ways through push-out tests. This study also compares the performance of steel structure epoxy bond for different roughness pattern. The properties of material adopted in this study involved the properties of ingredients of concrete and properties of fresh and hardened concrete. The outcome shows that the interface roughness affects the interface behaviour significantly.

Rashidul Islam et al., assessed the design factors affecting facilities management practices of buildings in Malaysia. 38 design factors were identified through literature review under three categories of architectural, structural and Mechanical-Electrical-Plumbing (MEP) design. Extensive literature review and survey questionnaire were used as the methodology in this study. The results then analysed using Relative Importance Index (RII). The outcomes comprise 17 factors under three categories which are architectural design quality, architectural materials and architectural accessibility. Majority of the respondents agree that the design related factors affecting Facilities Management (FM) practice is inadequate working drawing details followed by incomplete working drawing and specification.

Herry Suryadi Djayaprabha et al., explored the effect of curing conditions using slag based cementitious binder as an activator using the experimental program. The study shows the optimum addition of calcined dolomite as an activator was found to be the amount of 20 wt % (SL80CD20) with the 7-, 28-, and 365-day compressive strengths of 18.6, 23.8, and 31.3 MPa for water curing, respectively. From the microstructural observation, the identical hydration products were found for both the slag dolomite binders which were cured either in air or water.

Endah Kanti Pangestuti et al., studied the flexural strength of reinforced concrete beam with Carbon Fibre Reinforced Plate (CFRP) to estimate the effectiveness of using CFRP on the concrete structure as external reinforcement. The sample of two beams was used in this study using experimental research. The finding shows that the beam with CFRP located externally show not a good performance as compared to normal beam. It also can be concluded that the composite material has not been able to work optimally under static loading.

Samuel Ekung and Emmanuel Adu identified the limitations of risk identification tools applied in project management in the Nigerian construction industry. Mixed methodology was applied using the sample of project managers in the Nigerian construction industry. The findings show that majority of the respondents using Risk premium allowance, Assumptions and Constraints Analysis and WBS Review. The weakness of Assumptions and Constraints Analysis has been identified to prone to missing latent assumptions and system constraints. Besides that, the Document Reviews has identified to have weakness to only precedential risks.

Antas Sinaga et al., explored the seismic response of flexure-shear failure of Reinforced Concrete (RC) Structure designed using FEMA 356 criteria. The important factor for seismic evaluation is to identify appropriately, effective stiffness and plastic hinge model to evaluate the structures. The finding shows that theoretical hinge structure is slightly more rigid than FEMA 356 structure. Though previous research shown that member stiffness does not constantly and correlates with the reinforcement provided, but this research show that the theoretical hinge can deform further without losing the capacity.

Mohd Haziman Wan Ibrahim et al., assessing the experimental investigation on the effect of Coal Bottom Ash (CBA) as sand and cements replacement on compressive strength and water permeability of concrete. The different grinding times was set as 20, 30 and 40 hours where the replacement levels are 10%, 20% and 30%. The findings shows that at 20% replacement level of CBA in 30 hours grinding time has potential for cement replacement whereas 15% of replacement of CBA was the optimum for sand replacement.

Editorial Committee

A STUDY ON QUALITY OF COST DATA IN LIFE CYCLE COST ANALYSIS OF MAINTENANCE DURING THE IN-USE PHASES OF UNIVERSITY BUILDING

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Abstract
Life Cycle Cost (LCC) analysis is an economic assessment technique that can help building owners to determine the most optimum total cost of maintaining the building and to compare the most cost-effective of mutually exclusive alternatives by discounting the future cost to today's value. However, many commentators pointed out that the key setback of implementing LCC technique is the difficulty on identifying and procuring quality and reliable data, which can be used as inputs into the process of producing a complete and reliable LCC analysis. The objective of this paper is to present the outcomes of the study on the quality of cost data as inputs for LCC analysis of maintenance during the in-use phases of university building. The methodology employed for the study is a qualitative research strategy that comprises of literature review and semi-structured interview. The quality of cost data of the operation and maintenance cost of the university building was studied to determine the readiness of the cost data as inputs into the process of producing a comprehensive and reliable LCC analysis of maintenance during the in-use phases for the university building maintenance management practice. The semi-structured interview was carried out with the identified group of people that have knowledge, skills or experience in the field of university building maintenance practice with the objectives to obtain feedback and opinions on the state of data quality, and to identify strategies that can be recommended to improve the quality of data for the practice of LCC analysis of maintenance during the in use-phases of university building. The results have established that the operation and maintenance cost data of the university building are not readily available, accessible, current and reliable to be used as inputs in producing a comprehensive and reliable LCC analysis of maintenance during the in-use phases of university building. To overcome this setback, the majority of the interviewees recommended there is a need to develop an appropriate methodology encompasses with strategies that can improve the quality of data in effort to produce a comprehensive and reliable LCC analysis of maintenance during the in-use phases of university building.

Keywords: *Life Cycle Cost (LCC); cost data; quality; university building maintenance*

INTRODUCTION

Transforming Malaysia into a centre of higher education excellence is one of the important strategic missions that has been given high commitment by the government to transform Malaysia into a high-income nation (Olanrewaju et al., 2010; Olanrewaju and Abdul-Rashid, 2015). One of the key strategies initiated by the Malaysia government to achieve this strategic mission is to operate and maintain effectively the education buildings in Malaysia, particularly in the universities to ensure the teaching and learning facilities provided in the higher education centre can meet the physical needs of world-class teaching, learning and research environments (Olanrewaju et al., 2010). The education sector has received the highest national development budget every year, where large amount of money has been spent to maintain the education buildings in Malaysia universities. For example, in 2004, the Malaysia government has spent RM304 million for the maintenance of teaching and learning facilities in the education buildings in Malaysia universities. However, the maintenance expenditure in 2008 has been highly increased to RM600 million that is nearly double than the amount spent in 2004 (i.e. RM304) (Olanrewaju et al., 2010).

Generally, LCC analysis has been recognised as an economic assessment technique in the construction industry that can estimate all costs related to the ownership of the assets and buildings, which integrates the initial capital costs with future costs, i.e. financial costs, operation costs, maintenance and replacement costs and salvage cost throughout anticipated lifespan (Davis Langdon Management Consulting, 2006; BS ISO 15686-5, 2008; Langdon, 2010; Ayob and Abdul Rashid, 2015, 2016). LCC analysis is an appropriate technique that can compare the most cost-effective amongst the exclusive alternatives that eventually contribute to the cost savings measure (Boussabaine & Kirkham, 2006). The LCC analysis produces cost information that can facilitate the building owner to determine the most optimum budget required that can be budgeted and kept aside by the building owner from the very early stage of project life cycle to spend for the future costs of operating and maintaining the building during the in-use phases until the end of the service life (Davis Langdon Management Consulting, 2006; BS ISO 15686-5, 2008; Langdon, 2010; Ayob, 2014; Ayob and Abdul Rashid, 2015, 2016).

Nevertheless, the majority of the studies reported that the key limitation of applying LCC analysis is the problem of getting a comprehensive data that is reliable and current, which can be used as quality inputs in the estimation of LCC analysis (Siti Hamisah et al., 2007a; Masoud, 2009; Masoud et al., 2010 as cited by Ayob & Abdul Rashid, 2013, 2015, 2016). The literature study has established that the quality of data inputs used in LCC analysis is of paramount importance that should be emphasized by the cost estimators in order to produce a complete and reliable LCC analysis that can produce quality output. The literature study has identified the following as the key quality of data input requirements required for producing a comprehensive and reliable LCC analysis (Ayob, 2014; Ayob & Abdul Rashid, 2016, 2015, 2013, 2011; Ayob et al., 2017):

i. **Availability** of cost data indicates the level of data certainty (Gross and AEA, 2008; NATO Research and Technology Organisation, 2009; BS ISO 15686-5, 2008; BSI, 2008; Goh et al., 2010; Davis Langdon Management Consulting, 2007, as cited by Ayob, 2014; Ayob & Abdul Rashid, 2016, 2015, 2013, 2011; Ayob et al., 2017).
ii. **Accessibility** of cost data is defined as the ease of access to obtain cost data from data sources or suppliers within known background (NATO Research and Technology Organisation, 2009, as cited by Ayob, 2014; Ayob & Abdul Rashid, 2016, 2015, 2013, 2011; Ayob et al., 2017).
iii. **Current** data means as the most recent and advanced data that are updated on monthly basis or yearly basis (Free Dictionary, 2015, Khairani, 2009, as cited by Ayob, 2014; Ayob & Abdul Rashid, 2016, 2015, 2013, 2011; Ayob et al., 2017).
iv. **Reliability** of cost data refers to the consistency of data which implies how comparable the data to the actual value that arrived from similar and repetitive methods under the same research situation (Creswell & Clark, 2007, Neuman, 2003, Ashworth, 2004, Ayob et al., 2017; Ayob, 2014; Ayob & Abdul Rashid, 2016, 2015, 2013, 2011; Ayob et al., 2017).

The objective of this paper is to present the outcomes of the study on the quality of cost data as inputs for LCC analysis of maintenance during the in-use phases of university building. The study focuses only on the cost data of operation and maintenance of the university building that made available for viewing in one of the universities in Malaysia. The quality of the cost data of operation and maintenance of the university building was studied in order to determine the readiness of the cost data as inputs into the process of producing a comprehensive and reliable LCC analysis of maintenance during the in-use phases of the university buildings. However, the name of the chosen university building, which becomes the subject matter of the study is anonymised and not reported in this paper. This paper follows the other paper that has been presented elsewhere by the author (Bidi, 2015).

LITERATURE REVIEW

The maintenance management plays a vital role in maintaining the buildings to ensure the buildings are working in good performance condition and to maximize its service life. The role of maintenance management is not only to ensure the building can be functioned properly in meeting the desired requirements but also to optimise the maintenance expenditure throughout the entire service lifespan (Abdul Lateef et al., 2015).

Many scholars pointed out that LCC analysis should be applied in the investment decision-making process to help building owners and clients to optimise the total ownership costs of the building over the investment life. The output of LCC analysis is a very useful cost information to the building owners and clients, which can assist them to measure their financial capability in paying the initial and future costs of the building, and to compare the most cost-effective amongst the mutually exclusive alternatives prior to investment decision making (Kirk & Dell'Isola, 1995; BS ISO 15686-5, 2008; BSI, 2008; NATO Research and Technology Organisation, 2009; Kelly & Hunter, 2009; Ashworth, 2010; ASTM International, 2010, as cited by Ayob, 2014; Ayob and Abdul Rashid, 2011, 2013, 2015, 2016).

The literature study has identified that the cost components of LCC analysis of building are included of the initial capital costs, financial costs, operation costs, maintenance and replacement costs, and salvage cost (BSI 2008, BSI 15686-5, 2008; AS/NZS 4536, 2014). Table 1 illustrates the definition of each category of cost components of LCC analysis of the building.

The LCC analysis process can be categorized into three main phases, i.e. data inputs, conversion and output (BS ISO 15686-5, 2008; Kelly & Hunter, 2009; NATO Research and Technology Organisation, 2009; Rist, 2011, as cited by Ayob, 2014). Cost data are the most important inputs in LCC analysis, which have to be identified from a very early stage of LCC estimation process. In addition, the cost estimators must ensure that they identify cost data that are comprehensive, current and reliable, which can be used as quality inputs in the estimation of LCC analysis (Ayob and Abdul Rashid, 2011, 2013, 2015, 2016; Ayob, 2014; Ayob et al., 2017).

Table 1. The cost components of LCC analysis of building

Component	Description
Initial capital cost	- All investments towards completion including decommissioning by the end use of the facilities including land acquisition cost, construction work and other related cost and client definable cost.
Financial cost	- The cost of the money financing the business assets.
Operating cost	- The total cost expected to maintain in daily, weekly and monthly that are repetitive within one-year period for building and technical installation system to satisfy given functional demands and requirements.
Maintenance and replacement cost	- Maintenance cost includes all activities and efforts put forward in a period of more than one year by including planned maintenance, replacement, and emergency repairs in order to ensure the buildings and technical systems are in the original quality and function. - Costs of replacement and repairs are costs of activities that are undertaken to take into account the changes to particular elements, materials, and structures.
Salvage cost	- Salvage cost refers to the projected resale value of an asset at the close of its useful life. The salvage value would be deducted from the cost of a fixed asset to decide the quantity of the asset price that will depreciate.

(Sources: BSI, 2008; BSI ISO 15686-5, 2008; Langdon, 2010; Kristic & Marenjak, 2012; International Financial Reporting Tool, 2015; Matrixlab-Examples.com, 2015; Alberto Torres & Carlos Bustamante, n.d)

The Net Present Value (NPV) has been identified as the most popular mathematical cost model in LCC analysis to estimate the total cost of the building (Langdon, 2010, ASTM International, 2010). The basic requirements of NPV estimation are the study life (n), discount rate (i) and the yearly cost forecast estimates (Boussabaine & Kirkham, 2006). The NPV refers to an amount of investment required today to meet the future financial requirements over an anticipated period. The NPV method calculates the asset's net contribution by estimating the differences between the discounted present value and actual cost in today's value (Schade, 2007; BS ISO 15686-5, 2008; Langdon, 2010). Study life is an analysis period of LCC analysis that indicates the anticipated period of owning the asset until the end of the service life (Brandon, 1995; Stanford University Land and Buildings, 2005; BS ISO 15686-5, 2008, as cited by Ayob, 2014). A similar study period is appropriate to be used in the LCC of analysis when comparing the most cost-effective amongst the mutually exclusive alternatives. In the LCC analysis, the discounted rate is the parameter used to represent the time value for money. The time value for money depends on the inflation cost of capital, investment opportunities and personal consumption preferences (Davis Langdon Management Consulting, 2006).

There are several types of mathematical NPV cost models that can be applied in the LCC analysis of building maintenance in the construction industry. One of the NPV mathematical formulas used in the LCC analysis is shown as follows:

$$\boxed{NPV=C+R-S+A+M+E}$$

Where:

C = investment cost
R = replacement cost
S = the resale value at end of study period
A = annually recurring operating, maintenance and repair cost (except energy, cost)
M = non-annually recurring operating, maintenance and repair cost (except energy, cost)
E = energy cost

NPV mathematical formula was initially introduced by the American Society for Testing and Materials (ASTM 1983) to estimate the life cycle cost of building and system over the anticipated lifespan (Fuller, 2005; Nor Azizah & Zainal Abidin, 2010; Schade, 2010).

Based on a comprehensive review of literature, it was observed that there are six cost components of operation cost for the university building, which include utilities cost, cleaning cost, administration cost, security cost, overhead cost, local and statutory charges in connection with the building (Ayob and Abdul Rashid, 2011; Ayob, 2014). Table 2 provides the identified published cost data currently available in the Malaysian construction industry, and the alternative data sources that can be used as inputs to estimate the operation cost of the university building.

Table 2. Published cost data for the operation cost of university building

Cost component	Identified sources	Alternative data sources
Utilities cost *e.g. water, electricity, gas*	i) Tenaga Nasional Berhad, http://www.tnb.com.my/business/for commercial/pricing-tariff.html ii) Indah Water Consortium, http://www.iwk.com.my/v/customer /commercial iii) Syarikat Bekalan Air Selangor Sdn. Bhd (SYABAS), http://www.syabas.com.my/consumer/water-bill-water-tariff iv) Gas Malaysia and Suruhanjaya Tenaga http://www.st.gov.my/index.php/consumer/gas/tariff.html	-
Cleaning cost e.g. cleaning classroom, offices and other academic facilities	No published cost data	Historical data
Administration cost *e.g. salaries of supervisory staff, professional staff, administrative staff, and the use of the equipment and computer for supporting the administration of the maintenance*	No published cost data	Historical data
Security cost *e.g. high-security arrangements that required personnel, digital entry points, alarm system and closed-circuit television*	No published cost data	Data from the manufacturers, suppliers, contractors and testing specialists
Overhead cost *e.g. Insurance for the building paid by owner*	No published cost data	Historical data
Local and statutory charges in connection with building operation *e.g. charges in connection with the requirement of official approval, licenses or permit and pre-application advice particularly on the planning law and planning policy*	No published cost data	Historical data

(Sources: Al Hajj, 1991; Ashworth, 2004; BSI, 2008; Khairani, 2009; Langdon, 2010; Ayob and Abdul Rashid, 2011; SYABAS, 2011; Gas Malaysia and Suruhanjaya Tenaga, 2013; Department for Communities & Local Government, 2014; Business Dictionary.com, 2015; Indah Water Konsortium, 2015; Tenaga Nasional Berhad, 2015)

As referred to Table 2, it can be concluded that utilities cost are the only published operation cost data that are available for public viewing. The utilities cost data are published by the following agencies in their official websites:

i. Tenaga Nasional Berhad (TNB),
ii. Syarikat Bekalan Air Selangor Sdn. Bhd. (SYABAS),
iii. Indah Water Konsortium (IWK),
iv. Gas Malaysia Berhad, and
v. Suruhanjaya Tenaga.

However, the literature study has not identified cost data for other cost components of building operation cost, i.e. cleaning cost, administration cost, security cost, overhead cost, local and statutory charges because the data are not published for public viewing. To overcome the non-existence of published cost data that are required for the estimation of university building operation costs, Davis Langdon (2010) has suggested three categories of alternative data sources that can be referred by the cost estimators as inputs to estimate the operation costs of the university building:

i. Data from manufacturers, suppliers, contractors and testing specialists,
ii. Historical data, and
iii. Data produced from the application of mathematical models

Commonly, the data from manufacturers, suppliers, contractors and testing specialists are based on extensive knowledge of the performance and characteristics of their material and components. Whilst, the historical data is the data from the organisation's internal data, and feedbacks from operational assets and published benchmark sources from the past projects (Langdon, 2010). Based on the outcomes of literature review reported in Table 2, it is not misconception to state that the historical data of similar characteristics of building projects from the past arranged periods of study is the most appropriate alternative data source that can help cost estimators to overcome the limitation of data when there are no published cost data available for cleaning cost, administration cost, security cost, overhead cost and local and statutory charges to estimate the university building operation cost. Nevertheless, it is important for the cost estimators to take into account the problem of using of historical data that can provide misleading information because the data are less current. Hence, the historical data must be cleaned and updated first before they can be used as inputs in LCC analysis. This is to ensure the LCC outputs produced are current enough in facilitating the clients to establish updated and reliable decision making (Siti Hamisah et al., 2007; Masoud, 2009; Masoud et al., 2010; Ayob and Abdul Rashid, 2011, 2013, 2015, 2016; Ayob, 2014).

The maintenance and replacement costs are included as the cost components of LCC analysis of building maintenance during the in-use phases. These maintenance and replacement costs are paid by the clients and building owners when the building is started operating in keeping the building in good working performance condition and performance throughout the service life span. A new LCC guideline has been published by the British Standard, i.e. *BS 8544 (2013)-LCC of maintenance during the in-use phase of buildings* that takes forms of guidance and recommendations on improving the application of LCC analysis in building maintenance management practice. This guideline provides standardized rules and methodology that can guide the cost estimators on the methodologies and techniques of applying LCC analysis of maintenance and renewal during in-use phases of buildings or systems (as illustrated in Figure 1). The LCC analysis of building maintenance produces output, which can be useful cost information to the building owners and clients in facilitating them to plan and prioritise the maintenance budget before undertaking the building maintenance and renewal works (8544, 2013).

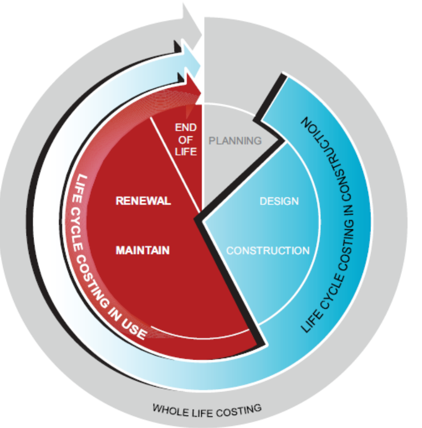

Figure 1. LCC analysis during the various phases of a building's life. (Sources: BS 8544, 2013)

Nevertheless, based on a comprehensive review of literature, there is no methodology of data collection currently available in Malaysia that can guide and facilitate the cost estimators to identify and collect maintenance cost data, and solution strategies that can help them to mitigate the setback of LCC practice in university building maintenance due to the problem of non-available maintenance cost data. Based on the outcomes of a comprehensive literature review, there are several types of alternative data sources that can be considered appropriate to overcome the problem of non-available maintenance cost data in the LCC analysis of university building maintenance. Table 3 below shows the alternative data sources that are appropriate to be used as inputs in LCC analysis of university building maintenance, which can be obtained by interviewing manufacturers, suppliers, contractors and the testing specialists that have wide knowledge, skills and experience in the construction industry (Flanagan and Jewell, 2005).

Table 3. Alternative data sources for LCC analysis of university building maintenance

Cost Component	Description	Alternative data sources
Cost of regular custodial care	A custodial service provides cleaning, moving, and general services such as pest control, recycling collection, reporting of maintenance needs, furniture moves, and others.	Data from the manufacturers, suppliers, contractors and testing specialists
Renewal cost (e.g. repair and replacement cost)	Repair and re-establishment of building component to original function.	Data from manufacturers, suppliers, contractors and testing specialists
Annual maintenance contracts	Fixed fee services provided by the contractor for maintenance of the building.	Historical data
Maintenance management	Maintenance management software such as Computerized Maintenance Management System.	Data from manufacturers, suppliers, contractors and testing specialists
Refurbishment cost of the building	Renovation, cleaning up, restoration to its better-looking condition.	Data from manufacturers, suppliers, contractors and testing specialists, Data from modelling techniques
Redecoration cost of the building	Beautify or furnish something with ornamental e.g. redecorate wall with murals.	Data from manufacturers, suppliers, contractors and testing specialists

Cost Component	Description	Alternative data sources
Salaries of facility staff performing maintenance task	Salary to facilities maintenance technicians that perform building maintenance task such as cleaning, maintaining, and repairing.	Historical data

(Sources: BS ISO 15686-5,2008;BSI, 2008;Fuller,2009; Kelly and Hunter, 2009; University of Wisconsin, n.d; Langdon,2010; Ayob and Abdul Rashid,2011; BS 8544, 2013;Business Dictionary.com, 2015; Dictionary.com,2015; The Free Dictionary,2015; Gateway Technical College,2015)

METHODOLOGY DESIGNED FOR THE STUDY

There are three types of research strategies, i.e. qualitative research, quantitative research and mixed method research. For this study, the qualitative research strategy was chosen because the nature of LCC data input research is subjective and profound. Hence, this study requires the researcher to study the quality of cost data inputs in LCC of maintenance based on the opinions, views and feedbacks from a group of people that have knowledge, skills and experience in university building maintenance management practice (Ayob and Abdul Rashid, 2011, 2013, 2015, 2016; Ayob, 2014; Mansor et al., 2016).

The qualitative research strategy for this study comprises of literature review (secondary data collection) and semi-structured interview (fieldwork approach). A comprehensive literature review was carried out in the initial stage of the study to gain a clear picture and understanding on the research background and topic.

FIELDWORK APPROACH

A semi-structured interview was chosen as the most appropriate fieldwork approach rather than other typical approaches (e.g. surveys, case studies, action research) because the nature of LCC data input research is subjective that requires the researchers to obtain opinions, views, and perceptions from people that have skills, experience, expertise and knowledge in the field of university building maintenance (Ayob, 2014; Ayob and Abdul Rashid, 2015, 2016). Besides that, the literature study has identified several researchers that had chosen interview as the fieldwork approach to study the LCC data inputs of building, which include Abdul Lateef et al. (2010), Mohammed Rum and Akash (2011) and Ayob (2014). The semi-structured interview was carried out in the study to achieve the following objectives;

i. To identify the availability, accessibility and currency of cost data as input for LCC analysis of maintenance during the in-use phases of university building; and
ii. To obtain valuable opinions and feedback from experts regarding the most appropriate strategies to improve the quality of cost data as inputs for producing a comprehensive and reliable LCC analysis of university building maintenance.

Figure 2 shows a schematic flow of the interview process that begins with designing research questions and ending with summarise the interview responses. The researchers constructed the questionnaire in an open-ended format with the objective to obtain a wide range of responses from the respondents. In order to improve the quality of the questionnaire, a pilot questionnaire was carried out through face-to-face interaction with research supervisor and academicians. Revision was made to the questionnaire based on the outcomes of pilot questionnaire. The questionnaire is divided into two sections, i.e. Sections A and B. Section A requires the respondents to complete the profile information and professional qualification with the objective to identify and establish the expertise, skills, and knowledge of the respondents in the field of university building maintenance. Whilst, Section B is designed to obtain valuable opinions and feedback from the respondents in determining the state of data availability, accessibility and currency as inputs required for generating a complete and reliable LCC analysis of maintenance during the in-use phases of the university building. Apart from that, the respondents were required to propose strategies that can improve the quality of cost data used as inputs in LCC analysis of university building maintenance. The researcher identified suitable interviewees in the study through literature searches, recommendations from the institutions and snowball method. The snowball method was carried out to obtain recommendations from experts of people that have knowledge, skills, and experience in the university building maintenance management practice that are potential to be invited to become respondents in the interview study. The respondents that agreed to be involved in the interview must be able to understand the aim and objective of the research, and competent to make judgement based on the evidence of expertise (e.g. positions, experiences, publications, etc.). The study has identified seven potential people that that met all the criteria required and potential to become interview respondents. However, only five respondents were able to be interviewed and completed the interview questionnaire (see Table 4).

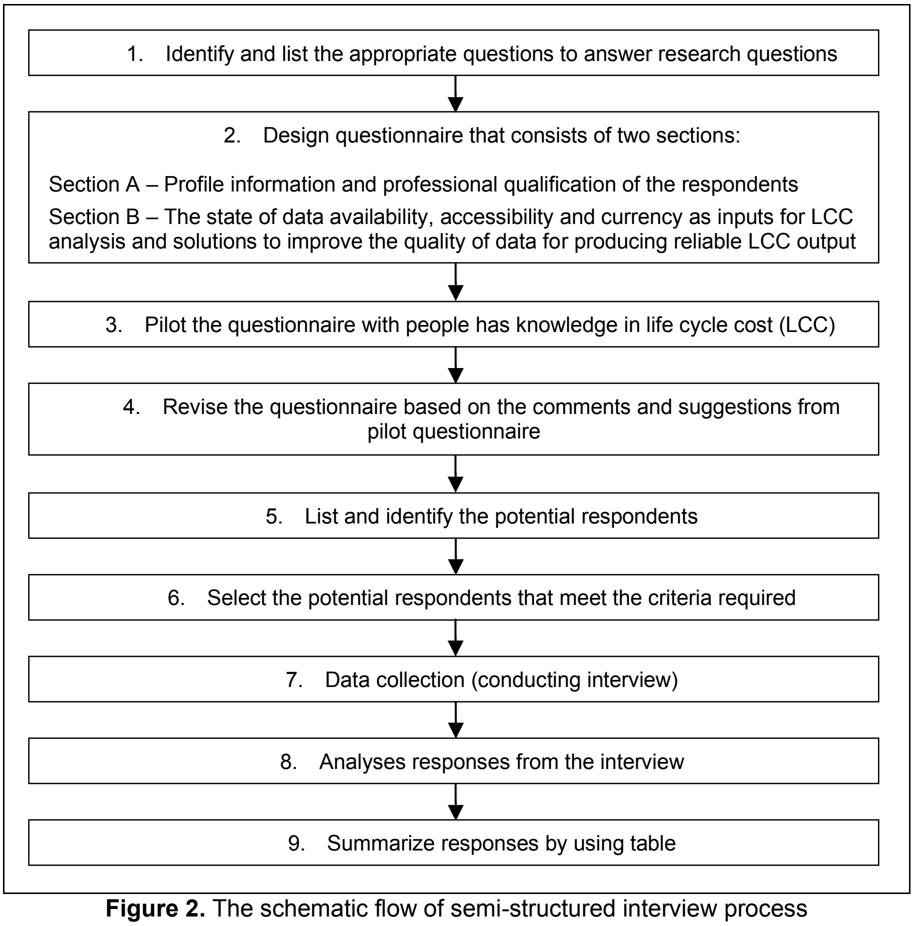

Figure 2. The schematic flow of semi-structured interview process

Table 4. List of interviewees participated in the study

No	Respondent	Designation	Current Organization
1	A	Architect, Academic	Faculty of Architecture
2	B	Engineer	University Development Division
3	C	Cost/Financial Planner	University Development Division
4	D	General Manager	Facility management organization
5	E	Cost/Financial Planner	University Finance Division

THE SETBACKS OF INTERVIEW APPROACH

The following are the key setbacks of implementing the interview approach in collecting primary data for the study:

i. The researchers have limited time to complete the study within six (6) months. Due to this time constraints, the researchers decided to focus only to get views and observe opinions from related respondents that have knowledge, skills and experience in the university building maintenance management practice.
ii. The researchers found difficulty to get agreement from the respondents to meet at a specified time for possessing the interview due to their tight schedules. Hence, the researchers have to contact the respondents several times to find their availability time for the interview.
iii. The researchers have limited access to the internal database of university building maintenance that confined by the facility management companies in the library. Besides that, the data are not freely published and restricted for public viewing.

RESULTS AND DISCUSSION

The results of the semi-structured interview questionnaire are presented in two different sections, i.e. Sections A and B. The Section A presents the demographic profile of the respondents, and which information that establishes the skills, knowledge or expertise of the respondents in the field of cost management in university building maintenance practice. Whilst, the Section B presents the outcomes of data analyses based on the feedback and opinions given by the respondents on the state of data availability, accessibility and currency as inputs for the practice of LCC analysis of maintenance, and suitable solutions that can be recommended to improve the quality of data used for the practice of LCC analysis of maintenance during the in use-phases of university building. The summary of the responses collected from the semi-structured interview are presented in Table 5. As referred to summary of the responses reported in Table 5, the majority of the respondents chosen historical data as the most preferred data that appropriate to be referred in the estimation of LCC analysis of maintenance during the in use-phases of university building. The finding is consistent with other scholars such as Flanagan and Jewell (2005), Schade (2007) and Langdon (2010) that suggest the historical data is appropriate to be used as an alternative data to help the cost estimators in computing the LCC analysis of university building maintenance. However, Ayob (2014) in his LCC data input study has suggested the data which are published by several local agencies in the official websites, i.e. Tenaga Nasional Berhad (TNB), Indah Water Konsortium (IWK), Syarikat Bekalan Air Selangor Sdn. Bhd (SYABAS), etc. are appropriate to be used as inputs in LCC estimation because the data are real time and always updated by the respective agencies.

There are many strategies recommended by the respondents, which can be considered to improve the quality of data used in the LCC analysis of maintenance during the in use-phases of university building. The results in Table 6 show the three top priority strategies recommended by the respondents, which worth considering for recommendation to overcome the problem of data improving the quality of data for enhancing the practice of LCC analysis of maintenance during the in use-phases of university building.

Table 6. The top three priority strategies for recommendation to overcome the problem of quality data in LCC analysis of maintenance during the in use-phases of university building

Key quality of data inputs	Recommendations
Data availability	• The estimators have to refer to the past data that have the same work or equal project and detailed billing but limit to first five years project, i.e. year 2011 and above (2 respondents) • Referring to Facility Management. (1 respondent)
Data accessibility	• The Government should make data accessible for the good use of practitioner i.e. creating database. (1respondent)
Data currency	• Comparative analysis is proposed and looking increment year by year (first 5 years) to extrapolate for future cost. (2 respondents)

The results from the interview show that the majority of the respondents believed the main problem of carrying out LCC analysis of maintenance during the in-use phases of university building is the difficulty to obtain complete, current and reliable cost data as the inputs required for producing correct and reliable LCC output. Hence, to overcome the limitation of data in the LCC analysis of maintenance during the in use-phases of university building, the majority of the respondents recommended that the past data from the first five years of similar project types can be chosen as an alternative data source in LCC analysis. However, the data must be cleaned and updated first before they can be used as inputs for LCC analysis of maintenance during the in use-phases of university building. In addition, the majority of the respondents recommended that the Government through related technical agencies can take initiative to develop a database system, which can be can be subscribed by the cost estimators through the subscriber-base information services providers in facilitating them to undertake the LCC analysis of maintenance during the in use-phases of university building. On the other hand, the study carried out by Ayob (2014) has suggested that the quality of cost data used in LCC analysis of building can be improved by developing a protocol of LCC data input requirements process that provides procedures to identify, access, update and check the reliability of data before the data can be used as inputs into for producing reliable LCC outputs for the practice of university building maintenance in the Malaysian construction industry. Hence, it is not a misconception to state that there is a need of study to be carried out to identify ways to improve the practice of LCC analysis of maintenance during the in use-phases of university building by giving greater emphasis on improving the quality of data.

Table 5. Summary of responses collected from the semi-structured interview

COST COMPONENTS	Source of Reference											
	Services Providers (TNB, SYABAS, etc.)	Bills	Subcontractor Quotation	Facility Managers	Public Work Department	Contract Document	Supplier	Past data record of project	Insurance Companies	Historical Data	Not available, accessible and current	Number of Respondents Total
Utilities cost	2	1								1		4/5
Cost of regular custodial care and cleaning cost			1							3		4/5
Administration cost				1	1					2		4/5
Security cost			1							1	3	5/5
Local and statutory charges						3				1		4/5
Repair and Replacement cost							1			3		4/5
Salaries of facility staff performing maintenance task					1			1		2		4/5
Maintenance management				3							1	4/5
Annual Maintenance Contracts						2				2		4/5
Redecoration cost										1	3	4/5
Refurbishment cost										2	2	4/5
Overhead cost									1	1	3	5/5
Total	2	1	2	4	2	5	1	2	1	19	12	

CONCLUSION AND RECOMMENDATIONS

This paper has presented the outcomes of the study on the quality of cost data used as inputs in LCC analysis of maintenance during the in-use phases of the university building. The methodology implemented in the study comprises of two approaches, i.e. literature review (secondary data collection), and semi-structured interview (primary data collection). The semi-structured interview was carried out with a selected group of people that have knowledge, skills or experience in the field of university building maintenance practice. The outcomes of the study have established that the operation and maintenance cost data of the university building in the Malaysian construction industry are not readily available, accessible, and current, and therefore the data are not reliable enough to be used inputs for LCC analysis of maintenance during the in-use phases of the university building. However, the majority of the respondents have reached agreement that the data of similar project types can be chosen as an alternative data source for the estimation of operation and maintenance cost in LCC analysis of maintenance during the in-use phases of the university building. However, the historical data that are chosen to be used in LCC estimation must be cleaned and updated first before can be used as inputs in the calculation of LCC analysis. The study has proposed there is a need to develop appropriate methodologies of data input requirements for LCC analysis of maintenance during the in-use phases of the university building that can facilitate the LCC estimators to identify, collect and record quality cost data in producing a comprehensive and reliable LCC analysis of maintenance for the university building. Further research is recommended to be carried out as second part of the study to develop a protocol that is embedded together with procedures, which can enhance the quality of cost data for improving the practice of LCC analysis of maintenance during the in-use phases of university building in Malaysia.

ACKNOWLEDGEMENT

The authors would like to express highly appreciations to International Islamic University Malaysia (IIUM) for supporting this research and to the Ministry of Education Malaysia for providing partial funding for the research under RAGS (ref: RAGS/1/2014/TK02 /UIAM//1)”.

REFERENCES

Abdul Lateef, Olanrewaju, Idrus, Arazi and Khamidi, Mohd Faris (2010). Building Maintenance Management in a Malaysian University Campuses: A Case Study. *Australasian Journal of Construction Economics and Building*, 10 (1/2) 76-89.

Abdul Lateef, Olanrewaju and Abdul-Rashid, Abdul-Aziz (2015). Building Maintenance Process and Practices. The case of a fast developing country. New York, London.

Alberto, T. and Carlos, B. (n.d.). Cost of Capital, Economic Profit and ABC. Focused Management Information. Retrieved March 6, 2015. http://www.focusedmanagement.com/.

Al Hajj, Assem Nazih (1991). *Simple cost-significant models for total life cycle costing in buildings*. Unpublished doctoral dissertations, Department of Civil Engineering, University of Dundee. Retrieved April 16, 2015. http://discovery.dundee.ac.uk/portal/

Ashworth, A. (2004). Cost studies of buildings (4th Ed.). London: Pearson Education Limited

ASTM International (2010). E917-05: *Standard Practice for Measuring Life –Cycle Costs of Buildings and Buildings Systems*. United States: ASTM International. Retrieved October 6, 2010 from Georgia Institute of Technology database.

Ayob, Mohd Fairullazi and Abdul Rashid, Khairuddin (2011). Proposing a Methodology to Investigate the Reliability and Validity of Data Inputs for Building (LCC). In: Management in Construction Researcher's Association (MICRA) 10th Annual Conference and Meeting 25, 25th-26th July 2011, Gombak, Kuala Lumpur. (Unpublished).

Ayob, Mohd Fairullazi and Abdul Rashid, Khairuddin (2013). *Strategies to enhance quality data input requirements of life cycle cost (LCC).* Journal of Architecture, Planning and Construction Management, 3 (2). pp. 44-67. ISSN 2231-9514. Publisher: Kulliyyah of Architecture and Environmental Design, International Islamic University Malaysia (Indexed by Google Scholar and Open Access).

Ayob, Mohd Fairullazi (2014). Cost data inputs in life cycle cost analysis in Malaysia: Assessment of data and development of proposed strategies and protocol to enhance data quality.

Ayob, Mohd Fairullazi and Abdul Rashid, Khairuddin (2015). *Investigation of Quality of Cost Data Input in Life Cycle Cost (LCC) Analysis in Malaysia.* Journal of Architecture, Planning and Construction Management, 5 (2). ISSN 2231-9514. Publisher: Kulliyyah of Architecture and Environmental Design, International Islamic University Malaysia, Google Scholar, Open Access).

Ayob, Mohd Fairullazi, and Abdul Rashid, Khairuddin (2016). *Review Of Methodology Designed To Investigate Quality Of Cost Data Input In Life Cycle Cost.* Malaysian Construction Research Journal (MCRJ), Volume 18, Issue 1, 2016. ISSN No.: 1985-3807. Publisher: Construction Research Institute of Malaysia (Indexed by Scopus Elsevier).

Ayob, Mohd Fairullazi, and Bidi, Nor Khalisah and Ahmad Jasmi, Siti Zulaiha and Wan Omar, Wan Imran and Ali, Maisarah (2017). *Life Cycle Cost and Performance Assessment: Comparison between Reconstruction and Cold-In-Place Recycling (CIPR) Methods*. Malaysian Construction Research Journal (MCRJ), Volume 21, 2017. Publisher: Construction Research Institute of Malaysia (In Press) (Indexed by Scopus Elsevier).

Bidi, Nor Khalisah (2015). Life Cycle Costing of University Building Maintenance Cost Rate. Unpublished bachelor degree dissertation, International Islamic University Malaysia.

Boussabaine, H.A, and Kirkham, R.L. (2006). Whole life cycle costing: risk and responses. Oxford, UK: Blackwell Publishing Ltd.

BSI ISO 15686-5 (2008). *International Standard: Building and constructed assets-Service Life Planning. Part 5; Life Cycle Costing* (pp 1-40). Standards Policy and Strategy Committee. Retrieved April 16, 2015, from University of Bradford database

BSI (2008). *Standardized method of life cycle costing for construction procurement; a supplement to BS ISO 15686-5 Buildings and constructed assets- Service Life Planning. Part 5: Life Cycle Costing* (pp 1-87). London: British Standards Institutions. Retrieved April 16, 2015 from Universiti Tun Hussein Onn Malaysia database.

BS 8544 (2013). *Guide for life cycle costing of maintenance during the in use phases of buildings.* The British Standards Institution 2013.

Business Dictionary.com (2015). Maintenance contract. Retrieved April 23, 2015 from http://www.businessdictionary.com/definition/maintenance-contract.html

Che Ghani, N.Z., Myeda, N.I. & Ali, A.S. (2016). *Operation and maintenance cost for stratified buildings: A Critical Review.* MATEC Web Conferences 66, 00041 (2016) IBC 2016. Published by EDP Sciences.

Davis Langdon Management Consulting (2006). Literature review of life cycle costing (LCC) and life cycle Assessment (LCA). Draft Literature Review for LCC Methodology Project.

Department for Communities & Local Government (2014). Planning Practice Guide. Retrieved April 16, 2015. http://planningguidance.planningportal.gov.uk /blog/guidance/when-is-permission-required/what-is-development/

Dictionary.com (2015). Decorate. Retrieved April 23, 2015 from http://dictionary.reference.com/browse/redecorate

Flanagan, R. & Jewell, C. (2005). Whole Life Appraisal for construction. Oxford: Blackwell Publishing Ltd.

Fuller, S. K. (2005). Guidance on Life-Cycle Cost Analysis Required by Executive Order 13123. Washington: Sieglinde, K. F. Retrieved March 24, 2015.

Gas Malaysia and Suruhanjaya Tenaga (2013). Natural Gas Tariff for Gas Malaysia Bhd. Customer. Retrieved March 25, 2015. http://www.st.gov.my/index.php/consumer/gas/tariff.html

Gateway Technical College (2015). Facilities Maintenance. Retrieved April 24, 2015. https://www.gtc.edu/programs/technical-diploma/facilities-maintenance

Indah Water Konsortium (2015). Sewerage Charges. Kuala Lumpur: Author. Retrieved March 25, 2015. http://www.iwk.com.my /v/customer/commercial

International Financial Reporting Tool (2015). Salvage Value. Retrieved March 24, 2015. http://www.readyratios.com/

Jennifer, R. (2012). Conducting research interviews. *Management Research Review*, Vol. 35 Iss 3/4 pp. 260 – 271. Retrieved March 8, 2015 from Emerald insight database.

Kelly, J.R. and Hunter, K. (2009). Life Cycle costing of sustainable design. London: RICS. Retrieved April 16, 2015. http://www.rics.org/site/scripts/download _info.aspx?downloadID=4529-24k

Khairani, A. (2009). Construction Economics, Selangor, Malaysia; Prentice Hall

Krstić, H. and Marenjak, S. (2012). Analysis of buildings operation and maintenance costs. Retrieved September 26, 2014.

Langdon, D. (2010). Development of a Promotional Campaign for Life Cycle Costing in Construction Final Report 19 January 2010. United Kingdom: Davis Langdon.

Mansor, Nur Syaimasyaza and Ayob, Mohd Fairullazi and Abdul Rashid, Khairuddin (2016) *Incomplete Contract in PFI: A Modified Delphi Study.* Advanced Science Letters. ISSN 1936-7317 (O), 1936-6612 (P) (In Press), (Indexed by Scopus).

Matrixlab examples.com (2015). Salvage value calculator. Retrieved March 24, 2015. http://www.matrixlab-examples.com/

Mohammed Rum, Nor Azizah and Akash, Zainal Abidin (2010). Implementing Life Cyle Costing in Malaysia Construction Industry: A Review. *International Building & Infrastructure Technology Conference.*

Schade, J. (2007). Life Cycle Cost Calculation Models for Buildings. Lulea University of Technology, Lulea, Sweden. Retrieved February 10, 2015.

SYABAS (2011). Water Tariff. Retrieved March 25, 2015. http://www.syabas.com.my/consumer/water-bill-water-tariff

The Free Dictionary (2015). Refurbishment. Retrieved April 23, 2015. http://www.thefreedictionary.com/refurbishment

Tenaga Nasional Berhad (2015). Tariff rates. Kuala Lumpur: Author. Retrieved March 25, 2015. http://www.tnb.com.my/residential/pricing-and-tariff/tariff-rates.html
University of Wisconsin Eau Claire (n.d.). Data collection method. Retrieved March 25, 2015. http://people.uwec.edu/piercech/researchmethods/data%20collection%20methods/data%20collection%20methods.htm
Wan Hassan, Wan Nur Hamizah , Zakaria, Norhanim and Ismail, Muhammad Azzam (2014). The Challenges of Life Cycle Costing Application of Intelligent Building in Malaysia Construction Industry. *Journal Design + Built.* Vol 7.

AN EXPERIMENTAL STUDY ON ASSESSMENT OF STRENGTH OF EPOXY BOND BETWEEN STEEL AND ROUGHENED CONCRETE INTERFACE

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Abstract
At present, adhesive bonding as an alternative to mechanical connectors does not belong to the standard practice in structural applications of the building and construction industry. The objective of this study is to evaluate the effect of surface roughness and adhesion strength between steel and concrete surfaces in two different ways. In this study, push-out tests were conducted to evaluate the interface behavior between steel and concrete connected by bonding using epoxy. Roughness is obtained by creating grooves on the surface of the specimen. The Ultimate push-out loads, slips, load–slip relationships and stress-strain relationships are reported and analyzed for different roughness patterns in this study. It is observed that the roughness pattern significantly effects the load-slip behavior between the steel-concrete interface. It is further concluded from the study that the relationship between the load and slip is nonlinear, and the relative curves of load–slip are ascent segments with no descent part representing a brittle failure. The experimental results show that the interface roughness affects the interface behavior significantly. Further, it can be concluded from this study that the epoxy used in this study can safely used as bonding agent between Steel I-section and concrete deck with Compressive strength less than 50 MPa (as this work is conducted on M50 grade concrete).

Keywords: *Plate bonding; retrofitting; rehabilitation; slip; roughness pattern*

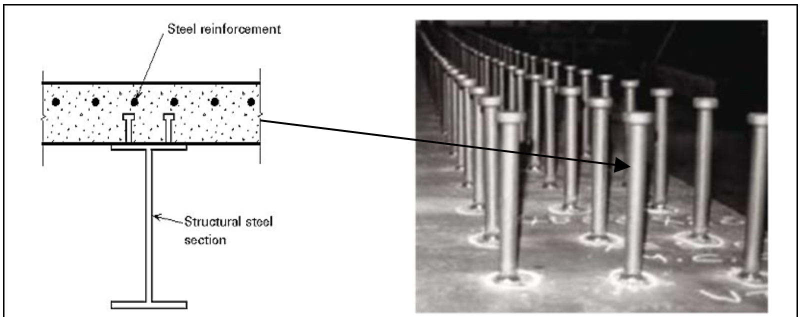
INTRODUCTION

General

The strength recoverey / rehabilitation / retrofitting technique lies in dovetailing structural engineering knowledge with the polymer / cement chemistry to offer innovative solutions to concrete corrosion, repairs, rehabilitation & construction problems. Retrofitting or Rehabilitation of reinforced concrete elements by plate bonding is one of the most popular methods adopted by construction industry. Plate bonding to concrete is also required in composite structures that are being used by Civil Engineers for a long time from now as they perform better than steel or cement acting alone. The performance of plate bonding technique predominantly depends on interfacial bond strength which involves three mechanisms. They are adhesion, mechanical interlocking and friction.

The most common adhesive used for Plate bonding is epoxy resin. Epoxies feature outstanding physical strength properties for high performance bonding, sealing, coating, potting and encapsulation. Epoxies have superior strength properties including cleavage strength, compressive strength, flexural strength, peel strength, shear strength and tensile strength.

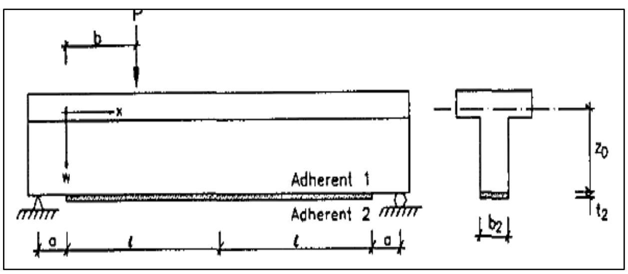
The conventional method of composite beams involves concrete deck/slab connected to steel I section by shear connectors (Figure 1). Soty and Shima (2011) opined that this method induces cracks in concrete and hence reduces durability and fatigue life of structure. Many researchers investigated on connecting steel and concrete using adhesive bonding. Push out test is a method to evaluate the interfacial behavior between steel and concrete connected by bonding (Berthet et al., 2011). Bjorn (1997) studied the problems associated with concrete members strengthened using epoxy-bonded plates (Figure 2). By using linear elastic theory, they derived expressions for shear and peeling stresses, and calculated the critical stress levels at the end of outer reinforcement plate that has been bonded with the beam.



(Source: Peter Helincks et al., 2012)

Figure 1. Cross-section of composite beam at an internal support with shear strudds

The results from shear stress theory, peeling stress theory and finite element analysis show that the magnitudes of the stresses are not only influenced by geometrical and material parameters, but also by adhesive and strengthening material.



(Source: Bjorn, 1997)

Figure 2. Beam strengthened with outer reinforcement bonded with epoxy to its underside, loaded with point load.

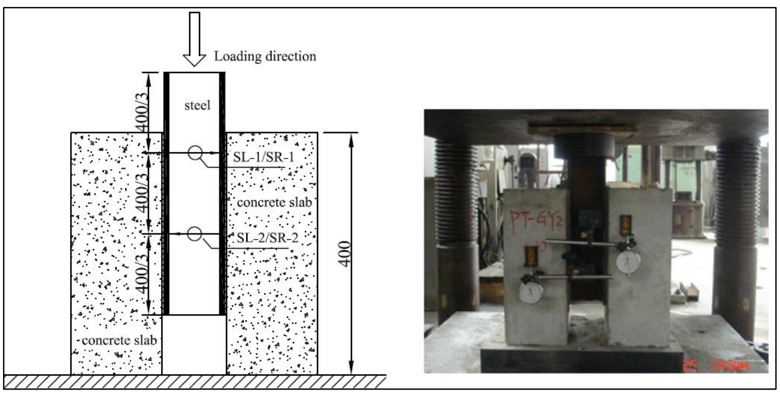
Wolfgang et al. (2008) evaluated the performance of adhesive bonding between steel and concrete. Currently, adhesive bonding as an alternative to mechanical connectors does not belong to the standard practice in structural applications of the building and construction industry. The scope of the study involved investigation of the behavior of adhesively bonded joints between steel and concrete (Figure 3). The main attention was focused on shear stressed; tension stressed and combined stressed connections. The typical characteristics of adhesives in combination with steel and concrete with different high-strength poly-addition curing adhesives, one polyurethane and two epoxy adhesives were examined for different aggregates. It was concluded from the study that cleaning and pre-treatment of the surface prior to bonding is of outmost importance to result in a good adhesive bond. Therefore different pre-treatment for steel and concrete surfaces and their effect on the load behavior and deformation capacity were considered. The concrete surfaces used were grit-blasted and cut with a diamond saw. The steel adherents were grit-blasted and coated with primer prior to bonding.

(Source: Wolfgang et al., 2008)

Figure 3. Shear test specimen with primed steel surface

Yulin Zhan Wu et al. (2016) conducted experiments on twelve sets of specimens with different adhesive thickness, modulus of elasticity and interface roughness (Figure 4). An analytical model is proposed to predict the ultimate load value and load slip relationship. It is concluded from the test results that:

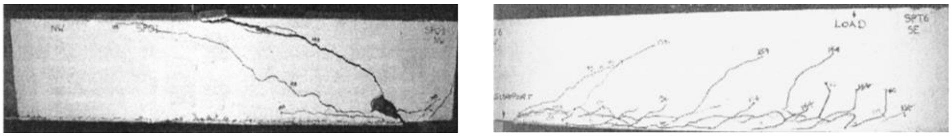
1. The interface capacity increased with increase in adhesive thickness.
2. Interface stress concentration weakening the interface shear resistance may be induced when interface roughness is greater than 0.83 mm.
3. The addition of perpendicular or inclined grooves would reduce the interface strength.



(Source: Yulin Zhan Wu et al., 2016)

Figure 4. Push out test setup

Mohamed Ali et al. (2001) observed that the technique of adhesive bonding of steel to the surfaces of reinforced concrete structural elements is being adopted worldwide to strengthen or repair the reinforced concrete buildings and bridges. One of the major modes of de-bonding in plated RC beams is shear peeling (Figure 5) induced by formation of diagonal cracks that is caused by applied vertical shear forces. Mathematical models are developed specifically for simply supported beams subjected to point loads. It is a simple procedure to adapt as these models can be used for uniformly distributed loads and their load combinations for continuous beams and pre-stressed beams. The test results concluded that shear stirrups have an insignificant effect in controlling de-bonding caused due to shear peeling. This allows this practice of steel plating to be applied with confidence to a wider range of reinforced concrete structures. Stresses are large at end of the plate but they quickly diminish as we move nearer to center of the beam. Magnitude of stresses is influenced by adhesive and strengthening material. Increase in stiffness of adhesive increases shear and peeling stress. Shear and peeling stresses also increase with decrease in thickness of adhesive layer or increase in thickness of plate. If young's modulus of plate increases, shear and peeling stresses also increases.



a) Beam without shear stirrups (Test SP01) b) Beam with shear stirrups (Test SPT6)

(Source: Mohamed Ali et al., 2001)

Figure 5. Shear peeling in tension face plated beams

Jan Klusák et al. (2011) and Peter Helincks et al. (2012) observed that steel-concrete joints can suffer from premature fail due to inadequate shear bond between the two surfaces. In this paper the shear bond strength between steel and self-compacting concrete (SCC) without mechanical shear connectors (Figure 1) is evaluated through push-out tests. The test samples consist of two sandblasted steel plates (10 and 6 mm) and a concrete core, with connection between steel and concrete obtained by a 2-component epoxy resin, gritted with granulates. During the tests, the ultimate shear force is recorded as well as the slip between steel and concrete. All test members exhibited a concrete - adhesive failure, and indicate nominal shear bond stresses between 2.20 and 4.22 MPa. In addition, a substantial difference in measured shear bond stresses is found between the 6 and 10 mm steel plates, indicating unwanted secondary effects with the 6 mm plates. During testing, maximum slip values between 0.02 and 0.05 mm are recorded. In addition to the experimental tests, shear stress distribution in the epoxy-concrete interface is examined by finite element analysis (FEA). In this way, a non-uniform stress distribution between steel and concrete is found with the maximum shear value about 2.5 times higher than the nominal shear stress value. The FEA combined with the experimental results provide a reasonable understanding of the shear induced failure conditions at a steel-concrete joint, and create test data for a fracture mechanics approach.

Hosseini and Ahmad (2012) studied the effects of spiral diameter on the bond stress-slip relationship in grouted sleeve connector. 21 grouted splice connectors joining two main steel bars were tested until failure under gradually increasing axial load. The objective of this study is to investigate bond strength, slip and failure modes of the connected steel bars confined by the grouted sleeve connectors. Each sleeve connector consisted of steel pipe, steel spiral with vertical bars and cement grout. The key parameter was the diameter of the spiral reinforcement which confined the splicing of two steel reinforcement bars. The effects of spiral diameter of 35, 45, 55, 65, 75 and 85 mm on the bond strength and slip were critically investigated. The test results indicate that specimens with smaller spiral reinforcement accompanied by four shear keys provide greater bond strength. The improvement in bond strength is due to the confinement provided by the spiral reinforcement which eventually increases the axial tension capacity of the sleeve connector. Amar Prakash et al. (2012), Bouchair et al. (2013), Aida Mazoz et al. (2013) and many others researched on various other applications of Push out test.

Methods of Evaluating Steel-Concrete Bond Strength as Per Literature

In the Theory of pure bending of beams, a beam is considered to be made up of thin layers, one over the other. The bending stress is maximum at the bottom most layer of the beam and hence to retrofit beams, generally steel plates are bonded to the bottom of beam. This is called "Bonding plates to the bottom fiber".

The bonding of steel plates to concrete members has been undertaken by the several methods, the choice of method being dependent upon the particular circumstances. Three methods are given below:

1. Single plates of the required thickness are taken. A paste consistency resin is applied to the plate which is pressed into position against the soffit by wedging off a temporary stiff girder. The stiffness of these plates will not allow them to follow the concrete profile closely and variations in adhesive thickness will be apparent. A minimum adhesive thickness is maintained by the use of plastic spacers. Bonding can proceed quickly, although the quality of adhesive used is increased.
2. Multiple plates bonded in layers to give the required total thickness. Adhesive is applied to the plates and is offered to the soffit as described above. The advantages are that thinner, more flexible plates can be used which follow the profile of the existing concrete surfaces and a thin, constant thickness glue line is maintained. The disadvantage is a longer construction period.
3. Single plate of required thickness is placed on concrete and the gaps sealed at the edges between them. Resin is then pumped ensuring that no voids occur between the plate and concrete. This method is economical and quick but difficulties may arise over maintaining the standard of surface preparation of steel due to greater time lapse between steel preparation and resin injection.
4. The standard test method for Tensile strength of concrete repair and overlay materials by direct Tension (Pull-off Method) is being currently adopted as per ASTM C 1583 / C1583M-13. The test specimen is formed by drilling a shallow core into and perpendicular to the surface of the substrate, and leaving the intact core attached to the concrete. A steel disk is bonded to the top surface of the test specimen. A tensile load is applied to the steel disk until failure occurs. The failure load and the failure mode are recorded and the nominal tensile stress at failure is calculated. The test setup is presented in Figure 6.

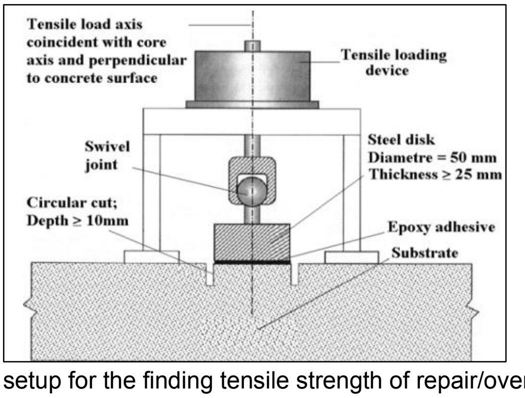


Figure 6. Test setup for the finding tensile strength of repair/overlay materials

Concluding Remarks from Literature

It can be observed from the literature that plate bonding is being widely used as a rehabilitation and retrofitting technique for enhancing flexure and shear capacity of beams. Further, it is also observed that connection of composite sections using shear connectors results in diagonal cracks. Hence there is a need to evaluate the performance of epoxy bonded concrete deck to steel I-section in the place of a shear connector.

The only method to evaluate the strength of the steel-concrete epoxy bond is by Push-Out test. Push out test is preferred instead of Push out test because while conducting Push out test, UTM displays both slip and elongation of the material of the specimen connected to the UTM. Further, perfect axial pull cannot be ensured unless the rod is connected exactly to the specimen at its center of gravity. However, the method involved cutting of steel I-section to a required size which is tedious and costly. This study aims at evaluating the feasibility of considering steel plates bonded by epoxy resin to concrete cubes for evaluating the bond strength as an alternative to concrete plates bonded to steel I-section using epoxy resin.

OBJECTIVES AND SCOPE OF STUDY

Objectives of The Study

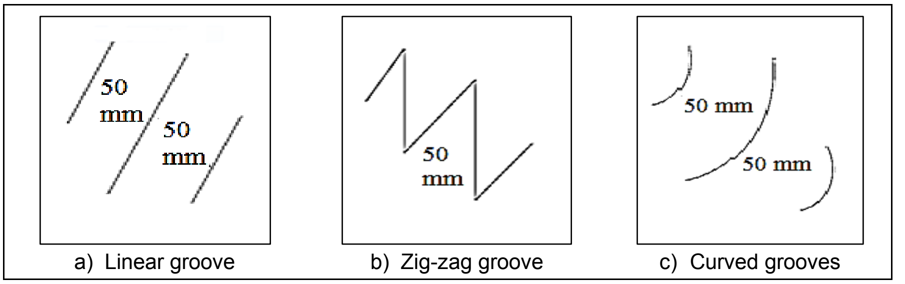
The basic objectives of this experimental study are:

1. To evaluate epoxy bond strength of steel and concrete surface in two different ways (i.e. steel plates bonded to concrete cubes and concrete plates bonded to steel I-section) by Push-Out test.
2. To compare the performance of steel-concrete epoxy bond for different roughness patterns (linear, zig-zag and curved).

Scope of The Study

Total 2 sets of four specimens in each set are tested with one set evaluating epoxy bond strength of the concrete plates bonded to steel I-section and the other set evaluating steel plates bonded to concrete cube.

Among the four samples in each set, the first sample is without any roughness pattern. The remaining three samples are prepared with different roughness patterns like linear, zig-zag and curved as shown in Figure 7 (a), Figure 7 (b) and Figure 7 (c) respectively. The roughness pattern is made over an area of 150 mm × 150 mm on concrete plates and over the entire concrete cube area which is 150 mm × 150 mm. Groves are made to achive these roughness patterns. A constant grove depth of 3mm with lengths of 140mm and 80mm were adopted for all the groves.



a) Linear groove b) Zig-zag groove c) Curved grooves

Figure 7. Roughness patterns on concrete cube surface and concrete plate surface

The notation adopted for the eight samples tested in this study is as follows.

SET 1
F – Steel I section attached to concrete plates having no grooves.
F1 – Steel I section attached to concrete plates having linear grooves.
F2 – Steel I section attached to concrete plates having curved grooves.
F3 – Steel I section attached to concrete plates having zig-zag grooves.

SET 2
C – Concrete cubes with no grooves attached to steel plates.
C1 – Concrete cubes with linear grooves attached to steel plates.
C2 – Concrete cubes with curved grooves attached to steel plates.
C3 – Concrete cubes with zig-zag grooves attached to steel plates.

EXPERIMENTAL PROGRAM

Properties of Materials Adopted in This Study

Properties of Ingredients of Concrete

Cement

Ordinary Portland cement of 53 grades conforming to IS 12269 (1987) was used for this experimental investigation. Various properties of cement were determined as per IS 4031 (1988). The results are presented in Table 1.

Table 1. Properties of cement

Property	Test result	Requirements as per IS: 12269 1987
Fineness	4 %	Should be < 10 %
Normal consistency	33 %	
Soundness	4 mm	Should be < 10 mm
Initial Setting Time	40 min	Should be > 30 min
Final Setting Time	230 min	Should be < 600 min
Specific gravity	3.15	
3 days Compressive Strength	28.5 MPa	Should be > 27 MPa
7 days Compressive Strength	37.4 MPa	Should be > 37 MPa
28 days Compressive Strength	54.1 MPa	Should be > 53 MPa

Fine Aggregate

Natural River sand conforming to IS 383 (1970) was used. The results of various tests on fine aggregate are given in Table 2.

Table 2. Properties of fine aggregate

Sl. No	Property	Value
1	Specific Gravity	2.71
2	Fineness Modulus	2.64
3	Grading	Zone II

Coarse Aggregate

Crushed granite coarse aggregate conforming to IS 383 (1970) was used. The results of various tests on coarse aggregate are given in Table 3.

Table 3. Properties of coarse aggregate

Sl. No	Property	Value
1	Specific Gravity	2.70
2	Fineness Modulus	7.20
3	Maximum Nominal Size	10 mm

Potable Water

Ordinary potable tap water available in laboratory was used for mixing and curing of reference concrete. It had a pH value of 7.1.

Mix Proportions

M55 is the highest grade that can be designed by IS 10262 (2009) and hence based on the properties of ingredients they can be proportioned to obtain the maximum strength. Table 4 presents the mix proportions for M50 grade concrete. The maximum cement content is restricted to 450 kg/m^3 of concrete. The mix proportions that have arrived after after trials are presented in Table 4 below.

Table 4. Mix proportions for mix designs

W/C Ratio	Cement Content (kg/m^3)	Fine Aggregate (kg/m^3)	Coarse Aggregate (kg/m^3)	Water (lit)	Mix Proportion
0.40	450	987.21	840.83	180	1:2.19:1.86

Preparation of Specimens

The ingredients for various mixes were weighed; required water was added and mixed by using a tilting drum type concrete mixture machine. Precautions were taken to ensure uniform mixing of ingredients. The specimens were cast in steel moulds and compacted on a table vibrator. The specimens of 100 mm × 100 mm × 100 mm size of cubes for compression, 500 mm × 100 mm × 100 mm size of prisms for flexural and 300 mm (L) × 150 mm (d) size of cylinders for split tensile strength and Modulus of Elasticity were cast as per Indian standard IS 516 (1959). The sizes were of various specimens were adopted as per IS: 516 1959 as they are sizes to be used for coarse aggregate size of upto 10 mm for the determination of compressive strength and durability characteristics. Specimens were de-moulded in 20 hours and cured as the top surface of the concrete in the moulds was hard enough by Spreading wet gunny bags over the moulds for 20 hours after the casting was carried out for the initial curing to avoid thermal cracks.

Properties of Fresh and Hardened Concrete

Workability and Tests on Workability of Fresh Concrete

The workability of a fresh concrete mix can be defined as the relative ease with which concrete can be mixed, transported, moulded and compacted. The workability of a fresh concrete mix can be experimentally determined by three methods as per IS 1199 (1959) and the test results are as presented in Table 6.

Table 6. Workability test results

Slump	Compaction Factor	Vee-Bee Test
20 mm	0.73	4 sec (3 cm)
Low	Medium	Medium

Mechanical Properties of Concrete

Compressive Strength

Concrete specimen cubes are used to determine compressive strength of concrete and were tested as per IS: 516 1959 in Compression Testing Machine, of 3000kN capacity. The load was applied at a rate of 140 kg/cm^2/min until the resistance of the specimen to the increasing load can be sustained. Based on a number of trail mixes, the maximum 7 days compressive strength is observed to be 42 MPa and the maximum 28 days compressive strength of 60.80 MPa was obtained and this mix is used for further study in this project.

Tensile Strength of Concrete

In experimental research, it is important to determine the properties of materials used to establish their suitability as per the respective codes of practice. In this study, to understand the failure behavior of steel-concrete interface (when bonded with epoxy), it is necessary to study the properties of steel and concrete. However, failure at interface can actually occur at interface of either of the two materials: concrete-epoxy interface or steel-epoxy interface. As steel is relatively very stronger than concrete and epoxy, the other mechanical properties of concrete are studied. As per IS 456 (2000), the tensile strength of Concrete should be $0.7\sqrt{f_{ck}}$ N/mm^2, where f_{ck} is characteristic compressive strength of concrete. Due to difficulty in applying uniaxial tension to a concrete specimen, the tensile strength of the concrete is determined by indirect test methods IS 516 (1959): Split tensile strength test (Figure 8) and Flexural Tensile strength Test (Figure 9). Concrete specimen cylinders (150 mm diameter, 300 mm height) are used to determine split tensile strength of concrete and Concrete prism specimens (700 mm × 150 mm × 150 mm) are used to determine flexural strength of concrete. The load was applied without shock and increased continuously at a nominal rate within the range 1.2 N/mm^2/min to 2.4 N/mm^2/min until failure.

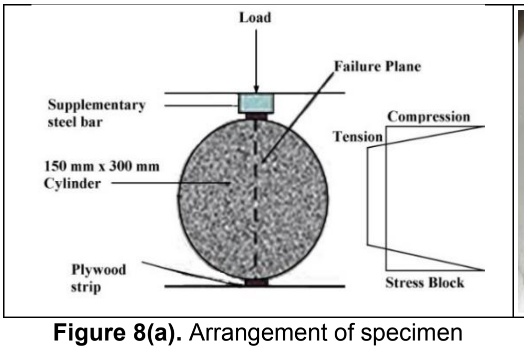


Figure 8(a). Arrangement of specimen

Figure 8(b). Test for split tensile strength

Figure 9. Flexural tensile strength test in Universal Testing Machine

The split tensile strength and flexural strength of concrete at curing periods of 7 days and 28 days are presented in Table 7.

Table 7. Tensile strength test results of concrete

Age	7 days	28 days
Split Tensile Strength (N/mm^2)	5.28	7.77
Flexural Strength (N/mm^2)	6.84	10.39

Materials for Making Specimens for Push-Out test

Steel Plates (Figure 10(a))

The use of high yield steel does not generally provide benefits as the modulus of elasticity is the same as mild steel. Stainless steels need not be used as there is little published information on the effect of its composition on the adhesive bond strength.

Concrete Suitability

A range of tests are available for determining the suitability of the concrete surface for bonding plates.The most suitable test method involves bonding a metal dolly to the concrete and subsequently pulling it off. Examination of the failure mechanism and failure load gives useful guidance. In this study, M50 grade concrete was adopted for casting concrete cubes and concrete plates and the plates were provided with HYSD reinforcement. In the concrete plate, 5 mm diameter steel Reinforcement with spacing of 55 mm is provided (Figure 10(b)).

Adhesive (Figure 10(c))

Epoxyresin adhesives have been found to be suitable for steel plate bonding. Their durability has been established by use over a period of fifteen years. Epoxy resin adhesives are therefore acceptable for strengthening proposals provided that they comply with the specification. This specification contains provisions for the acceptance of alternative equivalent adhesives.

Epoxy resin adhesives require care in use. Manufacturers or formulators commonly supply two part resins in containers suitably proportioned for mixing.It is important that all the hardener is added to the resin in its container and mixed with a slow speed mechanical mixer. High speed mixing entrains air and is less efficient.The resin and hardener should be of different colours and a uniform colour indicates adequate mixing.The speed of the chemical reaction increases with the temperature generated.

a) Steel Plates | b) Steel Reinforcement inside Concrete Plate | c) Hardener and Epoxy resin

Figure 10. Materials used in Push-Out test

Material Surface Preparation

Concrete Surface

The concrete surface of an existing member will usually be contaminated and have out-of-plane imperfections and will therefore require preparation before plates are bonded to it. Grit blasting is a preferred method of removing weak material, laitance and surface contamination. Scabbling and grinding could damage concrete and should only be used to remove minor protuberances. Cracks wider than 0.2mm which could allow loss of adhesive, and areas of concrete that appear porous should be sealed with a compatible resin.The prepared surface should be dustfree and surface dry.If moisture is picked up by absorbent paper pressed onto the concrete it is likely to be too damp for bonding. A heated enclosure may then be necessary.

Steel Surface

The surface of the steel to be bonded must be completely free of any mill-scale, rust, grease or other contaminants. For successful adhesion of the resin the contact surfaces of the steel plates should be degreased and blast cleaned The cleaning is done by grinding as shown in Figure 11. All surface dust on the plates should be removed by vacuuming immediately before application of the primer. The primer, for the epoxy resin adhesive, should be an epoxy based system which is compatible with the adhesive. It should be applied to the surfaces to be bonded at a dry film thickness not exceeding 50 microns within 4 hours of grit blasting and allowed to cure for the time specified by the manufacturer at the appropriate temperature.The primer for the alternative adhesive should be compatible with the adhesive and should be applied in accordance with the manufacturer's instruction. Subsequent handling should be undertaken by operatives wearing clean gloves. The plates should be wrapped in clean protection material in the presence of adesiccant and stored in clean dry conditions.

Figure 11. Cleaning surface of steel plate

Push-Out test procedure for determination of Load-Slip behavior and shear strength of epoxy Pushout test procedure

There is no procedure mentioned in codes of practice of any of countries for Push-out test. Based on the review of literature, the following procedure has been formulated.

Procedure for I-Section Bonded to Two Concrete Plates

STEP 1: Initially, steel I-section and two concrete plates are taken as per dimensions shown in Figure 12(a) and Figure 16(b) respectively. The dimensions are measured using a Vernier calipers of Least Count 0.02 mm.

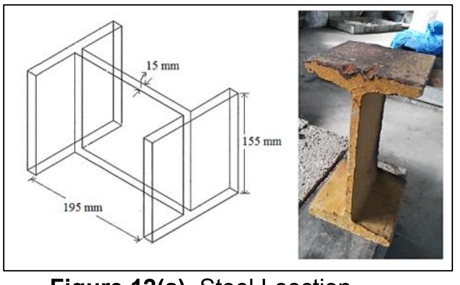


Figure 12(a). Steel I-section

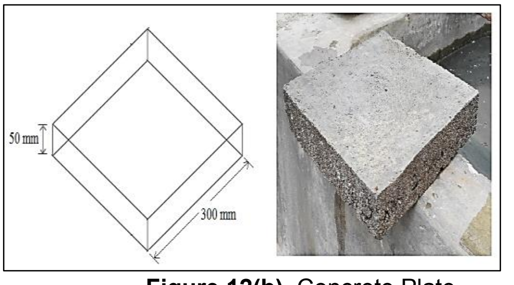


Figure 12(b). Concrete Plate

STEP 2: Make the markings on the concrete plate for bonding the flange of the steel I-section to ensure that the contact area is at the center of both surfaces.

STEP 3: Epoxy resin has to be mixed as per manufacturers specifications and applied to the concrete plate (Figure 13). The thickness of epoxy applied was in the range of 1 mm to 1.5 mm.

Figure 13. Mixing and Application of Epoxy to I-section

STEP 4: After setting of epoxy (in a couple of hours), whole specimen is placed on the UTM (Figure 14). Care should be taken to ensure that it is fixed around using clamps to avoid sudden spalling of specimen.

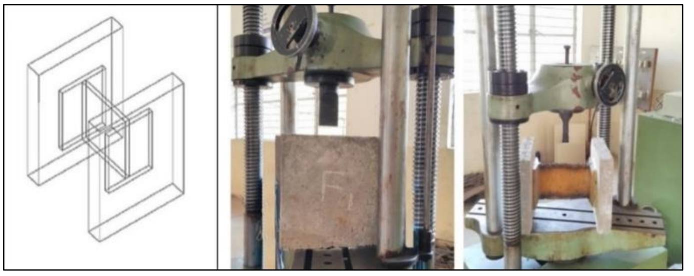


Figure 14. Arrangement of specimen in UTM and application of load on the specimen

STEP 5: Apply load on the I-section till the bond fails and note down the values of load and deflection at regular intervals. The rate of loading was maintained at around 4 mm/min. The typical experimental set up for this case is shown in Figure 15 (a) and the display of Load and Slip variation in Computer is shown in Figure 15 (b).

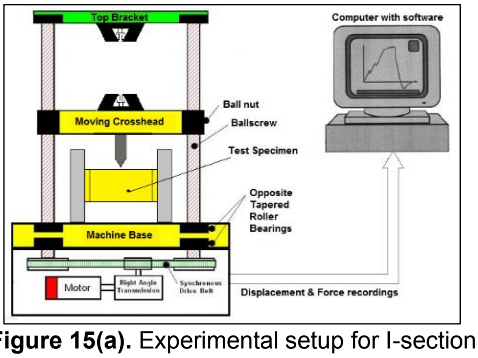


Figure 15(a). Experimental setup for I-section bonded to Concrete plates

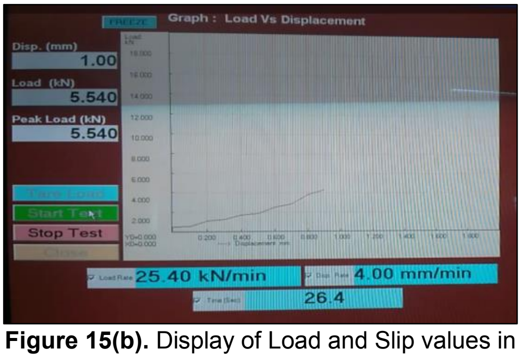


Figure 15(b). Display of Load and Slip values in the system

Procedure for Concrete Cube Bonded to Two Steel Plates

STEP 1: Two steel plates and a concrete cubes are taken as per dimensions shown in Figure 16(a) and Figure 16(b).

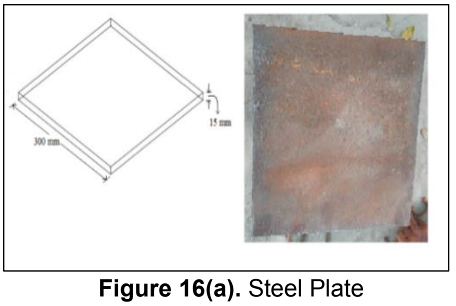


Figure 16(a). Steel Plate

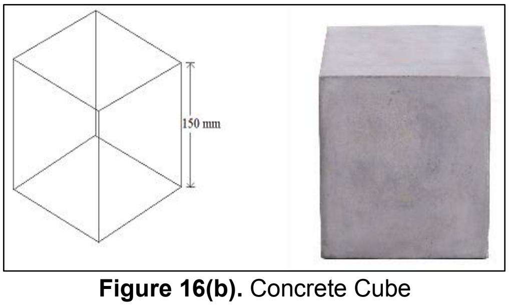


Figure 16(b). Concrete Cube

STEP 2: Mark the contact area of the concrete cube on the steel plate to ensure that the contact area is at the center of both the surfaces.

STEP 3: Epoxy resin is mixed as per manufacturers specifications and applied to the concrete and bonded to the steel plates. The thickness of epoxy applied was in the range of 1 mm to 1.5 mm (Figure 17).

Figure 17. Mixing and application of Epoxy to Concrete Cube

STEP 4: After setting, the specimen is placed on the UTM (Figure 18). It is made sure that it is fixed around using clamps to avoid sudden sapling of specimen.

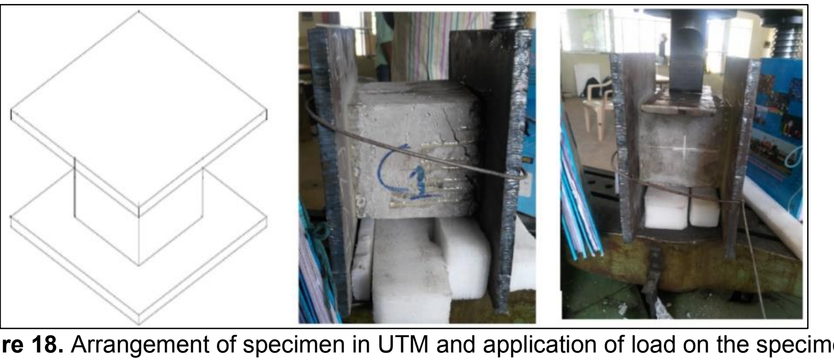


Figure 18. Arrangement of specimen in UTM and application of load on the specimen

STEP 5: Apply load on the concrete cube till the bond fails and note down the values of load and deflection. The rate of loading was maintained at around 4 mm/min. The typical specimen arrangement for this case is shown in Figure 19 and the display of Load and Slip variation in Computer is similar to that shown in Figure 15 (b).

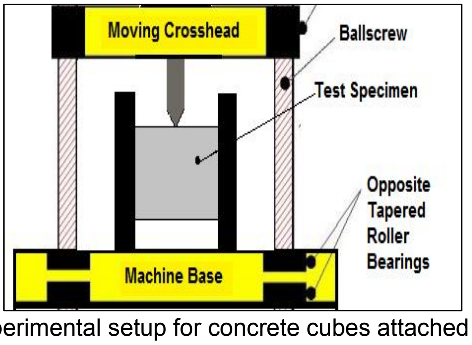


Figure 19. Experimental setup for concrete cubes attached to steel plates

The shear capacity and the load-slip relations are the most important characteristics for the design of the composite structure. The most appropriate test method to find them is a full-scale test. However, it can highly expensive and time-consuming. Therefore, the push-out test are an economical alternaive. Although ASTM C 1583 / C1583M-13 presents the test method to evaluates the adhesive strength of bonding agents for application as a repair, rehabilitation or retrofitting material, it can only give the tensile strength. It cannot determine the load-slip behavior of the adhesive – substrate interface like Push-out test.

RESULTS AND DISCUSSIONS

General

After testing the specimens, the thickness of the sheared layer is measured using Vernier Callipers (of Least count 0.02 mm). It was observed that the thickness of the sheared area varied across the area for all 8 specimens differently. However, the minimum thickness is observed to be in the range of 2 mm to 3 mm.

The stress is calculated by considering the contact area as

A = 2 × (155 mm × 150 mm) = 46,500 mm^2. (for Set 1 specimens)
A = 2 × (150 mm × 150 mm) = 45,000 mm^2. (for Set 2 specimens)

In all the eight specimens, it is observed that the failure is not at the epoxy-concrete interface but at the layer in concrete beyond the interface. Hence, all the push out loads and corresponding bond strengths calculated are corresponding to concrete shear strength and not the shear strength of the epoxy. Figure 20 and Figure 21 show the failure surface for a typical Set 1 and Set 2 specimens where the exposed surface is exclusively concrete and no traces of epoxy are visible.

Figure 20. A typical Set 1 specimen after failure

Figure 21. A typical Set 2 specimen after failure

The load vs. Slip (obtained from the Push-out test) and summary of Peak pus out load Bond shear strength for all the eight specimens are presented in this section. Although, bond slip is measured, it was not included in formulating inferences, discussions and conclusions as it was observed that the slip measured included the indentation on the specimen due to loading that cannot be measured. The Load vs. Slip for Set 1 and Set 2 specimens are presented in Figure 22 and Figure 23 respectively.

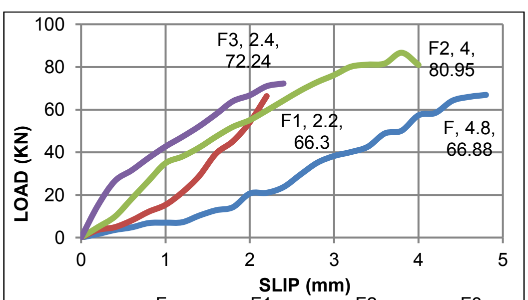


Figure 22. Load vs. Slip for Set 1 specimens

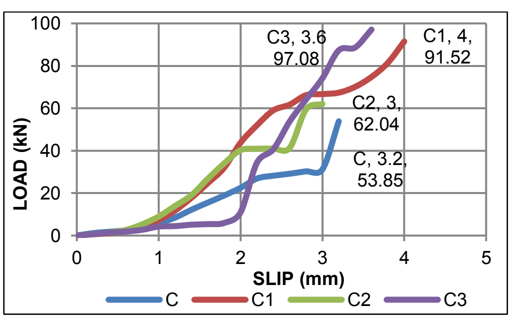


Figure 23. Load vs. Slip for Set 2 specimens

Table 8. Summary of Push-Out test results

SPECIMEN ID	PEAK LOAD (kN)	Bond Shear Stress (MPa)
F	66.88	1.438
C	53.85	1.197
F1	66.3	1.426
C1	91.52	2.034
F2	80.95	1.741
C2	62.04	1.379
F3	72.24	1.554
C3	97.08	2.157

The following are observed from Load vs. Slip behavior of Set 1 and Set 2 specimens and results presented in Figure 22, Figure 23 and Table 8.

1. From the results of samples with plane interface, it can be observed that the Ultimate push out load in C is 19.48 % less than that of F.
2. From the results of samples with linear groove interface, it can be observed that the Ultimate push out load in C1 is 38% greater than that of F1.
3. From the results of samples with curved groove interface, it can be observed that the Ultimate push out load in C2 is 23.36 % less than that of F2.
4. From the results of samples with Zig-Zag interface, it can be observed that the Ultimate Push-Out load in C3 is 34.38 % greater than that of F3.
5. Comparing F, F1, F2 and F3 for the four specimens of Set 1. The Ultimate Push-Out load is observed to be maximum for F2, followed by F3, F and F1. The Push-Out loads for F3, F and F1 are observed to be 10.75%, 17.38% and 18.09% less than that of F2 respectively. However, the difference between Push-Out load of F1 and F is insignificant and hence both can be considered to be same.

CONCLUSIONS

An experimental study was conducted to compare the interface behavior between steel and concrete connected by epoxy bonding in two different ways. Two sets of 4 specimens in each set were tested for assessment of the effect of roughness pattern (linear, curved and zig-zag) on the bond strength of the epoxy. One set of specimens were tested for bond between Steel I-section with concrete plates bonded to it on both the flanges and the other with Concrete cube to which two steel plates are bonded on opposite sides.

The following conclusions can be drawn from the study conducted within the scope of the study.

1. In all the specimens, a common failure mode was observed where the concrete surface just beyond the epoxy surface was sheared off. The compressive strength of concrete at the age of conduct of Push-Out test is 60.80 MPa. Hence, it can be concluded from this observation that the bond strength of epoxy is greater than the shear strength of concrete whose compressive strength is 60.80 MPa. Exact strength can be determined only when the shear strength of concrete is greater than bond strength of epoxy.
2. The Ultimate Push-Out loads are reported and analyzed for different roughness patterns in this study. It is observed that the roughness pattern significantly effects the failure mode and behavior between the steel-concrete interface.

3. Comparing F, F1, F2 and F3 for the four specimens of Set 1, The Ultimate Push-Out load is observed to be maximum for F2, followed by F3, F and F1. Further, the Push-Out loads for F3, F and F1 are observed to be 10.75%, 17.38% and 18.09% less than that of F2 respectively. However, the difference between Push-Out load of F1 and F is insignificant and hence both can be considered to be same.

In conclusion, it is observed that the failure of epoxy bond at the steel-concrete interface is very sudden and hence can be considered as brittle failure. In other words, all adhesive shear connections studied in this project exhibited non-ductile performance, most likely because of the characteristic of the bonding material i.e., epoxy. The experimental results show that the interface roughness affects the interface behavior significantly. Higher grades of concrete are required to assess the bond strength of epoxy adopted in this study. However, the epoxy used in this study can safely be used for Rehabilitation and Retrofitting of structural elements with Compressive strength of concrete less than 50 MPa. It can also be used as bonding agent between Steel I-section and concrete deck with Compressive strength less than 50 MPa.

ACKNOWLEDGEMENTS

I thank "V Ramya Nikitha Reddy, P Bharath Kumar Dora, T Sai Charan and V Pooja" my Undergraduate students of Civil Engineering at MVGR College of Engineering (A), Vizianagaram, India who conducted this experimental work as a part of their B.Tech Final year project.

REFERENCES

A. Bouchair, J. Bujnak, P. Duratna and A. Lachal (2012), "*Modeling of the steel-concrete push-out test*", procedings of the International Conference on Steel Structures and Bridges – 2012 published in Procedia Engineering, Volume 40, pp. 102 – 107.

Aida Mazoz, Abdelkader Benanane, and Messaoud Titoum (2013), *"Push-out Tests on a New Shear Connector of I-shape"*, International Journal of Steel Structures, Volume 13, Issue 3, pp. 519-528.

Amar Prakash, N. Anandavalli, C. K. Madheswaran and N. Lakshmanan (2012), "*Modified Push-out Tests for Determining Shear Strength and Stiffness of HSS Stud Connector - Experimental Study"*, International Journal of Composite Materials, Volume 2, Issue 3, pp. 22-31.

Berthet, J. F., Yurtdas, I., Delmas Y., and Li, A. (2011), *"Evaluation of the adhesion resistance between steel and concrete by push out tests."*, International Journal of Adhesion and Adhesives, Volume 31, Issue 2, pp. 75–83.

Bjorn Taljsten (1997), "*Strengthening of beams by plate bonding*", Journal of Materials in Civil Engineering, Volume 9, Issue 4, pp. 206-212.

C1583/C1583M – 13. (2013), "*Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)"*, American Soceity for Testing and Materials International.

Hosseini, S. J. A., and Ahmad, A. B. (2012), *"Effects of spiral diameter on the bond stress-slip relationship in grouted sleeve connector"*, Malaysian Construction Research Journal, Volume 12, Issue 1, pp. 1-19.

IS 10262 (2009), *"Indian Standard Code of practice for "Concrete Mix Proportioning"*, Bureau of Indian Standards, New Delhi.

IS 1199 (1959) (Reaffirmed 2004), *"Indian Standard "Methods of Sampling and Analysis of Concrete"*, Bureau of Indian Standards, New Delhi.

IS 12269 (1987), *"Indian Standard Code of practice for "Specification for 53 grade Ordinary Portland Cement"*, Bureau of Indian Standards, New Delhi.

IS 383 (1970), *"Indian Standard Code of practice for "Specification for Coarse and Fine Aggregates from natural sources for concrete"*, Bureau of Indian Standards, New Delhi.

IS 4031 (1988), *"Indian Standard Code of practice for "Methods of physical tests for hydraulic cement"*, Bureau of Indian Standards, New Delhi.

IS 456 (2000), *"Indian Standard: Plain and Reinforced Concrete - Code of practice"*, Bureau of Indian Standards, New Delhi.

IS 516 (1959), *"Indian Standard Code of practice for Methods of tests for strength of concrete"*, Bureau of Indian Standards, New Delhi.

IS 5816 (1999), *"Indian Standard Code of practice for "Splitting Tensile strength of Concrete- Method of test"*, Bureau of Indian Standards, New Delhi.

Jan Klusák, Stanislav Seitl, Wouter De Corte, Peter Helincks, Veerle Boel and Geert De Schutter. (2011), *"Failure conditions from push out tests of a steel-concrete joint: fracture mechanics approach"*, Key Engineering Materials, Volumes. 488-489, pp. 710-713.

M. S. Mohamed Ali,Deric J. Oehlers and Mark A. Bradford (2001), "*Shear peeling of steel plates bonded to tension faces of RC beams*", ASCE Journal of Structural Engineering, Volume 127, Issue 12, pp. 1453-1459.

Peter Helincks, Wouter De Corte, Jan Klusák, Stanislav Seitl, Veerle Boel and Geert De Schutter (2012), *"Failure conditions from push-out tests of a steel-concrete joint: Experimental results",* Key Engineering Materials, Trans Tech Publications, Switzerland, Volumes 488 – 489, pp. 714-717.

Soty, R., and Shima, H. (2011). *"Formulation for maximum shear force on L-shape shear connector subjected to strut compressive force at splitting crack occurrence in steel–concrete composite structures.",* Procedia Engineering, Twelfth East Asia-Pacific Conference on Structural Engineering and Construction, City Univ., Hong Kong, Volume 14, pp. 2420 – 2480.

Wolfgang Kurz, Christopher Kessler, Paul Ludwig Geiss and Sarunas Turcinskas (2011), "*Evaluation of Adhesive bonding between Steel and Concrete*", Proceedings of the International Conference on Composite Construction in Steel and Concrete, February 2008, pp. 1-12.

Yu-Fei Wu, Liang He and L. C. Bank (2016), "*Bond-Test Protocol for Plate to Concrete interface involving all mechanisms*", ASCE Journal of Composites for Construction, Volume 20, Issue 1, pp. 04015022-1 -15.

Yulin Zhan, Zhongguo John Ma, Renda Zhao, Guifeng Li and Tianyu Xiang (2016), "*Interface behavior between Steel and Concrete Connected by Bonding*", ASCE Journal of Bridge Engineering, Volume 21, Issue 6, 04016026-1-10.

DESIGN FACTORS AFFECTING FACILITIES MANAGEMENT PRACTICE OF BUILDINGS IN MALAYSIA

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Abstract

The prevalence of design effects and their resultant cumulative negative impact on the Facilities Management (FM) practices and the financial performance of building projects are a recurrent challenge in the construction industry. Effective integration of FM at design phase is essential to minimize design conflicts and the cost of building facilities services. In addressing the need to reduce the effect of design factors, the objective of this paper is to identify and assess the major factors affecting FM practice in building projects. Besides, the role of facility managers was explored. Following a rigorous literature review, in total, thirty-eight design factors were identified under three categories: architectural, structural and Mechanical-Electrical-Plumbing (MEP) design. Thereafter, a questionnaire was designed by utilizing a five-point Likert scale of importance. The mean and relative importance index (RII) of the factors were determined, followed by rank analysis. The survey was evaluated by the G7 FM organisation of Construction Industry Development Board (CIDB) and corporate member of Malaysian Association of Facilities Management (MAFM). The most important factors from the perspectives of FM practice were “incomplete working drawing and specification”, “inadequate working drawing details”, “non-availability of specific building materials in market”, “inappropriate selection and specification of materials” and “plaster crack at joint i.e. concrete-brick joints and wall-floor joints”. This paper findings are practically significant to all designers and facilities managers, as it will prompt the management of building facilities to concentrate on the most important design factors affecting the FM practices and thereby mitigate that cost.

Keywords: *Design factors; Facilities Management; buildings; Malaysia*

INTRODUCTION

Over the last two decades, the built environment has witnessed great development and progressive transformation in terms of use of innovative building materials and modern construction technologies (Hassanain et al., 2017). Towering sky-scrapers in hundreds of meters and over hundreds of floors now elegance many city centres all over the world. These modern structures accommodate various purposes, including residential, commercial office and entertainment. Consequently, this aspect requires complex and sophisticated support systems to fulfil the needs of the modern building facilities and intended end-users. Several studies have revealed the effective coordination with the professionals of support service in buildings is one of the grey areas affecting the overall success and performance of the building projects (Korman et al., 2010; Mohammad and Hassanain, 2010). In another study Lee (2014) reported that construction projects have a great witness of delays in projects schedule and wastage of financial and material resources due to coordination gap between traditional design process and construction process. Furthermore, wastage of money may vary depending on the volume and complexity of the project. A number of authors have reported a high percentage of design defects during the construction phase initiates from the decisions made at the time of development phases (Alaloul et al., 2016; Love et al., 2014). This phase involves multiple design specialist to constitute a complex design-subsector according to the project requirement. This is required to process the interdependency managements among all design professionals’ activities (Wan and Kumaraswamy, 2012).

Defects appear in buildings through inappropriate design specification and construction, thus impacting FM practice. Rahman and Salim (2013) indicated, there are three primary issues affecting FM practice: deficient budgetary, incompatible management and poor building detailing and design. Presently, for a project success most crucial factor is design defects perceived by both owners and contractors and have a tremendous impact on later expenditures like at post occupancy stage (Love et al., 2014). Wong and Chan (2014) concluded 58% of building defects originated from faulty design. Love et al. (2014) added that these defects can also incur more cost that adds in facility maintenance around 14.2%. Josephson and Hammarlund (1999) investigated many building projects to sort out the origin and causes of building defect and concluded that design is the main source and others client, materials, site management. Therefore, FM is continuously facing operability problem at post-occupancy phase. Jensen (2009) and Au-Yong et al. (2017) suggested that the common design defects and operational issues at post-occupancy phase could be resolved by integrating facility manager's participation during the early design phase.

With the same point of view, several researchers concluded that effective integration of FM in design process could manage all the efforts related to planning, design and managing buildings as well as their sophisticated systems, equipment or facilities in a rapidly changing world (Mohammad and Hassanain, 2010; Taleb et al., 2017; Islam et al., 2017). Similarly, the study by Riley et al. (2005) and Mohamed and Anumba (2006) indicated that without integration this can lead to field conflicts or clashes between various working drawings as they are performing work simultaneously. This high challenging field conflicts can lead to interfere all types of construction work progress associated with installation, demolition, replacement/rework at post-construction phase. It has been demonstrated that a high intake of design changes or defects increase the non-value-added demolition and repetitive work that ultimately consequence in material wastes (Wan and Kumaraswamy, 2012).

The perspective discussed so far indicates to a strong evidence that effective integration of FM is much needed at the early-design development stage of a building construction project. Proper integration can minimize the uncertainties and avoid downstream field conflicts. Clarification on design changes and related issues can be resolved before producing the working drawing for construction process (Riley et al., 2005).

Traditionally, excluding FM from the design process prevents opportunities to inject practical considerations which could improve design based on facility manager's vast knowledge and experience. Indeed, early involvement of FM specialist is also advantageous for providing a cost-effective building design solution (Love et al., 2014). On the other hand, establishing early-design factors can preclude probable premature replacement and maintenance work arising from design decision change or defect (Hassanain et al., 2017).

Limited number of studies have been conducted to investigate potential design factors affecting FM practice in the building construction industry. In construction industry, design development phase is iterative and evolutionary. Design phase involves various design disciplines exchanging their information and knowledge starting from briefing stage and ending with detailed design stage. Hence, this multi-disciplinary building design services lead to a highly challenging facility management process. There are several design factors that influence the FM practice; therefore, it is vital to categorize and measure its importance in sustainable practice. Therefore, the objectives set for this study are to identify and assess the major factors affecting FM practice in building projects. In addition, the role of facility managers was explored. The expected findings of this study have the potentiality to raise the awareness and narrow the focus on the factors that affects the building FM services and maintenance practice.

FM AT THE EARLY-DESIGN DEVELOPMENT STAGE

The importance of integrating FM into the design development stage has been the focal discussion of many previous research (Taleb et al., 2017; Au-Yong et al., 2017, Islam et al., 2018). It has conclusively been demonstrated that the successful incorporation of FM into the design process would be the delivery of an efficient and cost-effective facility (Enoma, 2005; Fähnrich and Meiren, 2007). In addition, Enoma (2005) pointed FM will also respond to generate the cost-effective design solutions to fulfil the requirements of building objectives.

Recent studies suggest that FM should be designed in such a way that it will support the core objectives of the facility or structure right from the early design stage (Hassanain et al., 2015). Moreover, another researcher highlighted that the practice of sustainable design and construction can contribute to the generation of competent and less costly building facilities over their life cycle besides improving productivity (Ogungbile and Oke, 2015) Subsequently, they demonstrated that active participation of FM into the planning, design and construction stages confirms the cost-effective FM practice in building projects.

Facilities management works on the basis of "brick and mortar ideas" and the vital role of FM in linking strategic level with operational level (Meng, 2014). Previously FM was usually considered as an intermediate management position. However, in recent years, FM has been moving towards strategic level from the operational level. Figure 1 illustrates the interconnection between two levels and Meng (2014) found that FM should be involved in strategic planning level to obtain the design efficiency. The involvement of FM could influence the decision making and play a critical role to pursue the best practice of sustainable policies at design stage.

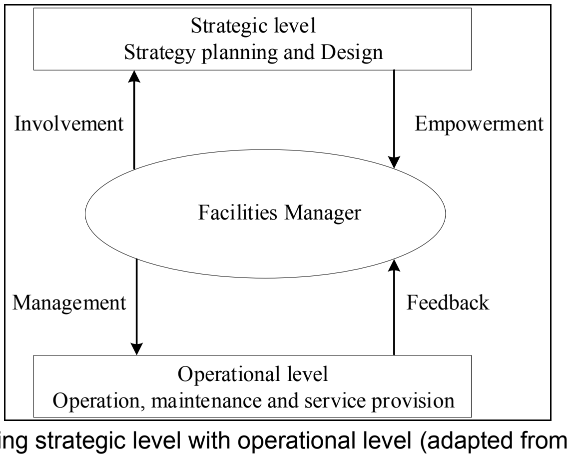

Figure 1. Linking strategic level with operational level (adapted from Meng, 2014)

Figure 2 illustrates the technical roles and activities of the FM in the integrated design team of building projects. A recent study by Ogungbile and Oke (2015) concluded that the participation of FM at design phase carries low maintenance cost at post-occupancy phase through reduction of design alteration and rework. Atkin and Brooks (2009) also reported provision of a facility that is better suited and attractive to potential end-users and clients. It is a facility that can respond to their desires and is easy to run, maintain, control and manage. Moreover, FM at the design development phase adds value to the designed facility through ensuring less "repetitive work", emphasising value for money and efficient control of teamwork.

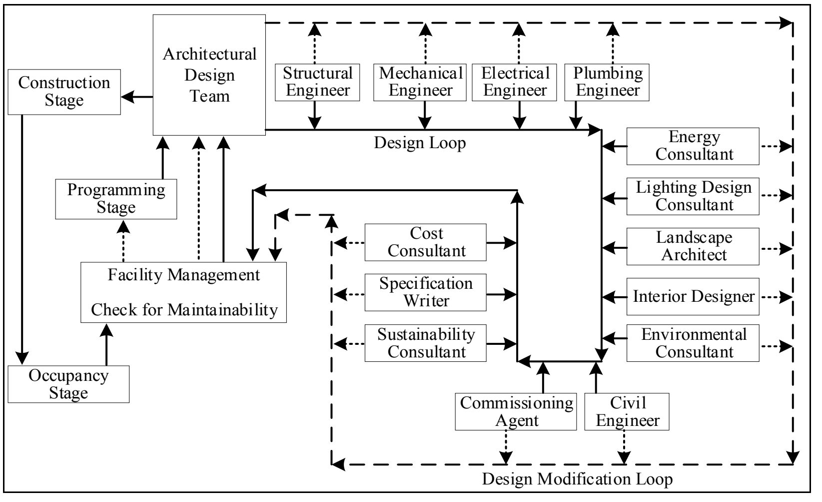

Figure 2. The role of FM within the integrated design team (adapted from Ogungbile & Oke, 2015; Mohammad and Hassanain, 2010)

THE EFFECTS OF DESIGN ON FACILITY OPERATION AND MAINTENANCE

Facilities operation and maintenance covers the wide range of services required to ensure that the built environment will accomplish jobs for built and designed facilities. Operations and maintenance commonly incorporate everyday events vital for the building and its frameworks and equipment to accomplish their proposed jobs. Therefore, architects need to put a great emphasis on producing fruitful designs for their client through inputs of the building systems.

Previous study mentions the significance of FM expert collaboration could strengthened the design output of the design method (Ramly, 2006). Planning for maintenance should be started during the design stage and proceed all through the lifespan of the building. These days, FM faces a large number of continuous and expensive building performance issues at the post-construction stage for operation and maintenance. Previous studies attempted to explain that the decisions adopted in planning and design development phases have a substantial cost effect on the future performance of the building throughout the design life (Ramly, 2006; Chong and Low, 2006). The pressure is augmented to enhance the design process performance due to extremely competitive market and increasing complexity of the modern building.

The quality of design and construction decisions is the main contributor for measuring the building performance. The shortfall in the building performance originates from inadequacies in design and construction which greatly reflects on the maintenance level of building design life. Building life cycle depends on the two vital criteria "design and maintenance" of the building process which are analysed by Kiong and Akasah (2012). By reducing design and construction defects a substantial amount of maintenance expenses can be decreased (Gatlin, 2013). Building maintainability cost is positively and negatively affected by the design decisions made at design stage. Hence, there is a need to consider potentials factors of FM at the design phase to avert unplanned maintenance cost during post occupancy.

ROLE OF FM IN BUILDING DESIGN

The role of FM profession in building project consists of not only coordinating the individual design specialist functions but also coordinating among interdependent specialist contractors as well as concurrently their lower tier of subcontractors' work (Ashuri, 2014; Assaf and Al-Hejji, 2006). However, the inherent gap may be generated intangibly among interdependent design activities and even among intrinsic process as illustrated in Figure 3. To prevent the delay damages, each involved contributor in the project tries to work fast with his/her own specialist perspective. Sometimes, this may lead to an inevitable "chaotic situation" in the associated sub-process due to sluggish information exchange and feedback between design and construction. This situation can also be due to design alteration or error at the upstream of design process (Cigolini et al., 2009; Kibert, 2012). Therefore, introducing the FM concepts to building construction project not only protect the inherent gap but also variations and variability.

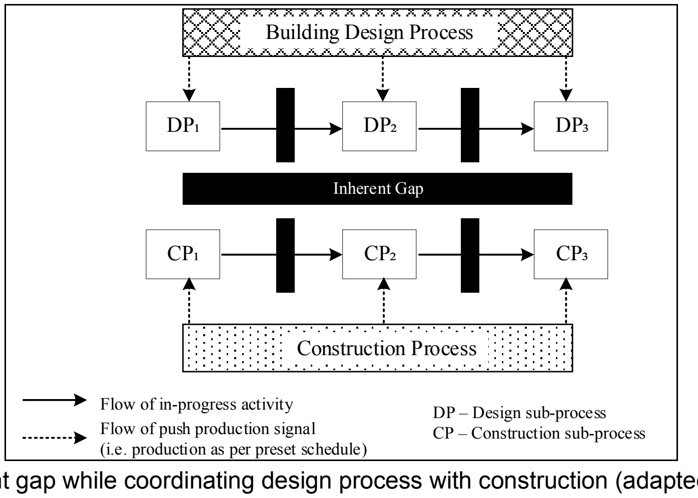

Figure 3. Inherent gap while coordinating design process with construction (adapted from Wan and Kumaraswamy, 2012)

Meng (2014) analysed the interviewed data of 40 FM professionals from different region of UK and revealed that FM should act as an integrator of sustainability in different dimension. In addition, he stated sustainable FM should be "economically efficient, environmentally friendly and socially responsible" at the same time. To better understand the role of FM in building projects, Meng (2014) listed the top five roles include:

i. Integrate all the design issues
ii. Collaborative strategic level with operational level
iii. Add FM knowledge and experience to the design
iv. Spread the sustainable ideas and educate the people
v. Sustainable development through innovation

METHODOLOGY

To achieve the objectives of this paper, an extensive literature review is considered as the methodological approach in the domain of building design and practice of FM is taken into account at present context. This comprehensive review of literature delivered an up-to-date understanding of the prevailing research efforts in these two fields. It was the first step to categorize the design factors that affect the FM practice. The search involved journal publications that discussed design and FM problems frequently. A list of 38 design factors was identified which were cited and chosen recurrently.

A survey questionnaire was then formulated to get the response from the enlisted FM organisations of CIDB and MAFM. CIDB is a principal government authority for the development of construction industry and MAFM is only leading professional institution for FM professionals in Malaysia. In this study total seventy-one (71) FM organizations were chosen as respondents from Kuala Lumpur, Selangor and Johor. and construction professionals in Malaysia. The sample size was restricted to G 7 FM organisations as they are ranked the first position in Malaysia FM industry. Among them twelve (12) were registered member in both CIDB and MAFM. On the other hand, fifty-six (56) organizations were only registered member in CIDB and three (3) were only member in MAFM. A previous study conducted by to Roscoe (1975) cited in Yap et al. (2017) indicated the acceptable range of sample size (f) is $30 \leq f \leq 500$ for most research.

The questionnaire and online survey link were forwarded through email to the respondents for easy access. At the end of the assigned timeframe to respond, 32 out of 71 responses had been completed. The level of consistency was checked for all 32 responses and found 31 valid responses for data analysis. In this research, an effective response rate was calculated 43.67% for online distribution method. A low response rate is not an uncommon phenomenon in the research of FM discipline, for example, 22.9 percent, 14.8 percent, 24.3 percent and 11.40 percent for Haynes and Price (2004), May and Pinder (2008), Meng and Minogue (2011) and Kalantari et al. (2017) respectively.

There were two sections in the survey. The first section was prepared to get the respondents profile and the second section was designed to understand their perception of design factors on FM practice. The questionnaire survey was completed by thirty-one respondents who are directly or indirectly involved with FM industry in Malaysia. Table 1 summarizes the demographic profile of respondents. Fifty-five percent (55%) respondents have more than 10 years of experience, which reflects credibility of the obtained data. Data represents that majority of the respondents in this study were engineers (39%). This was followed by "Facility Manager" 29% and "Quantity Surveyor" 16%. Usually, level of academic achievement represents the level of knowledge. Table 1 shows the highest level of academic achievement for majority of the respondents was bachelor's Degree (45%). Eighty-five (85) percent of the respondents were registered member in Malaysia of their professional bodies, which includes MAFM (48%), which is the only professional organisation for facility mangers in Malaysia.

Table 1. Demographic information of respondents'

Characteristics	Categorizations	Response Count	Response %
Profession of respondent	Engineer	12	39
	Quantity Surveyor	5	16
	Facility Manager	9	29
	Project Manger	3	10
	Others	2	6
Highest academic qualification	PhD	2	6
	Master's Degree	13	42
	Bachelor's Degree	14	45
	Diploma	2	6
Years of experience of respondent in FM	<5	8	26
	5-10	6	20
	11-15	12	39
	16-20	4	13
	>20	1	3
Professional membership	MAFM	15	48
	RISM	6	19
	IEM	5	19
	RICS	1	4
	Others	4	15

DATA ANALYSIS

All design factors drafted into questionnaire tool is designed based on Likert scale with the following values: 1 = "strongly disagree"; 2 = "disagree", 3 = "neutral"; 4 = "agree" and 5 = "strongly agree". The identified design-factors have been grouped into 3 main categories: "architectural design factors, structural design factors and MEP design factors". Then, each category has been divided into 3 sub-groups: "design quality, materials and accessibility".

Analysis of the Questionnaire Survey

The Statistical package for the social sciences (SPSS) was adopted as a statistical tool for frequency distribution analysis, mean, standard deviation, RII and rank analysis. This software is a comprehensive system and provides good precision data with automatically calculated statistics (Sarpin, 2015). In addition, Conbach's alpha coefficient was used to assess the reliability of data obtained from questionnaire which recognises as the most common technique for measuring the internal consistency of questionnaires data. In the survey, respondents were asked to rate importance level of all design factors in FM practice. The numerical score of each factor were transformed to measure RII value in order to assess the ranking and this technique has been adopted in numerous construction management research (Tam et al., 2000; Enshassi et al., 2007; Wan and Kumaraswamy, 2012). RII for each design factor was calculated using following equation:

$$\text{RII} = \frac{\sum_{i=1}^{n} W_i}{N\ X\ W_H} \qquad (0 \leq \text{RII} \leq 1)$$

Where, W_i is the score of each factor as rated by the respondents ranging from 1 to 5; N is the total number of respondents, and W_H is the highest value (i.e. 5) in the survey. Tan et al. (2014) asserted if two or more factors have the same RII value, the SD is compared, so the lower SD gives the higher rank. If the RII value and SD are both same, they were assigned the same rank.

Reliability of the Obtained Data

Before continuing the analysis, the Cronbach's alpha (α) value was calculated, as illustrated in Table 2, to test the internal consistency of the scale in providing appropriate ratings for the 38 design factors in design consideration. Yip and Poon. (2009) and Pallant (2010) indicated that "$\alpha \geq 0.7$" is acceptable, but values of "$\alpha > 0.8$" are more preferable. In this study, α value for "architectural design factors", "structural design factors" and "MEP design factors" were 0.912, 0.925 and 0.955 respectively, which showed strong internal consistency of the scale used and suggested reliable information had been collected.

Table 2. Cronbach's alpha calculation from SPSS

Reliability Statistics		
	Cronbach's Alpha	**N of Items**
Architectural Design Factors	0.912	17
Structural Design Factors	0.925	9
MEP Design Factors	0.955	12

Results and Discussions

Factors Related to Architectural Design

This group comprises seventeen factors under three sub-headings: architectural design quality, architectural materials and architectural accessibility as illustrated in Table 3. The respondents perceived "inadequate working drawing details" as the most significant factor in this group with a mean value of 4.35. This factor influences the project's "specifications and requirements" that affects the FM practice during construction phase. AL Mousli and El-Sayegh (2016) defined that the working drawings must be concise and clear to perceive the full concept of the design. On the other hand, it is the graphical way of communication between design phase and construction phase. Inadequate working drawing details could lead the misinterpretation of the actual project requirements. The effect of this can be seen at the operation and maintenance phase. All respondent perceived "incomplete working drawing and specification" as the second most important factor in this group as well as in overall ranking with a mean value of 4.30. Plans and specifications are the products of design phase. Appearance of incomplete plans and specifications at construction phase results in difficulties in interpreting the actual design information (Arain and Assaf, 2007). This leads not only to a wastage of time and effort at construction phase but also affect the maintenance budget at post-occupancy phase. In addition, respondents were indicated "inappropriate selection and specification of materials" and "non-availability of specific building materials in market" are also design related significant factors, since these clearly affect the post-construction activities and performance of building facilities at operation and maintenance stages. This could be attributed to the architectural design team and facility manager's concern at conceptual phase of design about the availability, suitability and quality of materials used in construction. The importance indices for all factors are illustrated in Table 3. It clearly shows that "inadequate working drawing details" and "incomplete working drawing and specification" are ranked as the most significant by all respondents, with a significant index of 0.87 and 0.86 respectively.

Table 3. Assessment of design factors affecting FM practice

	Design Factors	Mean	SD	RII	Rank	
					In Group	Overall
	Architectural					
	Architectural Design Quality					
1	Incomplete working drawing and specification	4.30	0.86	**0.86**	**2**	**2**
2	Inadequate working drawing details	4.35	0.70	**0.87**	**1**	**1**
3	Signs of stains and seepage around windows	4.09	0.72	0.82	5	6
4	Plaster decay on external wall surface	4.00	0.61	0.80	6	7
5	Moisture and vapours pass through	4.05	0.71	0.77	9	13
6	Unpleasant odor transmits due to inappropriate position of kitchen and toilets	3.61	0.92	0.72	14	24
7	Signs of stains visible lack of ventilation	3.87	0.74	0.77	9	11
	Architectural Materials					
8	Specification of bellow-standard quality tiles	3.87	1.15	0.77	9	14
9	Chipping around sharp wall edges	3.70	0.95	0.74	13	20
10	Non-availability of specific building materials in market	4.22	0.83	**0.84**	**4**	**3**
11	Inappropriate selection and specification of materials	4.22	0.98	**0.84**	**3**	**4**
12	Inability to maintain vertical riser for service shaft	3.87	0.80	0.77	9	12
13	Paint peeling, flaking, blistering, biological attack and efflorescence	3.91	0.88	0.78	7	8
14	Specify dark colour paint as an exterior finish	3.52	1.02	0.70	15	30
	Architectural Accessibility					
15	Difficult to reach and maintain fenestration	3.91	1.02	0.78	7	9
16	Challenging to move furniture and equipment due to the wrong dimension of doors	3.35	1.00	0.67	17	38
17	Building shape and orientation	3.48	1.02	0.70	16	32
	Structural					
	Structural Design Quality					
18	Plaster crack at joint i.e. concrete-brick joints and wall-floor joints	4.13	0.93	**0.83**	**1**	**5**
19	Inadequate concrete cover causes rebar corrosion	3.65	1.13	0.73	4	23
20	Moisture penetration at beam-wall joints, walls, and ceiling-wall joints	3.61	1.01	0.72	5	26
21	Inadequate structural design	3.74	1.11	0.75	3	19
	Structural Risk					
22	Differential settlement generates crack in floor slabs, walls, and tiles	3.52	0.97	0.70	7	29
23	Tile cracks, failure and fraction at fragile points	3.83	1.09	0.77	2	15
24	Cracks in columns and beams	3.43	1.17	0.69	8	37
	Structural Process					
25	Uncommon design and technology	3.43	1.14	0.69	8	36
26	Short time frame for design preparation	3.57	1.10	0.71	6	28
	MEP					
	MEP Design Quality					
27	Total power fail from one error	3.57	0.92	0.71	8	27
28	Short circuit in plug points	3.48	1.02	0.70	10	32
29	Lack of ventilation propagate unpleasant odor	3.70	1.08	0.74	5	21
30	Water ponding at roof due to inadequate drainage system	3.74	0.94	0.75	3	17
31	Shortfall of the HVAC system for comfortable temperature	3.61	0.97	0.72	7	25
32	Lack of condensation drainage systems causes water leakage	3.74	0.94	0.75	3	17
	MEP Materials					
33	Inadequate number and distribution of socket points	3.65	0.96	0.73	6	22
34	Water leakage through floors and walls	3.87	0.90	0.77	1	10
35	Inappropriate insulation causes noisy air-handling unit	3.43	0.88	0.69	12	35
	MEP Accessibility					
36	In appropriate placement of light switches, main boards and circuit breakers	3.48	1.10	0.70	10	34
37	Unable to reach pipelines and sewer line for maintaining	3.83	1.13	0.77	2	16
38	Unable to reach chillers and condensers for maintaining	3.52	1.14	0.70	9	31

Factors Related to Structural Design

This group consists of nine factors divided into three sub-headings: structural design quality, structural risk and structural process as presented in Table 3. The respondents perceived “plaster crack at joint i.e. concrete-brick joints and wall-floor joints” as the most important factor in this group, with a mean value and significant factor of 4.13 and 0.83 respectively, as shown in Table 3. This could be attributed to lack of drawing details and design errors generated from the preliminary stage. This design decision affects the quality of construction work, which subsequently results into incessant maintenance activities during the operational phase of building use. In a recent study by Al-Kafrawi (2011) and Hassanain et al. (2015) pointed that preliminary maintainability checking of drawing details can minimize the occurrence of this crack with specifying a mesh between two different building components. Additionally, the respondents were also highlighted the following two factors as significant in this group: “tile cracks, failure and fraction at fragile points” and “inadequate structural design”, with mean value of 3.83 and 3.74 respectively. The importance indices for these factors were also calculated as shown in Table 3, with a significant index of 0.77 and 0.75 respectively. Thermal movement was marked as the main cause for “tile cracks, failure and fraction at fragile points”. Appropriate designing of regular expansion and contraction joints at design phase by structural designer can reduce this design error (Chew, 2010). This is evident in the fact that structural design quality and process affect the coordination process with various design specialists. Consequently, difficulties arise in fulfilling the requirements of constructability and maintainability.

Factors Related to MEP Design

This group includes twelve factors related to three sub-classes: MEP design quality, MEP materials and MEP accessibility, as shown in Table 3. The average result of all respondents represents that “water leakage through floors and walls” was rated as the most significant factor in this group, with the mean value and important index of 3.87 and 0.77 respectively. It is caused due to lack of proper detailing of the waterproofing membrane around running pipelines penetrating at the floor levels. Waterproofing membrane provides monolithic entity for the construction work (Chew, 2010; Hassanain et al., 2014). The “inappropriate insulation causes noisy air-handling unit” was ranked as the least factor in this category by the respondents. May be this happened due to lack of experience and knowledge. Additionally, “unable to reach pipelines and sewer line for maintaining” is also rated as important factor in this cluster, with mean value and important index of 3.83 and 0.77 respectively. Ease of access is a key design criterion, as it influences the ease and effectiveness of maintenance operations of building services. This factor ranks expose a less amiable working coordination between the plumbing designer and the facility manager. This could be attributed to both parties being at the far ends during the design phase. It is also noted that “water ponding at roof due to inadequate drainage system” and “lack of condensation drainage systems causes water leakage” were ranked high in this category, with same measured mean value and important index of 3.74 and 0.75 respectively. Inadequate design details of roof sloops and unavailability of roof drain were pointed out as the main causes for water ponds, which lead to damage the interior finish by the pouring the water (De Silva, 2010; Hassanain et al., 2014). Based on the discussion of the survey result of this study, it is clearly perceived that any sort of design-drawing related issues will result in a waste of resources. Premature placement of building facilities services consequently cost over-runs. The authors are in strong agreement with the fact that appropriate design decision and maintainability check at design development phase will facilitate an easy FM practice.

CONCLUSION AND RECOMMENDATIONS

The rapidly increasing modern building with their sophisticated facilities service is already a cause for serious concern in Malaysia. This FM services subsector being a main part of the building construction industry is no exception in contributing to and sharing this prime concern. In recent days, cost-effective practice of FM is an emerging focus in this sector with the rapid growth and maturity of FM profession. It addresses design factors of building facilities during the operation and maintenance stage. This study presents a collection of thirty-eight design related factors affecting FM practice at the post-occupancy phase. These design factors have been identified from rigorous literary study and brainstorming exercise.

This was followed by a questionnaire survey and thirty-one respondents graded them in a five-point Likert scale during the online survey. The design factors were ranked accordingly using RII value, prior to this mean value and the standard deviation was computed through SPSS. From the primary data analysis, this study findings reveal that the most important design factors affecting the FM practice are: “incomplete working drawing and specification”, “inadequate working drawing details”, “non-availability of specific building materials in market”, “inappropriate selection and specification of materials” and “plaster crack at joint i.e. concrete-brick joints and wall-floor joints”, as presented in Table 4.

Table 4. The top five design related factors affecting FM practice

No	Design Factors	Rating	
		RII	Rank
2	Inadequate working drawing details	0.87	1
1	Incomplete working drawing and specification	0.86	2
11	Inappropriate selection and specification of materials	0.84	3
10	Non-availability of specific building materials in market	0.84	4
18	Plaster crack at joint i.e. concrete-brick joints and wall-floor joints	0.83	5

The identification of these design factors provides a useful guideline for various design practitioners and facility managers to pursue a cost-effective FM practice. Knowledge of the ranks of the various design factors can be used by the design professionals to prioritize strategies to enhance their practice in building project. Moreover, all design needs to pass through the maintainability check by FM team for a thorough analysis and detailing to ensure the specified requirements. Based on the discussions of the obtained results, the authors have strong agreement that quality assurance measure and FM concepts should be incorporated for all design outputs including conceptual and detailed design stages of building project. Consequently, an amiable working relationship will be developed for producing accurate drawings and other documentation, where diverse materials, all professional members, sub-contractors and suppliers are involved.

This research is inspiring for experts and researchers to devote themselves to the future development of FM practice in building projects. The design factors affecting FM practice is wide and the coverage shall not only be limited to current studies. Furthermore, a case study approach is suggested to validate the importance of design factors on FM performance.

ACKNOWLEDGEMENT

The authors would like to thank PRGS # R. J130000.7821.4L692, UTM, Malaysia for the support and facilities that made this paper possible.

REFERENCES

AL Mousli, M. H., & El-Sayegh, S. M. (2016). Assessment of the design–construction interface problems in the UAE. *Architectural Engineering and Design Management*, Vol. 12, No. 5, pp 353-366.

Alaloul, W. S., Liew, M. S., and Zawawi, N. A. W. A. (2016), Identification of coordination factors affecting building projects performance, *Alexandria Engineering Journal,* Vol. 55, No. 3, pp. 2689-2698.

Al-Kafrawi, M. S. (2011). Interview, Maintenance Manager at King Fahd University of Petroleum and Minerals, Maintenance Department, Dhahran, Saudi Arabia, November 20.

Arain, F. M., & Assaf, S. A. (2007). Consultant's prospects of the sources of design and construction interface problems in large building projects in Saudi Arabia. *JKAU: Environmental Design Sciences*, 5(2), 15-37.

Ashuri, B., Yarmohammadi, S., and Shahandashti, M. (2014), A critical review of methods used to determine productivity of mechanical, electrical, and plumbing systems coordination, *Proceedings of the Construction Research Congress*, American Society of Civil Engineers, May 19-21, Atlanta, Georgia, pp. 777-786.

Assaf, S. A., and Al-Hejji, S. (2006), Causes of delay in large construction projects, *International Journal of Project Management,* Vol. 24, No. 4, pp. 349-357

Atkin, B. and Brooks, A. (2009), *Total Facilities Management*, 3rd ed., Blackwell, Oxford.

Au-Yong, C. P., Ali, A. S., and Ahmad, F. (2017), Competency and commitment of facilities managers: keys to safeguard maintenance performance. *Malaysian Construction Research Journal,* Vol. 22, No. 2, pp. 35-46.

Chew, M. Y. L. (2010). Maintainability of facilities for building professionals, World Scientific, Singapore.

Chong, W. K., and Low, S. P. (2006). Latent building defects: Causes and design strategies to prevent them. *J. Perform. Constr. Facil*., volume 20:3(213), pp. 213–221.

Cigolini, R.D., Zwan, J.V.D., Straub, A., Martinez, D., Aiello, G., Mazziotta, V. and Micale, R. (2009), "Facility management, outsourcing and contracting overview", in Cigolini, R.D., Deshmukh, A.V., Fedele, L. and McComb, S.A. (Eds), Recent Advances in Maintenance and Infrastructure Management, Springer, London, pp. 225-290.

De Silva, N., & Ranasinghe, M. (2010). Maintainability risks of condominiums in Sri Lanka. *Journal of Financial Management of Property and Construction*, *15*(1), 41-60.

Enoma, A. (2005). The role of facilities management at the design stage. In *21st Annual ARCOM Conference* (Vol. 1, pp. 421-430).

Enshassi, A., Mayer, P.E., Mohamed, S. and El-Masri, F. (2007), "Perception of construction managers towards safety in Palestine", *The International Journal of Construction Management*, Vol. 2007, pp. 41-51.

Fähnrich, K. P., & Meiren, T. (2007). Service engineering: state of the art and future trends. *Advances in services innovations*, 3-16.

Gatlin, F. R. A. N. K. (2013), Identifying & Managing Design and Construction Defects'. *Insight from Hindsight, volume* 5, pp. 1-11.

Hassanain, M. A., Adewale, B., Al-Hammad, A. M., & Sanni-Anibire, M. O. (2017). Factors affecting building services coordination during the design development and review stages. *Built Environment Project and Asset Management*, (just-accepted), 00-00.

Hassanain, M. A., Al-Hammad, A. M. & Fatayer, F. (2015). Design-Phase Maintenance Checklist for structural durability. *Implementing Innovative in Structural Engineering and Project Management*, *28*(3), 1-6.

Hassanain, M. A., Fatayer, F., & Al-Hammad, A. M. (2014). Design phase maintenance checklist for water supply and drainage systems. *Journal of Performance of Constructed Facilities*, *29*(3), 04014082.

Haynes, B., and Price, I. (2004). Quantifying the complex adaptive workplace. *Facilities,* 22(1/2), 8-18.

Islam, R., Mohamed, S. F., Bjørberg, S., Misnan, M. S., & Yusof, Z. M. (2017) Towards a Framework to Integrate Facilities Management Cost Effective Parameters in Design Process. *International Journal of Engineering and Technology*, 9(6), 4515-4526.

Islam, R., Mohamed, S. F., Nazifa, T. H. and Yusof, Z. M (2018). Sustainable Design Parameters in Generation of Facilities Management Conceptual Model. Proceedings *IEEE International Conference on Computing, Electronics and Communications Engineering (iCCECE 2018)*, University of Essex, Southend, UK. pp. 156-161.

Jensen, P. A. (2009) Design Integration of Facilities Management: A Challenge of Knowledge Transfer. Architectural, Engineering and Design Management, 5, pp. 124-135.

Josephson, P. E., and Hammarlund, Y. (1999), The causes and costs of defects in construction: A study of seven building projects. *Automation in Construction*., Vol. 8, No. 6, pp. 681–687.

Kalantari Hematabadi, S.S. (2014). *Toward an economic design process: enhancing building performance through better integration of facility management in the design process*. Doctoral dissertation, TX A&M University, College Station, TX.

Kibert, C.J. (2012), *Sustainable Construction: Green Building Design and Delivery*, 3rd ed., John Wiley & Sons, Hoboken, NJ.

Kiong, N.B., and Akasah, Z.B. (2012), Analysis of building maintenance factors for IBS precast concrete system: a review. *International Journal of Engineering Research and Applications*, volume 2 (6), pp. 878-883.

Korman, T., Simonian, L., and Speidel, E. (2010). How building information modelling has changed the MEP coordination process", In Ghafoori (Ed.), *Challenges, Opportunities, and Solutions in Structural Engineering and Construction,* Taylor & Francis, London, 959-963.

Lee, G. (2014), Parallel vs. sequential cascading MEP coordination strategies: a pharmaceutical building case study, *Automation in Construction*, Vol. 43, pp. 170-179.

Love, P. E., Lopez, R., Kim, J. T., & Kim, M. J. (2014). Influence of organizational and project practices on design error costs. *Journal of Performance of Constructed Facilities*, *28*(2), 303-310.

May, D. and Pinder, J. (2008), "The impact of facilities management on patient outcomes", *Facilities*, Vol. 26 Nos 5/6, pp. 213-28

Meng, X. (2014). The role of facilities managers in sustainable practice in the UK and Ireland. *Smart and Sustainable Built Environment*, *3*(1), 23-34.

Meng, X., & Minogue, M. (2011). Performance measurement models in facility management: a comparative study. *Facilities, 29*(11/12), 472-484.

Mohamed, S. F., & Anumba, C. J. (2006). Potential for improving site management practices through knowledge management. *Construction Innovation*, *6*(4), 232-249.

Mohammed, M. A., & Hassanain, M. A. (2010). Towards improvement in facilities operation and maintenance through feedback to the design team. *The Built & Human Environment Review*, *3*, 72-87.
Ogungbile, A. J., & Oke, A. E. (2015). Assessment of facility management practices in public and private buildings in Akure and Ibadan cities, south-western Nigeria. *Journal of Facilities Management*, *13*(4), 366-390.
Pallant, J. (2010). SPSS survival manual: A step by step guide to data analysis using SPSS: Open University Press
Rahman, S. B. A. & Salim, N. A. (2013) A Study on Design Defects Affecting Maintenance. 5th *Undergraduate Maintenance and Facilities Management Conference* 2013,
Ramly, A. (2006) A link between design and maintenance. *Journal of Building Engineers*, volume 81 (5).
Riley, D. R., Varadan, P., James, J. S., and Thomas, H. R. (2005), Benefit-cost metrics for design coordination of mechanical, electrical, and plumbing systems in multi-storey buildings, *Journal of Construction Engineering and Management,* Vol. 131, No. 8, pp. 877-889.
Sarpin, N. (2015). *Developing people capabilities for the promotion of sustainability in facility management practices*. Doctoral dissertation, Queensland University of Technology.
Taleb, H., Ismail, S., Wahab, M. H. and Rani, M. W. N. W. (2017), Managing communication between architects and clients during design phase, *Malaysian Construction Research Journal,* Vol. 2, No. 2, pp. 69-79.
Tam, C.M., Deng, Z.M., Zeng, S.X. and Ho, C.S. (2000), Quest for continuous quality improvement for public housing construction in Hong Kong, *Construction Management and Economics*, Vol. 18 No. 4, pp. 437-46.
Tan, Y., Shen, L., Langston, C., Lu, W., & CH Yam, M. (2014). Critical success factors for building maintenance business: a Hong Kong case study. *Facilities*, *32*(5/6), 208-225.
Wan, S. K. M., and Kumaraswamy, M. M. (2012), Improving building services coordination at the pre-installation stage, *Engineering, Construction and Architectural Management,* Vol. 19, No. 3, pp. 235-252.
Wong, I., & Chan, A. (2014). How to Quantify Quality Architectural Design from the Aspect of Building Maintenance. In *ICCREM 2014: Smart Construction and Management in the Context of New Technology*, pp. 1472-1479).
Yip, C. P. and C. S. Poon. (2009). Cultural shift towards sustainability in the construction industry of Hong Kong. *Journal of Environmental Management*, vol. 90, no. 3616-3628.

EFFECT OF CURING CONDITIONS USING SLAG BASED CEMENTITIOUS BINDER AS AN ACTIVATOR

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Abstract
The effects of curing conditions on the compressive strength development and microstructure of no-cement cementitious binder produced by ground granulated blast furnace slag activated with calcined dolomite were investigated. The optimum addition of calcined dolomite was found at amount of 20 wt% (percentage by weight) for both curing conditions. The optimum 28-day compressive strengths of hardened slag-dolomite binder were approximately 23.8 and 30.7 MPa for air and water curing, respectively. The slag-dolomite binders with calcined dolomite amount of 10, 20, 30, and 40 wt% which were cured in the water had the higher 28-day compressive strengths by 16.5, 29.1, 26.9, and 14.4%, respectively, than those being cured in air. The water curing serves as an appropriate curing method for the slag-dolomite binder to reduce porosity as well as obtain a higher strength with better durability. The microstructural investigation revealed that the identical hydration products of slag-dolomite binder occurred in both curing conditions. The main hydration products of the hardened slag-dolomite binder with calcined dolomite amount of 20 wt% were the calcium silicate hydrate (C-S-H), Portlandite ($Ca(OH)_2$), silica (SiO_2), calcite ($CaCO_3$), and magnesium calcite ($Ca_{0.936}Mg_{0.064}CO_3$).

Keywords: *Activated slag; calcined dolomite; no-cement binder; curing condition*

INTRODUCTION

Ground granulated blast furnace slag (slag) is a by-product from the blast-furnace used to make iron from ore, which is rich in lime (CaO), silica (SiO_2), and alumina (Al_2O_3) (Mindness et al., 2008). Owing to the environmental concern associated with the high amount of carbon dioxide (CO_2) emissions from ordinary Portland cement (OPC) production. Many researchers have focused on the usage of ground granulated blast furnace slag as an OPC replacement (Kumar et al., 2008; Guneyisi et al., 2008; Mo et al., 2015). Figure 1 demonstrated that the chemistry of slag close to OPC inside the CaO-SiO_2-Al_2O_3 ternary diagram (Lothenbach et al., 2011). The slag without an activator reacts slowly with the water, due to the presence of an impervious coating of amorphous silica and alumina that form around slag particles early in hydration process (Mindness et al., 2008; Kim et al., 2011). Slag needs to be activated by alkaline compounds in order to improve its mechanical properties. The most common activator for the slag is alkaline activator such as sodium hydroxide (NaOH), sodium silicate (Na_2SiO_3), and potassium hydroxide (KOH) (Mindness et al., 2008; Palomo et al., 1999; Yang et al., 2012). On the other hand, the calcium oxide (CaO) and magnesium oxide (MgO) can also be acted as the alternative slag activator (Jeong et al., 2016; Kim et al., 2013; Jin et al., 2015). Dolomite ($CaMg(CO_3)_2$) is the mineral composed of the double carbonate of calcium (Ca) and magnesium (Mg) and crystallizes in a type of rhombohedral unit cell (Olszak-Humienik and Mozejko, 1999; McIntoch et al., 1990). The thermal decomposition of dolomite occurs in two stages (Olszak-Humienik and Jablonski, 2015) shown as follows:

$$CaMg(CO_3)_2 \rightarrow CaCO_3 + MgO + CO_2 \quad (1)$$

$$CaCO_3 \leftrightarrow CaO + CO_2 \quad (2)$$

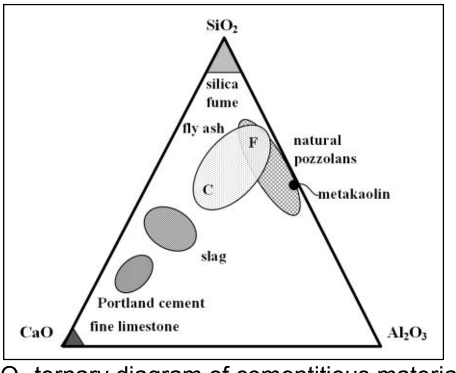


Figure 1. CaO-Al_2O_3-SiO_3 ternary diagram of cementitious materials (Lothenbach et al., 2011)

Some previous studies have shown that the calcined dolomite is a cementitious material but could also be used as a slag activator (Gu et al., 2014; Djayaprabha et al., 2017), a partial replacement of OPC (Jauffret et al., 2016), and an additive in the alkali-activated Portland slag cement (Yang et al., 2014). The proper curing method should be developed in order to allow the hydration and pozzolanic reactions to occur so that the optimum mechanical properties of the mixture may properly develop (ASTM C125-16). Some recent studies in alkali-activated slag binder have shown the effects of curing condition on the compressive strength development, for instance, Dung et al. (2014) reported that the slag activated by circulating fluidized bed combustion (CFBC) fly ash under either air or water curing can reach about 40-75 MPa at 28 days. Chen et al. (2015) reported that the slag activated by both CFBC and class F fly ashes with air curing could reach about 68 MPa at 28 days. Therefore, the investigation of appropriate curing method for the no-cement binder with two kinds of cementitious materials of both slag and calcined dolomite becomes substantial and still lack sufficient studies.

In this study, the compressive strength development of no-cement cementitious slag-dolomite binder produced by the slag activated by calcined dolomite was investigated using four different mixes with different replacement amounts of 10, 20, 30, and 40 wt% (percentage by weight). The effects of water and air curing conditions on the compressive strength development and microstructure were investigated. In addition, the thermal properties of dolomite were investigated with thermogravimetric analysis (TGA) and the microstructural analyses using Scanning Electron Microscope (SEM) and X-ray Diffraction (XRD) were used to determine the material properties and hydration products of the slag-dolomite binder.

EXPERIMENTAL PROGRAM

Materials and Mix Proportions

The base materials used in this study for producing slag-dolomite binder were the slag and natural dolomite powders with the physical properties and chemical compositions as shown in Table 1. The calcined dolomite powder was obtained by heating up the natural dolomite powder at complete decarbonation temperature 900°C for 1 h in a high-temperature electric furnace with a temperature ramp of 5 ± 0.2°C/min (Figure 2). The thermal behaviour of natural dolomite was investigated through the thermogravimetric analysis (TGA) as shown in Figure 3. The weight loss of 36.7% occurred between 631.2 and 810.2°C which was attributed to the decomposition of CaMg $(CO_3)_2$ to form CaO and MgO. Those oxides were used in order to trigger the hydration of the slag (Jeong et al., 2016; Kim et al., 2013; Jin et al., 2015).

Table 1. Physical properties and chemical composition of slag and natural dolomite

	Slag	Natural Dolomite
Specific Gravity	2.83	2.70
Blaine Fineness, cm^2/g	6000	-
SiO_2, %	33.54	2.71
Al_2O_3, %	13.00	-
CaO, %	39.85	90.36
K_2O, %	-	0.23
MgO, %	6.90	1.84
SO_3, %	1.67	-
Fe_2O_3, %	0.50	0.52
L.O.I, %	4.54	4.34

Figure 2. Natural dolomite calcination inside in a high-temperature electric furnace

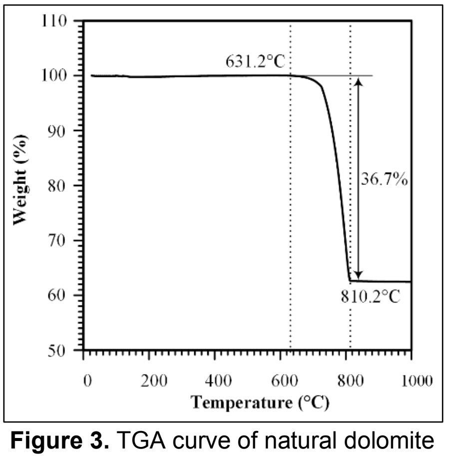


Figure 3. TGA curve of natural dolomite

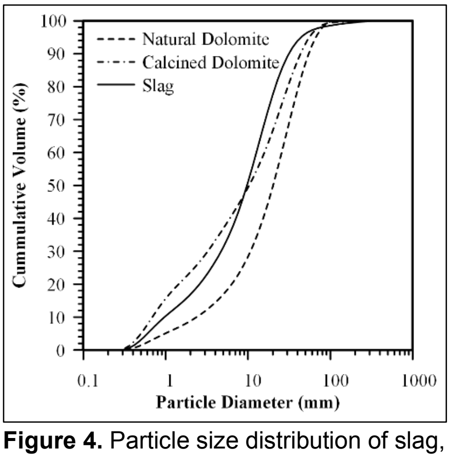


Figure 4. Particle size distribution of slag, natural dolomite, and calcined dolomite

Figure 4 demonstrates the particle size distribution of slag, natural dolomite, and calcined dolomite powders. The volumetric weighted mean of the slag, natural dolomite, and calcined dolomite are 17.85, 28.69, and 20.07 µm, respectively. The XRD verified that slag mostly includes CaO, SiO_2, and Al_2O_3 under amorphous phases (Figure 5a). Natural dolomite contains two significant phases of dolomite ($CaMg(CO_3)_2$) and calcite ($CaCO_3$) (Figure 5b). Calcined dolomite includes three major phases $CaCO_3$, CaO, and MgO (Figure 4c) in accordance with the thermal decomposition as shown in Equation (1) and (2). Four slag-dolomite binder mixes were calculated based on the volume method, in which the slag and calcined dolomite were denoted as SL and CD, respectively, and the following number represents the weight percentage of materials (Table 2). The water-to-binder ratio was fixed as 0.4.

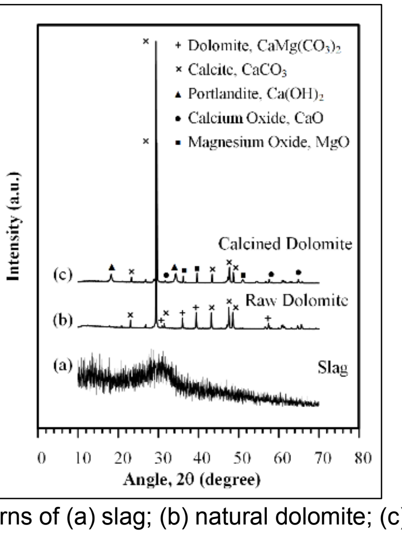


Figure 5. XRD patterns of (a) slag; (b) natural dolomite; (c) calcined dolomite

Table 2. Mixture proportions of the slag-dolomite binder units: kg/m^3

Mixture Code	Water	Slag	Calcined Dolomite
SL90CD10	531	1195	133
SL80CD20	531	1062	266
SL70CD30	531	930	399
SL60CD40	531	797	532

Notes: SL = slag, CD = calcined dolomite, the numbers denote the weight percentages.

Specimens, Curing Condition and Experimental Procedure

Three 50-mm cubic specimens (Figure 6a) for each mixture at different curing ages were prepared for determining the compressive strength development. The specimens were made by pouring fresh binder mixture into the cubic mould, which then was tamped to decrease air bubble, covered by clinging wrap, and demoulded after 24 hours. The specimens were cured in temperature 25±2°C with 70±10%RH (relative humidity) for air curing, while other specimens were immersed in saturated lime water at a temperature of 25±2°C for water curing.

The compressive strength test was carried out using a computer-controlled servo-hydraulic compression testing machine with a 2000-kN capacity (Figure 6b) according to ASTM C109/109M-16a. The compressive strength tests of the hardened slag-dolomite binder

(Figure 6c) were determined at the ages of 7, 28, and 365 days. The 7-day, 28-day, and 365-day tests were conducted to determine the early strength, design strength, and long-term strength, respectively.

(a) (b) (c)

Figure 6. Procedure of the compressive strength test

The 28-day microstructural examination of harden slag-dolomite binders with calcined dolomite amount of 20 wt% (SL80CD20) was investigated through SEM using JOEL model JSM-6390LV (Figure 7) and XRD using Brucker D2 Phaser (Figure 8).

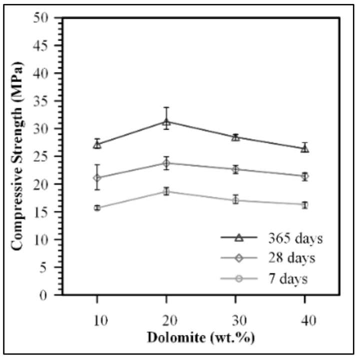
Figure 7. Scanning electron microscopy apparatus (JOEL model JSM-6390LV)

Figure 8. X-ray diffraction apparatus (Brucker D2 Phaser)

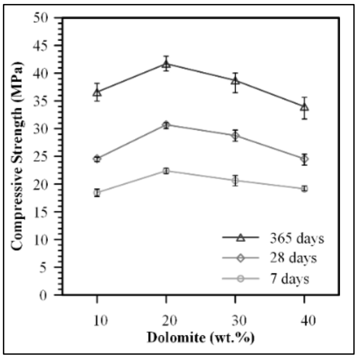
RESULT AND DISCUSSION

Compressive Strength Development

Figure 9 demonstrates the compressive strength development of the hardened slag-dolomite binders with four different amounts of calcined dolomite, which were cured in both air and water at the ages of 7, 28, and 365 days. The addition of calcined dolomite at amount of 20 wt% (SL80CD20) for both air and water curing conditions reached the highest 28-day compressive strength of 23.8 and 30.7 MPa, respectively. By comparing with SL80CD20, the 28-day compressive strength of slag-dolomite binder with calcined dolomite amount of 30 wt% (SL70CD30) and 40 wt% (SL60CD40) decreased by about 4.7 and 9.8% for air curing and 6.3 and 20.0% for water curing. The added amount of calcined dolomite being over 20 wt% decreased the compressive strength due to the local inhomogeneous regions of hydration product. The compressive strength continues to increase with the curing ages as a result of hydration process of the slag-dolomite binder. After curing for one year, the compressive strength of SL80CD20 which was cured in air and water reached 31.3 and 41.7 MPa, respectively. It is evident that the calcined dolomite can be an effective activator for slag.



(a) Cured in air



(b) Cured in water

Figure 9. Compressive strength development of slag-dolomite binder specimens with various dolomite content

Effect of Curing Conditions

The comparison of two different curing methods, water and air curing, were investigated on the optimum mixture with the calcined dolomite amount of 20 wt% (SL80CD20) at the ages of 3, 7, 14, 18, 56, and 365 days, as shown in Figure 10. The significant effect of curing method exhibited that the mixture of SL80CD20 being cured in water has a 28-day compressive strength higher by 29.1% than that cured in the air. The steeper slope of later compressive strength development was found in water curing and resulted in 33.3% higher compressive strength at the age of 365 days. Owing to the insufficient moisture, the retarded hydration process and formation calcium silicate hydrate (C-S-H) gel of slag-dolomite binder which was cured in air resulted in a lower compressive strength.

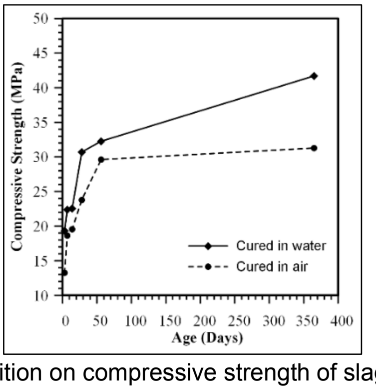
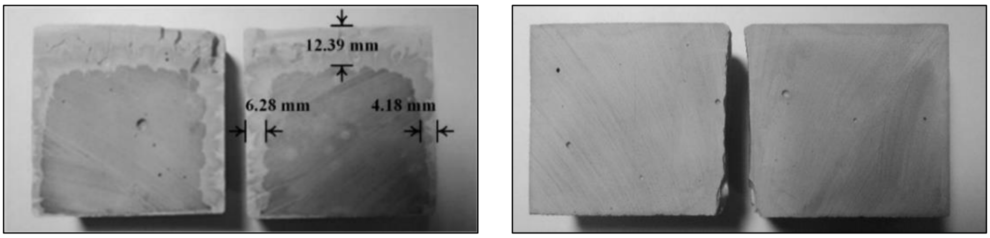


Figure 10. Effect of curing condition on compressive strength of slag-dolomite binder with calcined dolomite amount of 20 wt%.

In order to study the effect of long-term curing method on the durability, the one-year-old 50-mm cubic specimens of the slag-dolomite binder with calcined dolomite amount of 20 wt% (SL80CD20) which was cured both in air and water were cut through its middle line, as shown in Figure 11. The effect of unavoidable carbonation on the harden slag-dolomite binder which was cured in air can be clearly recognized by the presence of a lighter colored zone, as shown in Figure 11a. The carbonation occurred on all exposed sides, except for the bottom side being touched with the base during the curing time. The average carbonation depth on the top side was around 12.39 mm, whereas the others were around 4.18 and 6.28 mm respectively (Figure 11a). As a result of CO_2 attack, the unreacted CaO in the slag-dolomite binder turns back to $CaCO_3$, in accordance with the reversibility of chemical reaction as shown in Equation (2). The carbonated slag-dolomite binder with a lighter colored zone has a lower strength than that of the darker colored zone of complete hydration. The uniformity of hydration product of slag-dolomite binder was found in the specimen, which was cured in the water, as shown in Figure 11b. This phenomenon explains the reason that the slag-dolomite binder which was cured in water has a higher compressive strength and better durability than those cured in air.



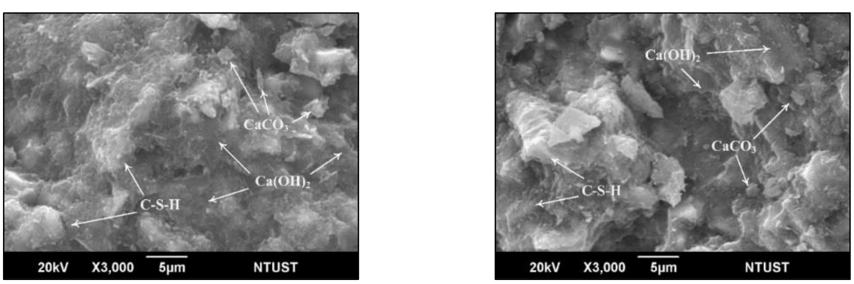
(a) Cured in air (b) Cured in water

Figure 11. The cross-section of the one-year-old hardened slag-dolomite binder with calcined dolomite amount of 20 wt%.

Microstructural Analyses

The 28-day SEM micrographs for harden slag-dolomite binders with calcined dolomite amount of 20 wt% (SL80CD20) for the air and water curing conditions were provided in Figures 12a and 12b, respectively. The micrographs revealed that hydration products of slag-dolomite binder on both curing conditions are mainly the calcium silicate hydrate (C-S-H), calcite ($CaCO_3$), and Portlandite (Ca $(OH)_2$). At the curing age of 28 days, the specimens being cured under either air or water curing condition resulted in the identical degrees of hydration (Figure 12a and 12b). The micrographs proved that a sufficient amount of water in the mixture can assure the continuous hydration process in the slag-dolomite binder.

The XRD pattern of slag-dolomite binder with calcined dolomite amount of 20 wt% (SL80CD20), as shown in Figure 13 exhibited the humps at approximate $2\theta = 30°$ which indicated a presence of the amorphous C-S-H gel. Moreover, $CaCO_3$ is detected at $2\theta = 23.1°$, 29.4°, 43.1° and 57.4°, and $Ca(OH)_2$ is detected at $2\theta° = 18.1°$, 34.1°, and 47.5°, and silica (SiO_2) is detected at $2\theta = 26.6°$, and magnesium calcite ($Ca_{0.936}Mg_{0.064}CO_3$) is detected at $2\theta =$ 35.9°, 39.4°, and 48.5°, which are consistent with the previous results (Djayaprabha et al., 2017).



(a) Cured in air (b) Cured in water

Figure 12. Micrographs of harden slag-dolomite binder with calcined dolomite amount of 20 wt% at curing age of 28 days

It is recognized that the hydration process of slag-dolomite binder involved reaction of CaO with H_2O to form $Ca(OH)_2$, as shown in Equation (3). The hydration of CaO rises the heat and accelerates the dissolution of the active silica in the slag and produces C-S-H, as shown in Equation (4) (Dung et al., 2014).

$$CaO + H_2O \rightarrow Ca(OH)_2 + Heat \tag{3}$$

$$XCa(OH)_2 + SiO_2 + (y-x)H_2O \rightarrow C_xSH_y \tag{4}$$

It can be confirmed that the later strength of slag-dolomite binder was developed by the formation of C-S-H gel during the hydration process.

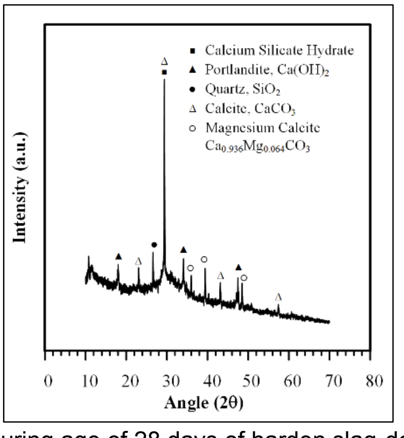


Figure 13. XRD patterns at curing age of 28 days of harden slag-dolomite binders with calcined dolomite amount of 20 wt% which being cured in water

CONCLUSIONS

The utilization of calcined dolomite as an activator for the ground granulated blast furnace slag to produce no-cement cementitious binder has been presented in this study. The optimum addition of calcined dolomite as an activator was found to be the amount of 20 wt% (SL80CD20) with the 7-, 28-, and 365-day compressive strengths of 18.6, 23.8, and 31.3 MPa for air curing and 22.4, 30.7, and 41.7 MPa for water curing, respectively. It is obvious that slag as a low-hydrating cementitious material can achieve high compressive strength when activated by calcined dolomite. The effects of curing condition on the compressive strength development were investigated in both air and water curing conditions. The slag-dolomite binder with calcined dolomite amount of 20 wt% (SL80CD20) being cured in water has the 7, 28, and 365-day compressive strengths higher by 20.1, 29.1, and 33.3%, respectively as compared with those cured in air. Therefore, the water curing ensures that a sufficient water supplied is necessary for slag-dolomite hydration to take place and generate a greater amount of C-S-H gel to reduce porosity in order to reach a higher strength and better durability. From the microstructural observation, the identical hydration products were found for both the slag-dolomite binders which were cured either in air or water. The main hydration products of the hardened slag-dolomite binder with calcined dolomite amount of 20 wt% (SL80CD20) were the calcium silicate hydrate (C-S-H), Portlandite ($Ca(OH)_2$), silica (SiO_2), calcite ($CaCO_3$), and magnesium calcite ($Ca_{0.936}Mg_{0.064}CO_3$).

ACKNOWLEDGMENTS

The authors would like highly to acknowledge the financial aids from both the Ministry of Science and Technology through the grants of MOST 103-2221-E-011-078-MY3 and the National Taiwan University of Science and Technology (Taiwan Tech) to conduct this research work.

REFERENCES

ASTM C109/C109M-16a (2016), Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens), American Standard Testing and Materials, USA.

ASTM C125-16 (2016), Standard Terminology relating to Concrete and Concrete Aggregates, American Standard Testing and Materials, USA.

Chen, C.T., Nguyen, H.A., Chang, T.P., Yang, T.R., and Nguyen, T.D. (2015), Performance and microstructural examination on composition of hardened paste with no-cement SFC binder, Construction and Building Materials, 76, 264-272.

Dung, N.T., Chang, T.P., and Chen, C.T. (2014), Engineering and sulfate resistance properties of slag-CFBC fly ash paste and mortar, Construction and Building Materials, 63, 40-48.

Djayaprabha, H.S., Chang, T.P., Shih, J.Y., and Chen, C.T. (2017), Mechanical properties and microstructural analysis of slag based cementitious binder with calcined dolomite as an activator, 150, 345-354.

Gu, K., Jin, F., Al-Tabbaa, A., and Shi, B. (2014), Activation of ground granulated blast furnace slag by using calcined dolomite, Construction and Building and Materials, 68, 252-258.

Guneyisi, E. and Gesoglu, M. (2008), A study on durability properties of high-performance concretes incorporating high replacement levels of slag, Materials and Structures, 41, 479-493.

Jauffret, G., and Glasser, F.P. (2016), Thermally activated dolomite as pozzolanic addition to Portland cement, Advances in Cement Research, 28(6), 378-388.

Jeong, Y., Park H., Jun, Y., Jeong, J.H., and Oh, J.E. (2016), Influence of slag characteristics on strength development and reaction products in a CaO-activated slag system, Cement and Concrete Composite, 72, 155-167.

Jin, F., Gu, K., and Al-Tabbaa, A. (2015), Strength and hydration properties of reactive MgO-activated ground granulated blast furnace slag paste, Cement and Concrete Composite, 57, 8-16.

Kim, H.S., Park, J.W., An, Y.J., Bae, J.S., and Han, C. (2011), Activation of Ground Granulated Blast Furnace Slag Cement by Calcined Aluminate, Materials Transaction, 52, 210-218.

Kim, M.S., Jun, Y., Lee, C., and Oh, J.E. (2013), Use of CaO as an activator for producing a price-competitive non-cement structural binder using ground granulated blast furnace slag, Cement and Concrete Research, 54, 208-214.

Kumar, S., Kumar, R., Bandopadhyay, A., Alex, T.C., Kumar, B.R., Das, S.K., and Mehrotra, S.P. (2008), Mechanical activation of granulated blast furnace slag and its effect on the properties and structure of portland slag cement, Cement and Concrete Composite, 30, 679-685.

Kusbiantoro, A., Nuruddin, M. F., Qazi, S. and Shaiq, N., (2010), The Effect Of Sugar Based Retarder On Geopolymer Concrete In Different Curing Conditions, Malaysian Construction Research Journal, 7(2), 39-46.

Lothenbach, B., Scrivener, K., and Hooton, R.D. (2011), Supplementary cementitious materials. Cement and Concrete Research, 41 (12), 1244-1256.

McIntosh, R.M., Sharp, J.H., and Wilburn, F.W. (1990), The thermal decomposition of dolomite. Thermochima Acta, 165, 281-296.

Mindness S., Young, J.F., and Darwin, D. (2008), Concrete, 2nd edition, Pearson Education Taiwan, Ltd., Taiwan, 102-103 pp.

Mo, K.H., Alengaram, U.J., Jumaat, M.Z., and Yap, S.P. (2015), Feasibility study of high volume slag as cement replacement for sustainable structural lightweight oil palm shell concrete, Journal of Clean Production, 91, 297-304.

Olszak-Humienik, M. and Jablonski, M. (2015), Thermal behavior of natural dolomite, Journal of Thermal Analysis and Calorimetry, 119, 2239-2248.

Olszak-Humienik, M. and Mozejko, J. (1999), Kinetics of Thermal decomposition of dolomite, Journal of Thermal Analysis and Calorimetry 56, 829-833.

Palomo, A., Grutzeck, M.W., and Blancoa, M.T. (1999), Alkali-activated fly ashes a cement for the future. Cement and Concrete Research, 29, 1323-1329.

Yang, T.R., Chang, T.P., Chen. B.T., Shih, J.Y., and Lin, W.L. (2012), Effect of alkaline solutions on engineering properties of alkali-activated GGBFS paste, Journal of Marine Science and Technology, 20(3), 311-318.

Yang, W.H., Ryu, D.W., Park, D.C., Kim, W.J., and Seo, C.H. (2014), A study of the effect of light-burnt dolomite on the hydration of alkali-activated Portland blast-furnace slag cement, Construction and Building Materials, 57, 24-29.

FLEXURAL STRENGTH OF REINFORCED CONCRETE BEAM WITH CFRP

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Abstract
The flexural strengthening of reinforced concrete beam can be considered with applied externally bonded steel plate or carbon fibre composite. This external plate is bonded to the tension face of the concrete beam experimental study on reinforced concrete strengthening with Carbon Fibre Reinforced Plate (CFRP) has been conducted to estimate the effectiveness of using CFRP on the concrete structure as external reinforcement. Two beams were tested in this study to test the flexural strength effect of externally bonded CFRP composite in the reinforced concrete beam. Two beams were tested in this study to test the flexural strength effect of externally bonded CFRP composite in the reinforced concrete beam. One of the beams is a control beam and other beam is the concrete beam without steel bar and laminated CFRP to the bottom of the beam with epoxy. The dimension of the beams is 150x250 mm with length 2000 mm. All beams were tested using two-point loading to get pure bending in the middle span. The result of the experimental research showed that in spite of using laminated CFRP on the concrete beam had increased the cracking moment, but laminated CFRP on the concrete beams as external reinforcement without reinforcing steel bar has no significant effect on the behaviour of beams. Premature failure of externally bonded CFRP plates had occurred before the ultimate flexural capacity of the strengthened section is achieved.

Keywords: *Beam; concrete; Carbon Fibre Reinforced Plate; externally reinforcement bar*

INTRODUCTION

The value of the moment capacity (M) of the reinforced concrete beam (RC Beam) is determined by the distance between centre of concrete to centre of reinforcement bar *(jd)*. If the distance of *jd* is big then the value of moment capacity that occurs will be huge. In order for the moment capacity of the beam to work optimally, then the reinforcement bar must be placed at the bottom surface of the beam, in order to get the maximum value *jd*, therefore the steel reinforcement is placed at the outer tensile part of the beam where $jd2 > jd1$ so that M2 > M1 as shown in Figure 1. However, it does not provide sufficient attachment between steel and concrete in the beam, and the expected composite action cannot happen. Moreover, reinforcement bar is a material that is susceptible to corrosion if it is exposed to the air. Therefore, it required some minimum concrete cover as shown in Figure 1.

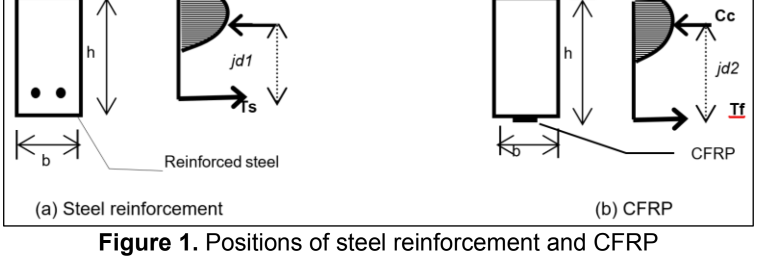


Figure 1. Positions of steel reinforcement and CFRP

Currently there are materials that offer some advantages that reinforced steel do not have, such as CFRP. It has a tensile strength which is higher than the tensile strength of the reinforcing steel, non-corrosive material, light less dense with weight over volume of 1.5 g / cm^3 and is easy to install at side. The bonding between the concrete and CFRP can be formed by using an adhesive material that provides adhesive between the two materials during loading condition. The use of adhesive material on the concrete structure by CFRP reinforcement will have a positive impact on the structure because CFRP placement of the concrete does not need to be embedded in concrete, because the material is corrosion resistant, it is made of non-metallic material. CFRP composite material is not required a cover, so *jd* distance is expected to be optimized to produce maximum bending moment capacity as well.

A summary of research in this area is available by Teng et al. (2002) explained the behaviour flexural and shear strengthening of beams, column strengthening flexural strengthening of slabs. Both the flexural and shear strengths of RC beam can be substantially increased using externally bonded FRP reinforcement in the forms of plates. The failures of such FRP strengthened RC beam often occur by the debonding of the FRP plate from the RC beam in a number of modes. Teng at al. (2009) studied the strengthening of RC structures with bonded FRP reinforcement, seismic retrofit of RC structures, durability of FRP strengthened in RC structures, hybrid FRP concrete structures and smart FRP structures. Brena et al. (2003) reported debonding of longitudinal carbon FRP (CFRP) sheets at deformation levels less than half the deformation capacity of control specimens. A similar result has been reported by Kuriger et al. (2001), Alessandra et al. (2001), DeLorenzis and Nanni (2001). Noorwirdawati et al. (2016) studied RC beams which strengthened in shear using CFRP strips with such scheme only focused on CFRP strips oriented at 45°/135°. The experimental result, beams wrapped with CFRP strips recorded shear capacity enhancement of around 19.05% to 43.74% as compared to the control specimens. Chin (2012) studied experimental and analytical numerical analysis of RC beams with large square openings placed in the shear region and strengthened by CFRP.

This research focuses on an experimental study of reinforced concrete beam by placing CFRP at a bottom of the beam and acting as an external reinforcement. The aims to determine the benefits of using CFRP as an external reinforcement of reinforced concrete beams, and to determine the flexural strength and moment capacity of RC Beam under two-point loads.

FLEXURAL STRENGTH OF CONCRETE BEAM

In this section, the moment capacity of the normal and CFRP beam are determined using simplified stress block method. Figure 2 shows the cross section of normal concrete beam together with the stress–strain relationship of the concrete and reinforcement bar. The moment capacity of this concrete beam is determined using simplified stress block method.

From Figure 2, the compressive force is given by Equation 1.

$$Cc = 0.85 f'c \,.\, a. \, b \qquad \text{where } a = \beta 1.c \tag{1}$$

The tension in steel is given by Equation 2.

$$Ts = As \,.\, fy \tag{2}$$

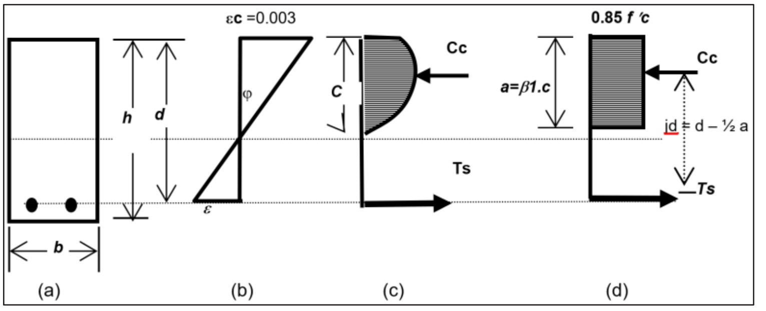


Figure 2. Distribution of stress–strain of normal beam

The equilibrium of force between the tension of steel and compression of concrete is given by Equation 3, 4 and 5.

$$Cc = Ts \tag{3}$$

$$0.85\, f'c\, .\, a\, .\, b = As\, .\, fy \tag{4}$$

$$a = \frac{As\, .\, fy}{0.85\, f'c\, .\, b} \tag{5}$$

The nominal moment capacity of the normal reinforced concrete beam is given by Equation 6.

$$Mn = As\, .\, fy\, .\, jd \qquad \text{where } jd = \text{d} - \tfrac{1}{2}\, \text{a} \tag{6}$$

When the reinforced concrete beam is mounted by the CFRP external reinforcement the bending strength of the beam occurs as proposed by Kuriger et al. (2001) is shown in Figure 3, where T_F is the tensile force of CFRP and jd_F is the distance from Cc to T_F.

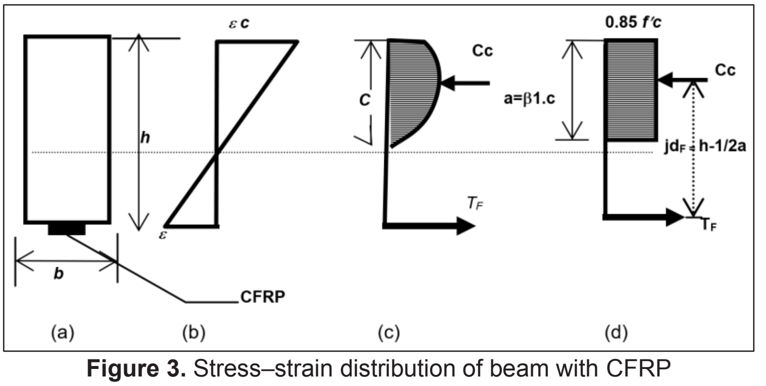


Figure 3. Stress–strain distribution of beam with CFRP

From Figure 3, the nominal moment of the cross section is given by Equation 7.

$$Mn_F = As_F\, .\, fy_F\, .\, jd_F \qquad \text{where } jd_F = \text{h} - \tfrac{1}{2}\, \text{a} \tag{7}$$

It can be seen that the moment capacity of beam with CFRP is bigger than normal beam due to the lever arm ($jd_F > jd$). Furthermore, the tensile strength of CFRP is higher than reinforcement bar. Based on Figures 2 and Figure 3 are seen that $jd_F > jd$ so that both equations (7) and (8) will produce a larger moment (Mn_F), or it can be said that with the installation of CFRP externally the bending capacity will increase proportionally.

CONCRETE COMPRESSIVE STRENGTH

The concrete compressive strength is determined by the highest voltage of the 150 mm and 300 mm diameter cylinder test pieces tested after 28 days; it was tested using a compression test machine until the specimen is destroyed. The amount of compressive strength of concrete is calculated using Equation 8.

$$\sigma_c = \frac{Pc}{Ac} \tag{8}$$

Where:

σ_c = concrete compressive strength
Pc = axial force
Ac = cross-sectional area

According to Park and Paulay (1975), the behaviour of concrete compressive strength can be illustrated by using the Hognestad stress-strain parabolic curve as shown in Figure 4, with the concrete compressive strength as follows:

$$fc = f'c\,(2\varepsilon c/\varepsilon o - (\varepsilon c/\varepsilon o)^2) \tag{9}$$

Where fc is the compressive stress of concrete, f'c is the maximum stress of concrete, and εo is the strain of concrete at the maximum compressive stress.

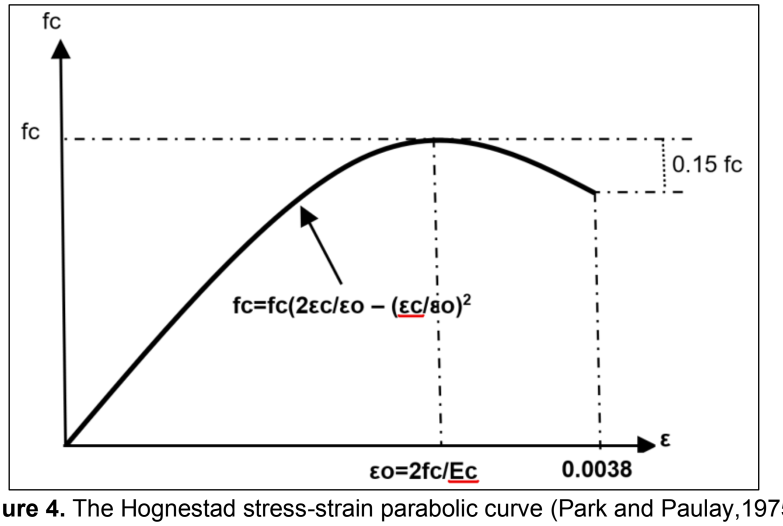


Figure 4. The Hognestad stress-strain parabolic curve (Park and Paulay,1975)

Properties of CFRP

CFRP is a flexible bending material and is mounted on the lower surface of the beam. The CFRP technical data specifications used is shown in Table 1.

Table 1. Properties of CFRP

Properties of CFRP	
Tensile Strength	2800 MPa
E-Modulus	165,000 MPa
ε_{cu}	> 1.7 %
Thickness / width	0.8 mm / 50 mm
Density	1.50 g/cm^3

Properties of Epoxy (Adhesives)

The usage of CFRP as an external reinforcement in a concrete structure requires a binder to obtain a composite action between the concrete and CFRP by epoxy adhesives that applied to the surface of concrete and CFRP surface evenly with thickness about 2 mm each side. Specification of epoxy technical data used is shown in Table 2.

Table 2. Properties of epoxy

Properties of Epoxy	
E-Modulus	12, 800 MPa
Bond Strength	> 4 MPa

FINDINGS FROM PREVIOUS RESEARCH

Nanni (2001) had compared the static behaviour of full-scale simply supported highway bridge deck panels which had strengthened in flexure using internally placed Near Surface Mounted (NSM) CFRP bars. Failure of the CFRP laminate reinforced deck spans was through a combination of rupture and peeling of the CFRP laminates. The NSM CFRP-reinforced span failed by tensile rupture of the CFRP bars. Relative to the capacity of an unstrengthened control deck, moment strength increases of 17 and 29% were reported for decks which retrofitted with externally bonded CFRP laminates and internally placed NSM CFRP bars, respectively.

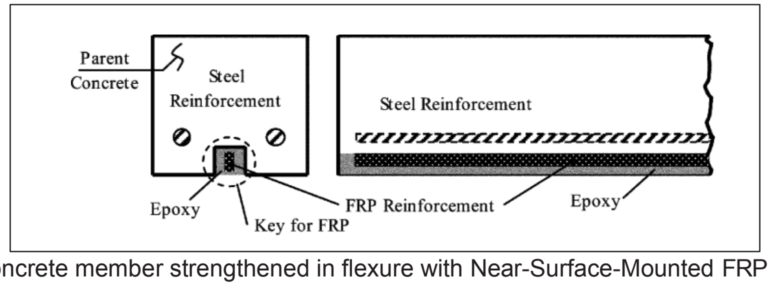


Figure 5. Concrete member strengthened in flexure with Near-Surface-Mounted FRP (Nanni, 2001)

DeLorenzis et al. (2001) had tested three steel-reinforced concrete T-beams strengthened in flexure with NSM glass FRP (GFRP) and NSM CFRP bars. The CFRP retrofitted beams experienced increases in strength of 30% (two No. 3 CFRP bars) and 44% (two No. 4 CFRP bars) over an unstrengthened control specimen. Both CFRP strengthened beams failed due to debonding of the NSM rods. The specimen strengthened with two No. 4 GFRP bars also failed due to debonding of the NSM GFRP bars at a load 26% higher than the control specimen.

Kuriger et al. (2001) reported the pattern of rupture in the beam structure given CFRP can be divided into three types: shear failure, debonding failure, and rupture of CFRP, as shown in Figure 6. Of the three types of failure it is desired that the collapse in CFRP rupture, because then all CFRP forces can work optimally. Research showed that flexure testing of CFRP on concrete beams would result in a reduction in strain of 11.5% to 58.6% in tensile reinforcement, and reduction of 3% to 33.5% concrete press strain and reduced deflection at Beam 8% to 53.1%. The type of failure that occurs is the shear failure in the concrete, and debonding on CFRP. Brena et al. (2003), experimental test had indicated that debonding of the bottom strip from the concrete surface is the mode of failure for concrete beams strengthened CFRP. Due to FRP debonding failures, the strength utilization ratio was sometimes only 15 - 35%, depending on the cause of debonding.

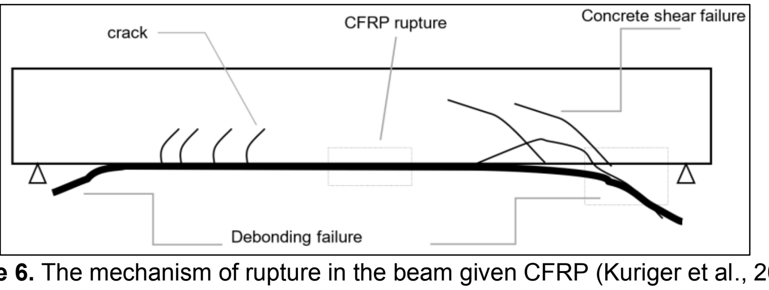


Figure 6. The mechanism of rupture in the beam given CFRP (Kuriger et al., 2001)

Aprile et al. (2001) reported that the CFRP plate attached to the bottom of the beam is calculated as a unity of structures that accepts the loads together. Such composite action can only occur because of the good bond between the two materials. The performance of bond is very important in delivering stress from concrete to CFRP. Reinforced concrete beam failure with CFRP plates always begins with debonding on the plate. Hedong and Zhishen (2005) reported that the debonding behaviour and failure mechanisms caused by different types of flexural crack distributions in FRP-strengthened R/C beams, which has not been solved so far. Using a discrete crack model for concrete crack propagation and a bilinear bond–slip relationship with softening behaviour to represent FRP–concrete interfacial behaviour, a nonlinear fracture mechanics-based finite-element analysis is performed to investigate the effects of crack spacing and interfacial parameters such as stiffness, local bond strength, and fracture energy on the initiation and propagation of the debonding and the structural performance. Hawileh et al. (2014) investigated the behaviour of reinforced concrete beams strengthened in flexure by means of different combinations of externally bonded hybrid Glass and Carbon Fibre Reinforced Polymer (GFRP/CFRP) sheets. The RC beams were tested under four point bending to study the flexural effectiveness of the proposed hybrid FRP sheets. It is observed that the increase in the load capacity of the strengthened beams ranged from 30% to 98% of the unstrengthened control RC beam depending on the combination of the CFRP/GFRP sheets.

RESEARCH METHODOLOGY

Figure 7 shows the reinforced concrete beam with simply beam is symmetrically loaded with two forces (P) as far as a from the supported, a pure bending state occurs where the constant moment in the region between the two loads P. All test beam had a shear span to steel reinforcement-depth ratio (a/d) ≥ 3. This ratio was intentionally designed so that ultimate strength would be controlled by flexural failure and not shear failure will occur.

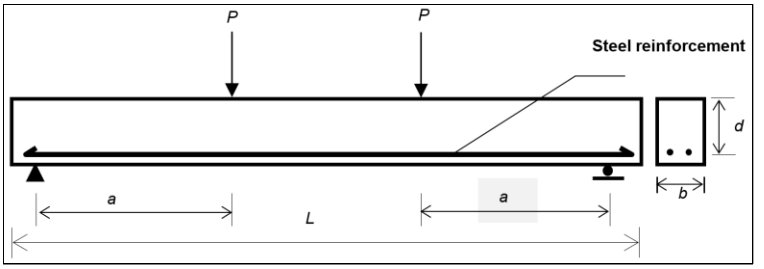


Figure 7. Experimental set-up in normal beam

The test specimens in this research were made two beams, such as the beam with steel reinforcement is called as normal beam (BN) and non-reinforced beam with CFRP as an external reinforcement is called as BF. Figure 8 shows the reinforcement detailing for BN and beam with CFRP (BF). The dimensions of concrete beams are 150 mm wide, 250 mm height and 2000 mm long. The static loading was represented by two points symmetrically with a distance of 600 mm between the point of loading and as far as 650 mm from each support.

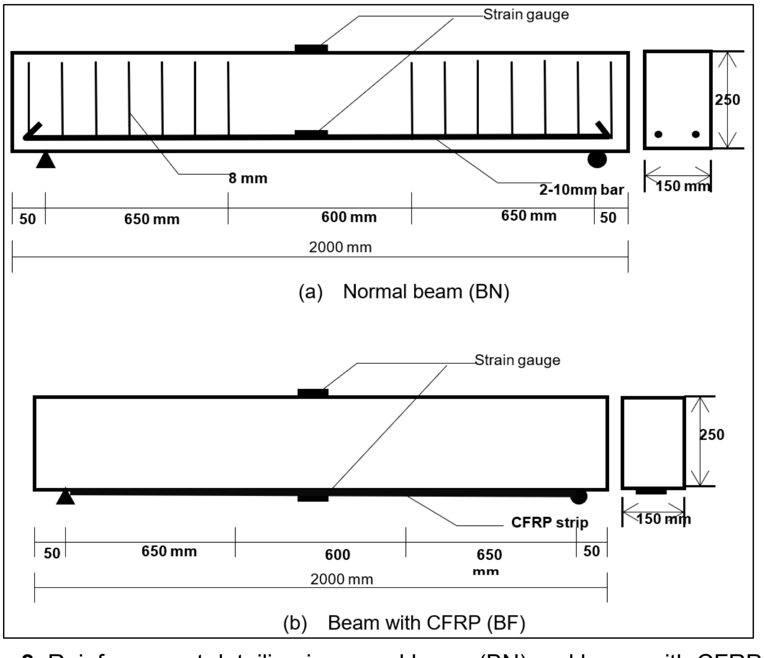


Figure 8. Reinforcement detailing in normal beam (BN) and beam with CFRP (BF)

The specimen was made from concrete with an average compressive strength (f'c) of 30 MPa. Two tensile reinforcement bars with diameter 10 mm are placed at a depth (d) = 203.5 mm of the beam. In order to measure the strain on the concrete of the specimens, a strain gauge was located on the top compression side of the centre span, to measure the tensile strain of reinforcement and CFRP one strain gauges was placed on their surface. Linear Vertical Displacement Transducers (LVDT) were installed to measure displacement that occurs at the centre of span test beam. Figure 9 displays the experimental set-up for the normal beam and beam with CFRP. The static load, deflection and strain data were recorded using the data logger. The applied loading was stopped if the specimen has collapsed and the data logger reading of the load from the load cell cannot increase anymore.

Figure 9. Experimental set-up for the normal beam and beam with CFRP

EXPERIMENTAL RESULT

Material Properties

The reinforcing steel used in this study is deform reinforcement with a diameter of 10 mm. The result of test of tensile strength of steel obtained the yield strength value of 340.50 MPa and the ultimate tensile strength of 454 MPa. Based on the results of the compressive strength test cylinder with size 150 mm x 30 mm, tested using Universal Testing Machine (UTM). The average compressive strength of the concrete obtained 32.55 MPa greater than the compressive strength of concrete designed of 30 MPa.

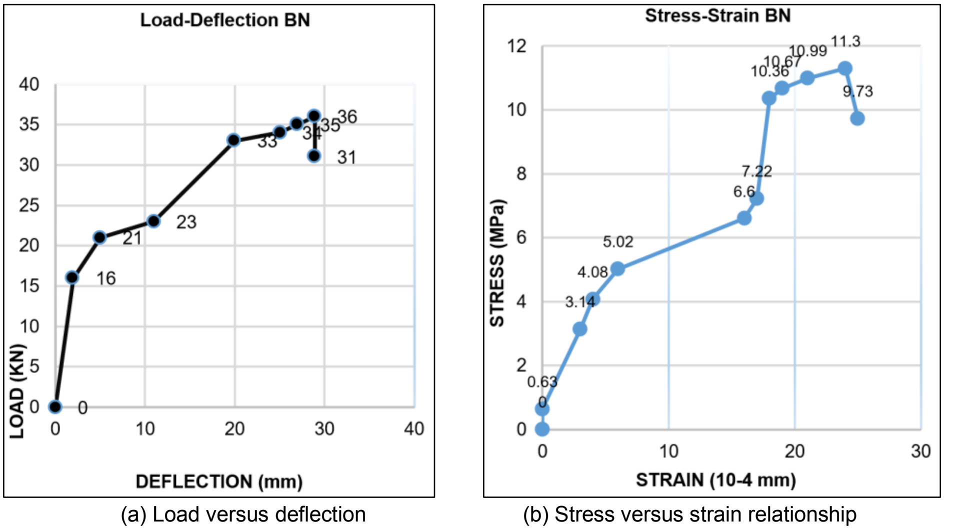
Table 3. Average compressive strength of concrete

Specimen	Average Compress Concrete (MPa)
BN	34.0
BF	31.1

Flexural Strength for Normal Beam

The first crack was occurred 16 kN where the concrete entering the cracked stage and passed through the limit of its tensile strain. The tensile force that appear is bonded by the reinforcement bar with the appearance of cracks propagation in the bending area of the beam, shown at Figure 10(a) shows the load versus deflection and Figure 10(b) shows the stress versus strain relationship of BN.

Initially, the specimen test section is uncracked zone until 16 kN as shown in Figure 9(a). The cracking load P_{cr}, behaviour changes from uncracked to cracked-elastic zone. The cracked load occurs with a very small increment of deflection, meaning to say the load-deflection response in the BN beam shows rigidity of concrete beam. The strain increase in steel reinforcement at the beginning of loading to 16 kN and proving that in under uncracked stage conditions the concrete is still able to hold up the tensile forces, so the role of reinforcement bar still carrying the tensile force but with small force. As the load is increase further, the section responded elastically until the yield strength of steel reinforcement (fy) is reached. The steel yield fy occur at 16 kN corresponds to a flattening of the load-deflection trace and stress-strain response as shown in Figure 10. When the load is increased to 36 kN, the ultimate vertical load (P_{max}) occurred by concrete crushing, and the deflection is stopped at 28.9 mm as shown in Figure 11. There were shear cracks and flexure cracks at the both of point load the compression side concrete collapse in horizontal cracking.



(a) Load versus deflection (b) Stress versus strain relationship

Figure 10. Load–deflection and stress–strain for normal beam (BN)

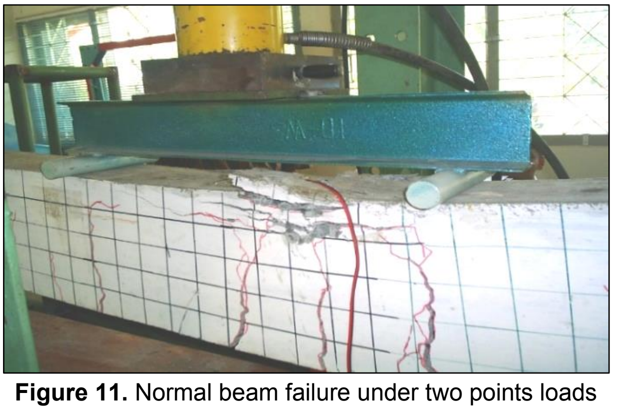
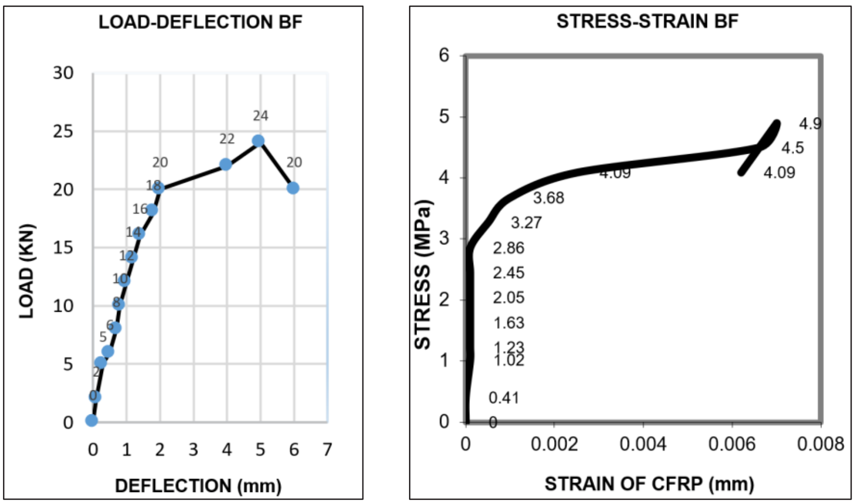


Figure 11. Normal beam failure under two points loads

Flexural Strength Beam with CFRP (BF)

The first crack is occurred at 18 kN which indicated that the concrete entering the cracked stage, passed through the limit of its tensile strain (uncracked condition). The tensile force in the beam with CFRP carried more load, it is signed with the appearance of cracks in the bending area of the beam.

Figure 12 shows that at 14 kN deflection of concrete beam occurs very small increment where the static response in the BF beam shows greater stiffness then BN beam. The load has reached maximum load at 24 kN and then decrease back until 20 kN. The maximum deflection of concrete beam has reached at 6 mm and less ductile as compare with BN beam. After the load of 18 kN, the CFRP strain has grown larger which proves that the tensile force is fully borne by CFRP. This situation lasts until the load is almost 24 kN as shown in Figure 12. The stress-strain relationship shows there is changing from the elastic zone to plastic zone. At a load of 24 kN, CFRP is debonded at one end side, which is indicated by a reversed graph toward zero, followed by collapse of the beam. This indicates that when CFRP was separated from concrete, the beam where adhesive does not have enough capability to borne tensile force so that the beam ruptures as shown in Figure 13.



(a) Load versus deflection (b) Stress-strain relation

Figure 12. Load–deflection and stress–strain relationship for beam with CFRP (BF)

Figure 13. Beam with CFRP failed through debonding mechanism

Comparison of Moment Capacity between Normal Beam and Beam with CFRP

From experimental results, the maximum load of the normal concrete beam (BN) reached 36 kN and BF with CFRP reached 24 kN and then these loads are used to calculate the ultimate bending moment that occurs as tabulated in Table 4.

Tabel 4. Comparison of Ultimate Moment BN and BF

Specimen	Maximum Load (kN)		Maximum Bending (kNm)		% Different
	Experiment	Theory	Experiment	Theory	
BN	36	31	11.70	10.07	116 %
BF	24	82.5	7.80	26.11	29 %

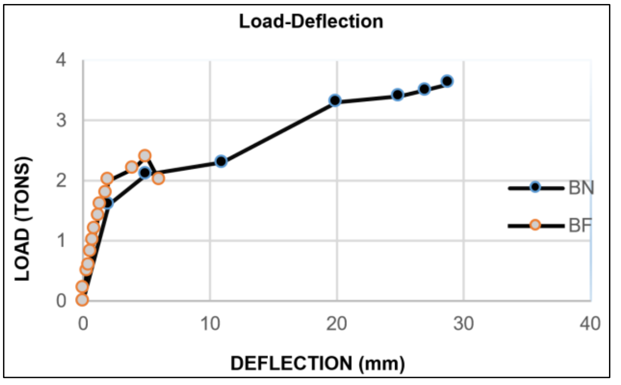


Figure 14. Comparison between load versus deflection between BN and BF

Failure Mode

The normal concrete beam (BN) had few flexural cracks pattern with large width due to concrete compressive crushing after steel yielding as shown in Figure 15. Concrete cracking was identified in specimens at a load of about 16 kN, then propagated towards to up, after which the steel reinforcement maintains an almost linear strain increase until yielding, and until load at 36 kN then concrete crushing.

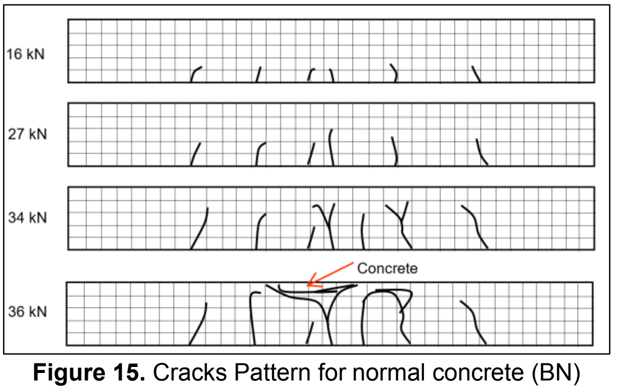


Figure 15. Cracks Pattern for normal concrete (BN)

For concrete beam with CFRP (BF), the cracks had initiated in the pure bending region of the beam at a load of 18 kN, then the cracks increased and propagated towards to up. At the load of 24 kN there was a debonding failure on CFRP had initiated at one of the support beams accompanied by a fracture. Due to the release of CFRP, the vertical crack in the concrete beam was not inhibited, so it propagated very fast towards to up and caused the concrete beam was broken, as shown in Figure 13. The cracks pattern of the normal concrete beam BN as shown in Figure 16.

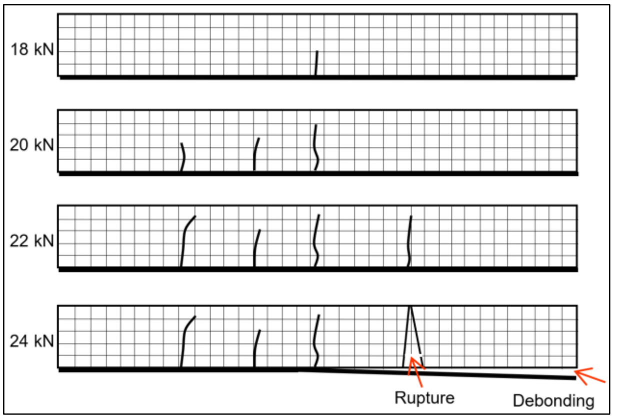


Figure 16. Cracks Pattern of BF

Effectiveness of CFRP

Based on the results from experimental work, beam with CFRP located externally show not a good performance as compared to normal beam (BN). As the load on the BF beam reaches ultimate, the CFRP strain occurs at 0.0070, thus the working tensile strength of 812.5 MPa or only 30% of maximum strength is 2800 MPa, because of debonding on CFRP before CFRP works optimally in increasing its moment capacity. Normally, CFRP is used to repaired and retrofitted the existing structure which suffered severe damage under earthquake or impact load.

DISCUSSION

The role of bond is very important in forming a composite action between concrete and CFRP. Bonds between concrete and CFRP were used based as the adhesive properties is shown in Table 2 where the bonding strength is only more than 4 MPa, the expected composite structure does not occur. Furthermore, the modulus of elasticity of epoxy used is E = 12000 MPa which is smaller than the modulus of elasticity of concrete which is 20000 MPa. There is very weak bonding between epoxy resin with the concrete surface which required better bonding between them.

The contact surface between concrete and CFRP is less widespread because it occurs on one side of the surface and not between steel reinforcement and concrete with less contact areas on the entire surface of the reinforcement. Thus, the bond required by CFRP to be a composite unit with concrete becomes less perfect and should provide better bonding for future research.

CONCLUSIONS

The following conclusions can be drawn based on the experimental and observation:

1) The position of CFRP plates as external reinforcement on the tensile fibre of the beam may inhibit the appearance of first crack of the beam as an indication of concrete tensile strain achieved. The ability of the beam to hold the load until the initial crack increase on the BF beam by 12.5% as compare with the BN beam.

2) By putting the CFRP plates as external reinforcement (BF beam) is less effective, because the bending strength decreases by 33.3% and the deflection decreases 79% as compared with BN beam. This is because the debonding failure of CFRP of the concrete beam is not able to hold the pull force that occurs, consequently beam collapses and failed due to brittle condition.
3) The collapse occurred on the test beam with the external reinforcement of CFRP is the occurrence of debonding failure such as loose bond between the concrete with CFRP and it can be said that the composite material has not been able to work optimally under static loading.

REFERENCES

ACI Committee 440 (2006). *Report on Fibre-Reinforced Polymer (FRP) Reinforcement for Concrete Structures*. Farmington Hills, USA: American Concrete Institute.

ACI Committee 440 (2008). *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*. Farmington Hills, USA: American Concrete Institute.

Aprile, Alessandra; Spacone, Enrico; Limkatanyu, Suchart, 2001, *Role of Bond in RC Beams Strengthened with Steel and FRP Plates*, Journal of Structural Engineering, December 2001, page 1445 – 1452.

Brena, S. F.; Bramblett, R. M.; Wood, S. L.; and Kreger, M. E., 2003, *Increasing Flexural Capacity of Reinforced Concrete Beams Using Carbon Fiber-Reinforced Polymer Composites*, ACI Structural Journal, V. 100, No. 1, Jan.-Feb., pp. 36-46.

Chin, SC; Shafiq, N; Nuruddin, MF, 2012, *Strengthening of RC Beams with Large Openings in Shear by CFRP Laminate: Experiment and 2D Nonlinier Finite Elemen Analysis*, Research Journal of Applied Science, Engineering and Technology, V.4, No 9, Maxwell Science Publishing.

Carl W. Niemitz; Ryan James; Sergio F Brena, 2010, *Experimental Behaviour of CFRP Sheet Attached to Concrete Surface Using CFRP Anchor*, Journal of Composite for Construction, V 14, issue 2, April 2010.

DeLorenzis, Laura dan Nanni, Antonio, 2001, *Characterization of FRP Rods as Near Surface Mounted Reinforcement,* Journal of Composite for Construction, May 2001, page 114 – 121.

DeLorenzis, L., and Nanni, A., 2002, *Bond between Near-Surface Mounted Fiber-Reinforced Polymer Rods and Concrete in Structural Strengthening*, ACI Structural Journal, V. 99, No. 2, Mar.-Apr., pp. 123-132.

Harmon, Thomas; Kim, Yoo, John; Kardos; Johnson, Timothy; Stark,Andrew, 2003, *Bond of Surface–Mounted Fiber Reinforced Polymer Reinforcement for Concrete Structures*, ACI Structural Journal, V.100, No. 5, September – October 2003, page 557 – 564.

Hawileh R, Abdalla JA, Tamimi A., (2011), *Flexural Performance of Strengthened RC Beams with CFRP Laminates Subjected to Cyclic Loading*. Key Engineering Materials 2011; 471-472: 697-702.

Hawileh, R. A., Rasheed, H. A., Abdalla, J. A., Al-Tamimi, A. K. (2014). *Behaviour of Reinforced Concrete Beams Strengthened with Externally Bonded Hybrid Fiber Reinforced Polymer Systems*. Materials and Design, 53, 972-982. Retrieved from http://krex.ksu.ed

Kuriger,Rex ; Sargand,Shad; Ball, Ryan and Alam, Khairul, 2001, *Analysis of Composite Reinforced Concrete Beams*, Department of Mecahanical Engineering, Ohio University.

Masood Rafi, Muhammad; Najai Ali and Faris Ali, 2008, Aspect of Behaviour of CFRP Reinforced Concrete Beams in Bending, Construction and Building Materials, V 22, Issue 3, March 2008, page 277 – 285.

Niu, Hedong; Wu Zhinshen., 2005, *Numerical Analysis of Debonding Mecanisms in FRP Strenghtened RC Beam*, Computer-Aided Civil and Infrastructure Engineering, Vol 20, issue 5, September 2015, page 354 – 368

Noorwirdawati Ali, Abdul Aziz Abdul Samad, J. Jayaprakash, Noridah Mohamad and Shahiron Shahid., 2016, *Pre-cracked RC Continuous Beam Strengthened in Shear Using CFRP Strips Oriented at 45°/135°*, Malaysian Construction Research Journal (MCRJ), V 20, No. 3, page 15 – 28.

Tavares, D.H; Giongo, J.S; and Paultre, P, 2008, *Behaviour of Reinforced Concrete Beams Reinforced with GFRP Bars*, Ibracon Structures and Materials Journal, V 1, N0 3, September 2008, p 285-295.

Teng, J. G.; Chen, J. F.; Smith, S. T.; and Lam, L., 2002, *FRP-Strengthened RC Structures*, John Wiley & Sons, West Sussex, UK, 266 pp.

Teng, J. G., Lam, L., Chen, J. F., Dai, J. G. & Yu, T. 2009, *FRP Composites in Structures: Some Recent Research*. Proceedings of the Second Asia-Pacific Conference on FRP in Structures (APFIS) (pp. 179-186). Korea International Institute for FRP in construction.

Vanden, Einde; Zao, L; Seibe, F; 2003, *Use of FRP Composite of Civil Structural Application*, Construction and Building Materials, V 17, 2003, page 389 – 403.

LIMITATIONS OF RISK IDENTIFICATION TOOLS APPLIED IN PROJECT MANAGEMENT IN THE NIGERIAN CONSTRUCTION INDUSTRY

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Abstract
The success of risk management (RM) relies significantly on front-end processes such as risk identification (RI). However, dimensions of procedural issues in RM performance have witnessed limited research attention across project management literature. This study examined RI practices adopted by project managers in the construction industry to unravel weaknesses in applied tools and their manifest problems. Explorative study involving mixed research strategy was conducted. The sample frame involved certified project managers in the Nigerian construction industry. Data analysis involved Conversational Analysis, Mean Item Score and one-sample Chi-Square tests. The findings revealed that increased awareness about risk consequences and their management tools have not assisted to improve RI by construction stakeholders in the study area. Applied RI tools are till date limited to intuitive-subjective assessment developed mainly from experts' assumption. These tools are inherently ripped with many defects including inability to mapped holistic risks, inhibition of interdisciplinary actions, and exclusion of relevant stakeholders. These lapses further trigger poor RM performance by promoting boxed opinion, overture of one professional's experience above others, biases, and inhibit knowledge dissemination. Concerted effort is therefore required to re-invigorate applied RI practices to ensure collaboration among stakeholders and quantitative appraisal of mitigation strategies for improved project risk management.

Keywords: *Construction; identification; limitation; weaknesses; risk; and risk performance*

INTRODUCTION

It has become the norm of the construction industry that risks cannot be separated from construction project management. Although, this postulation has helped to advance the high-risk nature of construction projects delivery (Zwikael & Ahn, 2011), expanding volume of literatures only reiterates that, risks often result in negative outcomes. Stakeholders' thinking therefore portrays risk as threats to project objectives (Association of Project Managers, [APM], 2012). Risk means events with detrimental outcome to project objectives. Addressing risks to deliver project objectives is therefore not optional but the main step to achieve successful project outcome (Kotb & Ghattas, 2017). Risk management involves multi-faceted processes encompassing: risk identification; assessment; quantification; planning; monitoring; and control (Odeyinka & Dada, 2016). Risk identification is defined as the 'systematic check of all the aspects of the work in order to decide causes of threat (Liaudanskiene & Ustinochius, 2010; Tipili & Yakubu, 2016). The objective is to assess and analyse extant threats in a project system with a view to develop mitigation strategies to prevent and manage occurrences. The task enlists numerous tools and standards ranging from simple intuition-based techniques to specialised application packages. The uses of these tools have not assisted to mitigate the increasing cases of cost overrun and other consequences of risk in construction project delivery (Amadi & Higham, 2016). Kotb and Ghattas (2017) categorically observed that, risk management tools have failed to enhance effective risk identification. However, limited literature narrative exists about the weaknesses of these tools that trigger inefficiency in project risk management.

Liaudanskiene and Ustinochius (2010) buttressed the need for risk management tool to identify appropriate risks without creating additional problems in the project system. The choice of risk assessment tool is therefore an important step in achieving successful risk management. Even though numerous studies have considered risk management practices across the global spread, persistence of risk threats on project performance portrays dearth of knowledge in certain perspectives (Van, Bowen & Akintoye, 2008; Amadi & Higham, 2016). Studies by Tsiga et al. (2017) and Kotb and Ghattas (2017) revealed that, risk management in construction project management has not improved significantly over the years, thereby pointing to imminent literature gap. One aspect that has seen limited research interests is procedural factors contributing to abysmal application/performance of risk management in construction project management.

Elfaki and Alatawi (2015) linked persistent poor performance in project delivery to the failure of stakeholders to learn and document inputs and challenges in implementing project system's processes. Rather, expanding research interests tend to consider only existing practices including assessment of key risks (Nnadi & Ugwu, 2013; Ogunbayo, 2014; & Tipili & Yakubu, 2016). The overarching finding in these studies suggests that, the overall performance of risk management is still a significant low. This suggests ineffective utilisation of risk management tools, and that, applied tools have failed to achieve targeted performance. Previous studies have not been able to state categorically the prevalent risk analysis tool along regional context, and why a number of tools have not gained widespread adoption in the construction industry.

This study postulates that, evaluating obtainable practices along regional context and their limitations will espouse defects of relevant tools and provide decision criteria for selecting appropriate risk management tools for adoption. Kotb and Ghattas (2017) posited that, persistent failings in project performance are unrelated with non-implementation of risk management, rather, that applied tools are unable to identify risk effectively. This study therefore evaluates risk identification tools adopted by project managers in the Nigerian construction industry, and their inherent weaknesses as correlates of ineffective risk management performance in construction engineering project management.

LITERATURE REVIEW AND PREVIOUS STUDIES

Overview of Risk Management Research in the Construction Industry

Farrokshand et al. (2016) classified existing researches in construction risk management literatures into two, using mainly, the management approach. The first conceived risk as an object (quantifiable, assessable, capable of being modelled and manageable) (Adedokun et al., 2011; Leslie & Lix, 2014). The second view sees risks as subjective and constructed social concept (Pott, 2008). Risk as an objective construct advocates quantitative assessment of threat events to determine their frequency and impact dimensions. The subjective approach involves mainly intuition and experience in place of formal risk management techniques. The quantitative approach, according to Zhang (2011), enlists efforts aimed at data collection and quantification of risk events with a view to identify, explain, forecast, and control them.

Inclusive discussions have also emerged about which approach is more beneficial and appropriate. The leading argument against the objective approach is that, it enlists standardized protocol and advanced tools which are difficult to implement across board (Zhang, 2011). Stalker (2003) contends that, the objective approach is value-neutral since it separates risk management practice from practitioner's mind and value system. To address these lacunas, the subjective approach has been increasingly recommended. Farrokshand et al. (2016) maintained that, subjective risk management is more appropriate because, people perceptions about risk event, as well as sense making, and management vary. This means that, the interface between people and events and their emotive respond impact how risks are perceived. This conception is also deemed to affect the risk management approach adopted.

RISK IDENTIFICATION TOOLS APPLIED IN THE CONSTRUCTION INDUSTRY

It is sometimes difficult to differentiate risk identification from risk analysis, notably when it comes to tools applied. As a result, most literatures are unable to treat comprehensively aggregated processes involved in risk management. The development can be attributed to the varying conventions and frameworks applied in risk management e.g. PMI and Prince2. Potts (2008) for instance, adopted cost management perspective to illustrate how risk analysis can be linked to cost implications. However, the basic difference between risk identification and analysis is that risk analysis quantifies the effect of identified risk on project objective in terms of cost, time and quality. Liaudanskiene and Ustinochius (2010) surveyed the application of risk assessment tools in the Lithuanian construction industry. Although, the actual level of use of these tools is not explicit from the study, five tools including: What-if-Method, Failure Mode and Effects Analysis (FMEA), Fault Tree Analysis Delphi-method and other specially organised methods were reviewed. A study by Wood and Ellis (2003) surveyed risk management practices of leading UK cost consultants and revealed that risk workshop and risk register were ubiquitous, while Monte Carlos simulation was prevalent based on the need to improve level of confidence. In a study of risk management practices applied by construction companies in Nigeria, qualitative analysis, consulting experts, decision analysis, intuitive approach, computer modelling and sensitivity received priority ranking (Ejohwomu, 2014). In the referenced study, risk premium (allocation of contingency) surprisingly received a low rating when it is actually, the most prevalent method used. A study by Ogunbayo (2014) also validated that, the use of qualitative risk management tool was prevalent and recommended synergistic application of both quantitative and qualitative risk management tools for optimal results.

Odenyinka and Dada (2016) explained that qualitative risk management simply ascribed risk identification using pure descriptive perception of their features and impact on project delivery. Qualitative tools consider the two dimensionalities of risk (Lu & Yan, 2013). The likelihood of occurrence and the probability of impact (P x I) underscore the fundamental principles of qualitative risk analysis. It could also be achieved using ISO risk curves and risk register (Odeyinka & Dada, 2016). However, the severity of risk event is predicated on the stakeholders' perception and judgment (Ogunbayo, 2014; Tipili & Yakubu, 2016). Qualitative risk identification tools include brainstorming, causes and effect diagram or Ishikawa diagram and checklist (Goh & Abdul-Rahman, 2013; Mahendra et al., 2013). Quantitative risk identification on the other hand, enlists numerical procedure in evaluating the impact of risks in the project. The approach quantifies likelihood of occurrence and evaluates impact on cost, schedule and quality including objectives (PMI, 2013). Walke and Topkar (2012) reprised that, quantitative approach is more accurate than the qualitative counterpart. Quantitative risk management tools include sensitivity analysis, probabilistic sums, expected monetary value, Monte Carlo Simulations, decision tree analysis, event and fault trees, Multi-criteria method, Fuzzy Logic and system dynamic (Walke & Topkar, 2012; PMI, 2013). Kotb and Ghattas (2017) classified risk management tools into assumption and constraint analysis tools, information gathering techniques, and diagramming techniques. Table 1 however presents a comprehensive list of requisite tools applied in project management.

METHODOLOGY

The study involved explorative research using mainly survey design. Mixed data collection approaches were adopted to generate requisite data to achieve the research question. Forty-two (42) and 91 certified project managers in the Nigerian construction industry were engaged in qualitative interviewing and industry-wide survey in a two-phase research. The mixed approach was adopted to facilitate data collection and to triangulate result obtained in one phase against another. The study enlisted two principal characteristics from the targeted population, mainly certification in project management and experiences in project delivery. The sample frame obtained through a preliminary investigation revealed there are 3,932 certified project managers within the built environment in Nigeria. Kish's (1965) formula was used to obtain the sample size of 91. The interview enlisted both open and structure approaches while the main research instrument -questionnaire consists of six questions in two sections. The first section elicited respondents background information such as professions, years of experience, and scope and quantity of project undertaken. The second section asked questions about the level of use of risk management tools, perception of inherent weakness and problems encountered using them. Data analysis enlisted conversional analysis, percentages, mean item score and Chi-Square analysis. Chi-Square was used to determine the level of association of inherent weakness in applied risk management tool as correlates of ineffective risk management performance. Reliability tests were conducted on the obtained data using Lee Cronbach Alpha tests, the result showed that the measurement variables and study instrument is consistent and coherent with values between 0.81-0.87.

Table 1. Summary of Risk Analysis Tools Applied in the Construction Industry

S/N	Tools	Description	S/N	Tools	Description
1	Assumptions and Constraints Analysis	Forecast expectations and their hindrances and determine them to be true or false (a)	12	Nominal Group Technique	Brainstorming-like session where participants deliberate issues before evaluation (a)
2	Document Reviews	Review of contract documents to uncover lacunas for making claims (b)	13	Cause and Effect (Ishikawa) Diagrams	Analyse effect of risk into main causes, sub-causes for easy understanding (a; d)
3	Checklists	Prepared by specialists from failures in past projects (a; c)	14	System Dynamic (Process Flowchart)	Modelling tool used to use to pattern, discern and debate a complex system (a; d)
4	Brainstorming	Typical risk management workshop session involving, open frank and in-depth discussion (c; a)	15	Influence Diagrams	Graphical and mathematical modelling of projects showing outcome of course of action (f; g)
5	Delphi techniques	Anonymous survey of expert opinion based on area of expertise (c; a)	16	Failure Modes and Effects Analysis (FMEA) – Fault Tree Analysis	Review project system capable of causing failure, their causes effects and causes (a; g)
6	Interviews	Face-to-face interaction with key stakeholders to uncover latent threats in projects (c; a)	17	Force Field Analysis	Determine forces driving change and forces against change in the project system (a)
7	Root-Cause Analysis	Identify actual sources of risk using basic symptoms of original forces (a, c. d)	18	Sensitivity Analysis	Determine how distinct values of independent variable impacts dependent variable under specified condition (h)
8	SWOT Analysis	Focus on threats inhibiting project potentials (a; e)	19	Monte Carlo Simulations	Used to approximate the probability of certain outcomes after multiple trials (h)
9	WBS Review	Identify risk using other techniques from WBS framework (a; e)	20	Expected Monetary Value	Weighs the probability of each outcome and determines the worth in monetary terms (i).
10	Questionnaire	Survey using checklist from literature and analysed to determine criticality (c; a)	21	Fuzzy Logic	Use to account for uncertainty in qualitative scenario by transforming fuzzy to standard set
11	Prompts Lists (Risk Categorisation Lists) e.g. PESTLE, TECOP & SPECTRUM	High level risk assessment using standard archetypes e.g. SLEEPT (a)	22	Multi-criteria Decision Tool	Operation research-based tool that appraise explicitly multiple standards in decision domains (d; i)

a = Kotb and Ghattas (2017); b = Kumar, Chandrashekhar and Singh (2017); c = Nnadi and Ugwu (2013); d = Ogunbayo (2014); e = Renault and Agumba (2016); f = PMI (2013); g = Ogwueleka and Mendie (2014); h = Touran (2006); i = Pott (2008); j= Salawu and Abdullah (2015)

FINDINGS AND DISCUSSIONS

Table 2 shows interviewees' profile. The sample consists of certified project managers. Project managers with dual professional roles that is, project management and their disciplines are mainly consultants. Core project managers that is, professional performing project management functions only constitute 19% of the sample. Another strata shows that project engineers and construction managers constitute 24% of the sample and operate mainly in the contracting sector.

Table 2. Interviewees' Background

Interviewees' Background and Designation	Sector	Number
Quantity Surveyors (Cost and Contract Advisors)	Consultancy	16(38%)
Architects (Design Consultant)	Consultancy	8(19%)
Project Managers	Consultancy	8(19%)
Construction Managers	Contracting	6(14%)
Project Engineers	Contracting	4(10%)

Table 3 presents background information relating to the quantitative phase of the study. Majority of project managers in the study's sample practiced project management alongside their respective professions. This indicates that, the trend has not changed significantly since Odusami et al. (2003) first reported the development. The proportion of respondents who specialises in project management only transit from allied professions to the built environment sector. Project management experiences of respondents have also reach deep with majority exceeding ten years in the industry. The respondents also have relevant project management skills based on their certification status with PMP and Prince2. The curricula of these programs significantly address key project management knowledge areas including risk management. The sample respondents are therefore deemed to understand the theoretic about the subject of research.

Table 3. Respondents Profile in Quantitative Phase

Practice Areas			Experience in Pjt. Mgt.			Qualification		
Variables	N	%	Variables	N	%	Variables	N	%
PM only	15	19	5-10years	63	21	PMP	70	88
PM + Others	65	81	10years & above	37	26	Prince2	10	12
Total	**80**	**100**	**Total**	**95**	**100**	**Total**	**95**	**100**

Table 4 shows results relating to actual risk management practice of the respondents. The study evaluated the depth of risk management experience (level of use during project implementation) and awareness about applied tools. The results in Table 4 indicate significant variation in stakeholders' level of knowledge and actual application in practice. Stakeholders demonstrate significant level of awareness about applied tools, but this awareness is unrelated to actual practice. Risk premium, assumption and constraint analysis, WBS review, document reviews, Delphi techniques and checklists are the top-ranking applied risk identification tools in the study area. The vastly applied tools are also the most familiar tools. The mean item score relating to level of use lies between 1.90 to 4.32 and averaged score of 3.11. This is not to say that, all the listed tools are being used by stakeholders in the study environment. Level of use and level of awareness are valid with mean item score greater 3.0.

Table 4. Level of Awareness and Use of Risk Identification Tools

S/N	Sub-Variables	Level of Use			Level of Awareness		
		MIS	Rank	Decision	MIS	Rank	Decision
1	Risk premium allowance	4.32	1st	Sig.	4.80	1st	Sig.
2	Assumptions and Constraints Analysis	3.84	2nd	Sig.	4.34	2nd	Sig.
3	WBS Review	3.72	3rd	Sig.	4.22	3rd	Sig.
4	Document Reviews	3.60	4th	Sig.	4.20	4th	Sig.
5	Delphi techniques	3.54	5th	Sig.	4.18	5th	Sig.
6	Checklists	3.40	6th	Sig.	4.15	6th	Sig.
7	Interviews	3.00	7th	Sig.	4.14	7th	Sig.
8	Brainstorming	2.87	9th	Sig.	4.10	8th	Sig.
9	SWOT Analysis	2.76	10th	Sig.	4.09	9th	Sig.
10	Questionnaire	2.50	12th	Sig.	4.07	10th	Sig.
11	Prompts Lists e.g. SLEEPT	2.88	8th	Sig.	3.89	11th	Sig.
12	Nominal Group Technique	2.65	11th	Sig.	3.80	12th	Sig.
13	Ishikawa Diagrams	2.20	13th	Not Sig.	3.79	13th	Sig.
14	System Dynamic (Process Flowchart)	2.00	21st	Not Sig.	3.68	14th	Sig.
15	Influence Diagrams	2.08	15th	Not Sig.	3.65	15th	Sig.
16	Failure Modes and Effects Analysis (FMEA)	2.02	20th	Not Sig.	3.64	16th	Sig.
17	Force Field Analysis	2.03	19th	Not Sig.	3.62	17th	Sig.
18	Sensitivity Analysis	2.07	18th	Not Sig.	3.60	18th	Sig.
19	Monte Carlo Simulations	2.08	17th	Not Sig.	3.58	19th	Sig.
20	Root-Cause Analysis	2.10	16th	Not Sig.	3.56	20th	Sig.
21	Expected Monetary Value	2.11	14th	Not Sig.	3.55	21st	Sig.
22	Fuzzy Logic	1.87	22nd	Not Sig.	3.54	22nd	Sig.
23	Multi-criteria Decision Tool	1.90	23rd	Not Sig.	3.51	23rd	Sig.

Sig. = significant

In fact, 48% of the listed tools obtained mean item score below 2.20. On the other hand, the level of awareness about the applicability of the listed tools is however high. The mean item score lies between 3.51 and 4.80, with an average score of 4.16. This is to say that, the knowledge about the existence and applicability of these tools in risk identification is vastly embedded among construction stakeholders. Results from interview (Table 5) however, corroborate survey findings. The interview result revealed that, the prevalent risk identification tool is based on assumption and intuitive-based judgment using mainly the professionals' experience. This practice gives rise to the proliferation of contingency allocation in time and cost to address imminent threats in project development. The implication is that, risk identification is not necessarily conducted in-depth, rather risks are conceived as a static process across projects and their mitigation could be achieved within a defined threshold of extra fund and time.

Table 5. Risk Identification Tools (Results of Interview)

S/N	Prevalent Risk Identification Tools
1	Risk Premium
2	Intuition-based judgment
3	Peculiar elements by element review
4	Checklists
5	Assumption and constraint analysis using past experience

Table 6 presents interview and survey results relating to inherent weaknesses in risk identification tools as correlates of ineffective risk management. Interviewees were asked to identify lacunas in the relevant tools with high awareness score. The weaknesses identified were employed in the quantitative phase to elicit effect of tools' weaknesses on ineffective risk management performance using five-point Likert scale. One sample Chi-Square test was conducted to determine whether tools' defects are associated with ineffective risk management. The result indicates that, each tool has certain degree of defect(s) that inhibits project risk management performance. Seventeen tools examined obtained p-values less than the critical p-value of 0.05 and are were validated significant limitation of effective risk management practices in the construction industry.

Table 6. Weakness in Risk Identification as Correlates of Ineffective Risk Management

S/N	Sub-Variables	Weaknesses/Deficiencies	X^2	P-Value	Decision
1	Assumptions and Constraints Analysis	Prone to missing latent assumptions and system constraints	49.00	0.000	Sig.
2	Document Reviews	Only precedential risks are identified	34.23	0.000	Sig.
3	Checklists	Recognised threats and ignore opportunities; is close ended in nature	22.34	0.000	Sig.
4	Brainstorming	Exclusion of stakeholders; bias resulting from dependence on top management and requires sieving of risks.	19.65	0.000	Sig.
5	Delphi techniques	Consider only technical risks, reliance on expert perception and experience only, and is time consuming.	12.09	0.000	Sig.
6	Interviews	Consumes time, and requires care filtering of risk to eliminate non-related risks	2.78	0.000	Sig.
7	Root-Cause Analysis	Excessive simplification of risks and is capable of omitting relevant risks	2.76	0.000	Sig.
8	SWOT Analysis	Generate high level risks and focus on risk within an organisation	4.32	0.000	Sig.
9	WBS Review	Exclude risks outside WBS	5.78	0.000	Sig.
10	Questionnaire	Requires prospering structuring of questions; and mainly close-ended	32.04	0.000	Sig.
11	Prompts Lists e.g. SLEEPT	High level risk classification technique providing mainly guidance with no specific risk identified	3.09	0.000	Sig.
12	Nominal Group Technique	Fatigue and generates varying perceptions	22.44	0.000	Sig.
13	Cause and Effect (Ishikawa) Diagrams	Generate complex diagrams	18.32	0.000	Sig.
14	System Dynamic (Process Flowchart)	Requires specialist application software and expert knowledge of IT and fails to consider probability	12.89	0.000	Sig.
15	Influence Diagrams	High level representation of situations requiring system thinking knowledge	11.09	0.000	Sig.
16	Failure Modes and Effects Analysis (FMEA) – Fault Tree Analysis	Requires expert tools and considers only threats	8.99	0.000	Sig.
17	Force Field Analysis	Complex, consumes time and considers specific project objective and not useful for holistic project evaluation	2.98	0.000	Sig.

The result in Table 6 was validated during the interview phase by asking respondents to identify how inherent weaknesses in risk identification tools trigger inefficiency in overall project risk management performance. Table 7 captures the conversational framings used by interviewees to address the problems created by weakness in risk identification tools. The causes of inefficiency in risk management practice linked with applied risk identification tools include, inability to capture fully risk events, boxed opinion, focused on individual experience only (as seen with tool such as Delphi method and checklists), biases and inability to enhance learning within risk management organisation. Other concerns include ineffective communication, provision of incomplete scope, and inhibition of collaborative working.

Table 7. Effects of Weaknesses in Risk Identification Tool on Performance

S/N	Frames	Descriptions
1	Inventive imagination	Requires apt ability to imagine future events
2	Focus in the now and project only	Consider only short-term risks thereby cloaking catastrophic risks
3	Promote boxed opinion	Inhibits knowledge sharing across project team
4	Inhibit learning and knowledge dissemination	Limit general risk management knowledge
5	Promotes biases	Imposing one's opinion on others based on the individual experience
6	Inhibits collaborative management	Inhibits teams work and collaborative risk management.
7	Treat risk using firefighting approach	Risk identification is practiced as a reactive measure to curb emerging problem rather than preventive actions.
8	Inhibits stakeholders' inclusion	Inclusive stakeholders enhance accurate risk identification; however, most tools are tied to individualized practices.
9	Gives incomplete scope	Limited ideas to expert view thereby omitting relevant risks in the projects.
10	Too many assumptions	Tools permit excessive assumptions which are most times untrue, unimaginable, and bias to results.
11	Inhibits effective communication	Leads to misunderstanding of inherent risks in a project and other stakeholder's view.

The study reveals that the application of risk management practices in the study area is still a significant low. This implies lack of improvement in the level of use of risk identification tools. The concern of this study is not whether, these results compare or differ from previous studies. It is concerned with what to do to improve state of risk management knowledge to benefit project performance. Widely understood and vastly applied risk management tools are qualitative. However, the prevalence of the generic qualitative risk management does not cut across industry strata exception of selected tools. Low application also means that risk management theories or protocols are not widely practiced. Inability to implement structured risk management protocol is responsible for the domination of traditional contingency allocation as a form of risk management. Although, this technique does not translate into any defined risk identification tool, intuition-based technique, however, explains its underlying strategy. The findings of the study therefore show no clear improvement from established findings in the literatures. Salawu and Abdullah (2014) reported that, the risk management maturity status of contractors in Nigeria is 'novice'. The novice status indicates risk perception is extreme low. Adedokun et al. (2011) and Ogunbayo (2014) reported that construction stakeholders in Nigeria are far from up scaling their risk management practices using quantitative approach. Nnadi and Ugwu (2013) attributed low awareness of risk identification tools to dearth of knowledge of potential risks affecting project delivery. It is therefore consistent to agree with Salawu and Abdullah (2014) that, risk management across contracting sector in Nigeria is 'novice' that is, very low.

In the international scene, Abdou et al. (2004) observed that the use of risk management in the general construction perspective is also low compared to other industries. The penchant to the use of subjective-intuitive risk management is also predominantly stressed in the literatures (Al-Zarooni and Abdou, 2000). Akintoye and Macleod (1997) asked its sample why certain risk analysis tools are not used; the reasons advanced include lack of familiarity, unrealistic estimate generated by the tools, lack of knowledge and doubts on the suitability of the techniques. In this study, issues relating to these dimensions are headlined and new grounds identified. These include disciplinary barriers; too much assumption;

inhibits innovation; and introduces ‘firefighting approach to risk management. Although, previous literature is divided about the contributions of risk identification to the overall ineffective risk management performance debate, Laryea and Hughes (2008) linked poor risk management performance to the use of advance tools. Zhang (2011) believed existing risk management tools lean towards increased standardization. On the other hand, Beck (1992) dissociates the effectiveness of risk management performance from risk identification tools. This means that stakeholder must begin to reconsider factors relating tools’ inherent weaknesses and the overall level of implementation of risk management practices.

Among the leading problems created by defective tools in risk management performance, poor communication and hindrance of teamwork standout undisputed. Ejohwomu (2014) identified that lack of cooperative risk management and lack of control and monitoring systems inhibits risk management performance in Nigeria. Nnadi and Ugwu (2013) maintained that effective communication between parties in the project delivery chain is essential to minimise risks. Osipova and Eriksson (2013) insisted that, mutual risk management requires effective communication between parties to promote a balanced view devoid of prescriptive biases. Tsiga et al. (2017) also advanced that interactive actions are essential to sustain interdisciplinary culture within risk management team. Stakeholders are therefore enjoined to apply risk identification tools that promote effective team communication and collaborative actions.

CONCLUSION

Risk management involves multi-faceted processes encompassing risk identification, assessment, quantification, planning, monitoring and control. However, the success of the overall risk management exercise relies significantly on the effective risk identification. This study evaluated risk identification practices of project managers in the Nigerian construction industry. The objective was to explore the weaknesses inherent in applied tools and emerging problems created that inhibits overall risk management performance. The study revealed that, increased awareness about risk management theory and developed tools has not assisted to improve risk management perception/practice of construction stakeholders. Risk management perception of construction stakeholders is till date unexpectedly very low and elementary. Applied risk identification tools are also limited to intuition-based subjective assessment developed mainly from assumption of expert’s perception. The implication is that, risk identification is not in essence, structured but vary based on individual project manager’s experience. Applied risk identification tools are also weak in many directions including inability to mapped holistic risks inherent in a project, inconsistency with tenets of team approach and exclusion of relevant stakeholders. Other lapses are reliance on individual experience, assumption laden, enlists varying perceptions, and complexity. These lapses trigger poor performance in risk management outcome by promoting boxed opinion of individuals. Other problems created by tools’ weaknesses include enhanced overture of a professional experience above others, induced biases, inhibits learning, and give incomplete scope of specific risks in a project. Strategies to improve existing practice and performance must be directed towards re-engineering applied tools to entrench interdisciplinary collaboration and team approach in risk identification. In this way, disciplinary practice will be limited. This will pave way for an objective approach that will not just assume risks exist but will also provide rational for measuring probability of occurrence and impact including quantitative appraisal of mitigation strategies.

REFERENCES

Adafin, J. and Rotimi, J.O. (2016). Determining Significant Risks in the Variability between Design-Stage Elemental Cost Plan and Final Tender Sum. *Journal of Management in Engineering,* DOI: 10.1061/(ASCE)ME.1943-5479.0000448.

Adedokun, O. A.; Ogunsemi, D. R.; Aje, I. O.; & Dairo, D. O. (2012). Evaluation of Risk Analysis Techniques in Selected Large Construction Companies in Nigeria. Emerald Group Publishing, Preprinted version.

Akintoye, A. S. and MacLeod, M. J. (1997). Risk Analysis and Management in Construction. *International Journal of Project Management*, 15(1), 31-8.

Amadi, A. and Higham, A. (2016). Geotechnical Characterisation of Cost Overrun Drivers in Highway Projects: Predicted on Heterogeneous Ground Conditions in the Niger Delta Region of Nigeria. *In*: P. W. Chan and C. J. Neilson (Eds) Proceedings of the 32nd Annual ARCOM Conference, 5-7 September 2016, Manchester, UK, ARCOM, Vol 1, 311-320.

Association of Project Managers [APM] (2013). *A Guide to the Project Management Body of Knowledge.* Fifth Edition Sylva, NC: PMI Publishing Division.

Beck, U. (1992). *Risk Society: Towards a New Modernity.* New York: Sage.

Ejohwomu, I. J. (2014). A Study of Risk Management Practices in the Construction Industry. M.Sc Thesis submitted to the University of Wolverhampton.

Elfaki, A.O. and Alatawi, S. (2015). Representing the Knowledge of Public Construction Project Cost Estimator by Using Rule-Based Method. *Journal of Building Construction and Planning Research*, 3, 189-195, http://dx.doi.org/10.4236/jbcpr.2015.34019

Farrokshand, M.; Chan, P. W. Blackwell, P. (2016). Opening up Risk Management through Gofffman's Dramatugical Approach. *In*: P. W. Chan and C. J. Neilson (Eds) Proceedings of the 32nd Annual ARCOM Conference, 5-7 September 2016, Manchester, UK, ARCOM, Vol 1, 237-246.

Goh, S. C., & Abdul-Rahman, H. (2013). The Identification and Management of Major Risks in the Malaysian Construction Industry. *Journal of Construction in Developing Countries* 18(1), 19-32.

Kotb, M. H. & Ghattas, M. M. (2017). An Overview of Risk Identification Barriers with Implementation of PMI Standards in Construction Projects. *PM World Journal*, VI(X), 1-12.

Kumar, R.; Chandrashekhar, I. K. and Singh, S. P. (2017). Quantification of Construction Project Risks by Analysis of Past Dispute Cases. *In:* Chan, P W and Neilson, C J (Eds) *Proceeding of the 33rd Annual ARCOM Conference,* 4-6 September 2017, Cambridge, UK, Association of Researchers in Construction Management, 532-541.

Laryea, S and Hughes, W (2008). How Contractors Price Risk in Bids: Theory and Practice. *Construction Management and Economics,* 26(9), 911-924.

Leslie, W. and Lix, L. (2014). Comparison between various Fracture Risk Assessment Tools. *Osteoporosis International,* 25(1), 1-21.

Liaudanskiene, R. & Ustinochius, L. (2010). Review of Risk Assessment Methods and the Peculiarities of their Application at Construction Sites. The 10th International Conference of Modern Building Materials, Structures and Techniques, May 19-21st, Vilnius, Lithuania.

Lu, S. & Yan, H. (2013). A Comparative Study of the Measurement of Perceived Risk among Contractor in China. *International Journal of Project Management*, 31:307– 12

Mahendra, P., Pitroda, J. R., & Bhavsar, J. (2013). A Study of Risk Management Techniques for Construction Projects in Developing Countries. *International Journal of Innovative Technology and Exploring Engineering.* 3(5), 139-142.

Nnadi, E. E. & Ugwu, O. O. (2013). An Appraisal of Risk Management in Nigeria Construction Industry. *International Journal of Research and Advancement in Engineering Science*, 3(2), 41-50.

Odeyinka, H. A. & Dada, J. O. (2016). Risk Assessment and Allocation in Budgeting. A paper delivered at the Nigerian Institute of Quantity Surveyors (NIQS) Workshop: Budgeting and Capital Project Monitoring and Evaluation an Era of Change Held on 27th to 28th July, 2016 at Precious Conference Centre, Makurdi, Benue State.

Odusami, K. T., Iyaba, R. R and Omirin, M. M. (2003). The Relationship between Project Leadership, Team Composition and Construction Project Performance in Nigeria. *Journal of International Project Management,* 21: 519-527.

Ogunbayo, O. M. (2014). Assessment of Risk Analysis Processes in the Nigerian Construction Industry: Contractors' and Project Managers' View. *International Journal of Advancement in Research & Technology*, 3(1), 36-42.

Ogwueleka, A. C. & Mendie, E. I. (2014). Evaluation of Safety and Quality Management in Construction Projects – A Study of Ibom Tropicana Construction Projects. *Journal of Construction Project Management and Innovation*, 4(1): 826-43.

Osipova, E. and Eriksson, P. E. (2013). Balancing Control and Flexibility in Joint Risk Management: Lessons Learned from two Construction Projects. *International Journal of Project Management,* 31(3), 391-399.

Pott, K. (2008). *Construction Cost Management Learning from Case Studies. First* Edition, London: Taylor & Francis.

Salawu, R. A. & Abdullah, F. (2014). Assessing Risk Management Maturity of Construction Organisations on Infrastructural project Delivery in Nigeria, *Procedia-Social and Behavioural Sciences*, 172, 643-650.

Stalker, K (2003). Managing Risk and Uncertainty in Social Work a Literature Review. *Journal of Social Work*, 3(2)**,** 211-233.

Taroun, A. (2014). Towards a Better Modelling and Assessment of Construction Risk: Insights from a Literature Review. *International Journal of Project Management*, 32(1)**,** 101-115.

Tipili, L. G. & Yakubu, I. (2016). Identification and Assessment of Key Risk Factors Affecting Public Construction Projects in Nigeria: Stakeholders Perspective. *International Journal of Engineering and Advanced Technology Studies*, 4(2), 20-32

Tsiga, Z., Emes, M., & Smith, A. (2017). Attitude and Personality towards Risk Management in the Construction Industry. Second edition, *PM World Journal,* VI(X), 1-11.

Van, R.; Bowen, P.; & Akintoye, A. (2008). Project Risk Management Practice: The Case of a South African Utility Company. *International Journal of Project Management*, 26, 149-163.

Walke, R., & Topkar, V. (2012). Risk Quantification Model for Construction Projects Using Score Model and EV Analysis Approach. *International Journal of Engineering Research and Applications*, 2312-2317.

Wood, G.D. & Ellis, R.C.T. (2003). Risk Management Practices of Leading UK Consultants. *Engineering, Construction, and Architectural Management,* 10(4), 254-262

Yauger, V. (2017). When Risk Management Meets Risks Realised: Mitigating Project Impact. *PM World Journal*, VI(X), 1-13

Zhang, H (2011). Two Schools of Risk Analysis: A Review of Past Research on Project Risk. *Project Management Journal*, 42(4), 5-18.

Zwikael, O and Ahn, M (2011). The Effectiveness of Risk Management: An Analysis of Project Risk Planning Across Industries and Countries. *Risk Analysis,* 31(1), 25-3

SEISMIC RESPONSE OF FLEXURE-SHEAR FAILURE OF RC STRUCTURE DESIGNED USING FEMA 356 CRITERIA

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Abstract
Traditional force-based design has been used to design many reinforced concrete (RC) structures. In this approach, the design only needs to satisfy the forces based on the load combination. The current seismic code requires members to undergo a certain rotation before collapsed, known as performance-based design. The shift from traditional forced-based design for performance-based design caused many old RC structures to have higher probabilities of collapsing under seismic event. These structures need to be evaluated carefully. RC members can fail either in ductile or brittle manner, depending on the detailing of the members. The important factor for seismic evaluation is to identify appropriately, effective stiffness and plastic hinge model to evaluate the structures. Current FEMA 356 plastic hinge model for RC member does not clearly address for member fails in flexure-shear mode. A 5-story building designed to fail in flexure-shear mode are used to study the response of the structure modelled with FEMA 356 plastic hinge model. This study will compare the plastic hinge from FEMA 356 with theoretical plastic hinge model, developed from moment-curvature analysis with a shear envelope to consider the flexure-shear failure mode. The effective stiffness of RC members resulted from moment-curvature analysis is used in modelling to study the deformation of the structure before collapsing.

Keywords: *Seismic response; flexure-shear; plastic hinge model*

INTRODUCTION

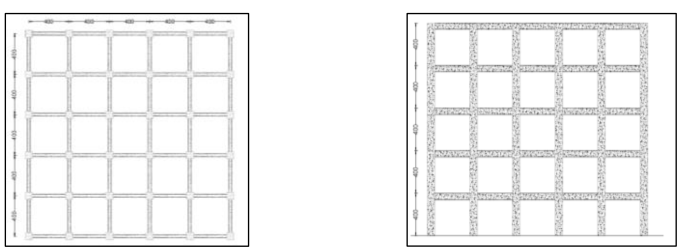
Catastrophic losses of human lives and financial resources resulting from large earthquakes near urban population centres remain one of the most daunting challenges in achieving a resilient human society – demographically, socially and economically. In the post-world wars construction booms, hundreds of thousands of RC structures were built around the world without much or adequate consideration for earthquake forces. Many structures were failed in flexure-shear failure. In the case of RC structures, the problem is compounded by the fact that this type of construction is widely used for critical public buildings such as schools, emergency services and public administration offices. As such, modern societies in seismically active regions are dealing with a ubiquitous amount of seismically-at-risk RC structures. For instance, in the Christchurch earthquake on the 22 February 2011, there were 182 fatalities, 135 of which were the unfortunate consequences of the complete collapse of two mid-rise RC structures (Kam, 2010). A proper structural design and analysis, accounting for sufficient strength and ductility under the influence of earthquakes must be considered to assure the structural safety. In fact, the number of multi-storey RC moment-resisting frame structures that enable their seismic energy to dissipate in a secure manner has rapidly been increased over the past half century around the world including Indonesia. Hence, the seismic performance of such structures should be appropriately assessed within the range of standard seismic structure code-based simulation procedures and then be stochastically evaluated using reliable technique (Anudai et al., 2016; Jaafar et al., 2016).

To evaluate the seismic performance of multi-storey RC structures in a codified and stochastic manner, both codified response spectrum analysis and nonlinear time-history analysis in conjunction with a conventional theoretical hinges theory and FEMA 356 criteria for plastic hinge were used for this study (Bentz, 2000; ASCE, 2000). This paper will provide an insight of seismic response of a 5-story RC structure with flexure-shear failure mode in the beams modelled using theoretical plastic hinge and using FEMA 356 plastic hinge.

This paper is composed of four sections. Section 2 focuses on describing the RC structure used in this study. The RC structure is an RC moment-resisting frame structure. Section 3 is devoted to outlining each procedure for the codified FEMA 356 plastic hinge criteria and theoretical plastic hinge analysis. The last section presents the comparison and conclusions of seismic performance in terms of flexural rigidities, yield displacements, and performance points of the structure between both approaches.

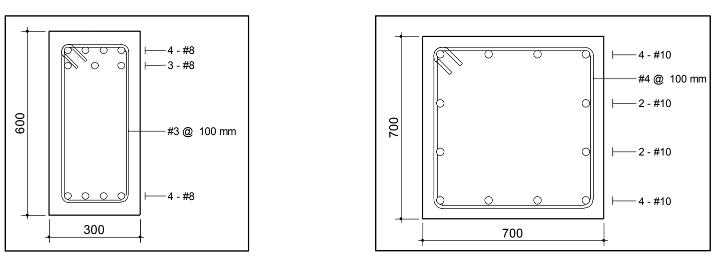
DESCRIPTION OF RC STRUCTURE

A five-story RC structure located in Jayapura, Indonesia was selected as the model structure for investigation in this study. The structure uses moment-resisting frames in both longitudinal and transverse directions. Overall height of the structure is 20 m. The floor height is 4 m each. The platform of structure is a square shape as shown in Figure 1a and the elevation view can be seen in Figure 1b. The structure has a total length of 20 m with five 4 m-long bays. The sizes and details for columns and beams were determined by designing them according to SNI 1726:2012 (2012) and SNI 2847:2013 (2013). The section of 0.3 m x 0.6 m is used for all beams on each floor. Moreover, the columns size is 0.7 m x 0.7 m. The beam and column details with their reinforcements can be seen in Figure 2a and Figure 2b, respectively. The compressive strength of the concrete (f_c') used is 25 MPa. The yield strength of steel reinforcement (f_y) used in the beams is 400 MPa and 240 MPa for longitudinal reinforcements and transverse reinforcements, respectively. Furthermore, a 400 MPa yield strength (f_y) is used in the longitudinal and transverse reinforcements of the column.



(a) (b)

Figure 1. Configuration of RC structure: (a) Plan view; (b) Elevation view



(a) (b)

Figure 2. Dimension and reinforcement details: (a) Beam; (b) Column

FEMA 356 AND THEORETICAL HINGE APPROACHES

Two standard plastic hinges computational approaches for the structure are described herein. This description deals with the details of flexural rigidity and assigned hinge properties specific to the structure in the following subsections.

FEMA 356 Approach

To obtain the actual deformation, effective stiffness should be taken into consideration when modelling the structure (Priestley, 2003). Table 1 shows the effective stiffness values specified by FEMA 356.

Table 1. Effective stiffness value

Component	Flexural Rigidity
Beams-nonprestressed	0.5EcIg
Columns with compression due to design gravity loads ≥ 0.5 Agf'c	0.7EcIg
Columns with compression due to design gravity loads ≤ 0.3 Agf'c or with tension	0.5EcIg

Figure 3 shows the generalized force-deformation relation to the concrete element associated with the flexure response from FEMA 356. The deformations are expressed directly using terms such as strain, curvature, rotation or elongation. Both failure conditions, ductile and brittle behaviour, are clearly specified by FEMA 356, but does not clearly specify hinge properties for members fail in flexure-shear behaviour.

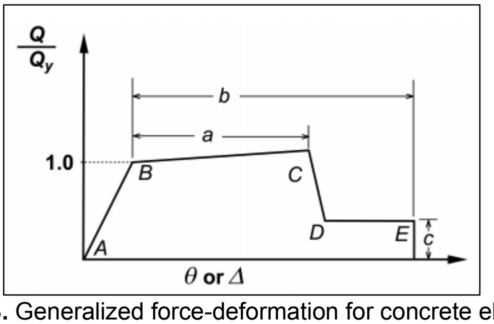


Figure 3. Generalized force-deformation for concrete elements

As beams and columns will fail in shear after reaching the yield capacity, 'Beams controlled by flexure' and 'Columns controlled by flexure' conditions from FEMA 356 will be used in evaluating the seismic performance of the structures. Table 2 and Table 3 shows the conditions for 'Beams controlled by flexure' and 'Columns controlled by flexure' from FEMA 356.

Table 2. Beams controlled by flexure

$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement	$\frac{V}{b_w d \sqrt{f'_c}}$	a	b	c
≤ 0.0	C	≤ 3	0.025	0.050	0.2
≤ 0.0	C	≥ 6	0.020	0.040	0.2
≥ 0.5	C	≤ 3	0.020	0.030	0.2
≥ 0.5	C	≥ 6	0.015	0.020	0.2
≤ 0.0	NC	≤ 3	0.020	0.030	0.2
≤ 0.0	NC	≥ 6	0.010	0.015	0.2
≥ 0.5	NC	≤ 3	0.010	0.015	0.2
≥ 0.5	NC	≥ 6	0.005	0.010	0.2

Table 3. Columns controlled by flexure

$\frac{P}{A_g f'_c}$	Transverse reinforcement	$\frac{V}{b_w d \sqrt{f'_c}}$	a	b	c
≤ 0.1	C	≤ 3	0.020	0.030	0.2
≤ 0.1	C	≥ 6	0.016	0.024	0.2
≥ 0.4	C	≤ 3	0.015	0.025	0.2
≥ 0.4	C	≥ 6	0.012	0.02	0.2
≤ 0.1	NC	≤ 3	0.006	0.015	0.2
≤ 0.1	NC	≥ 6	0.005	0.012	0.2
≥ 0.4	NC	≤ 3	0.003	0.010	0.2
≥ 0.4	NC	≥ 6	0.002	0.008	0.2

Theoretical Hinge Approach

Bilinear approximation from moment-curvature analysis, shown in Figure 4, is used for the flexural strength of the RC member (Hsu and Mo, 2010; Priestley et al., 1994). Three possible shear strength envelopes also plotted in Figure 4, each corresponding to different spacing of the transverse reinforcement. Strength envelope 1 refers to ductile behaviour where shear failure does not occur before full displacement capacity. Strength envelope 2 refers to semi-ductile behaviour where shear failure occurs after flexural strength but does not reach the full displacement capacity. Strength envelope 3 refers to brittle behaviour where shear failure occurs prior to the development of flexural strength.

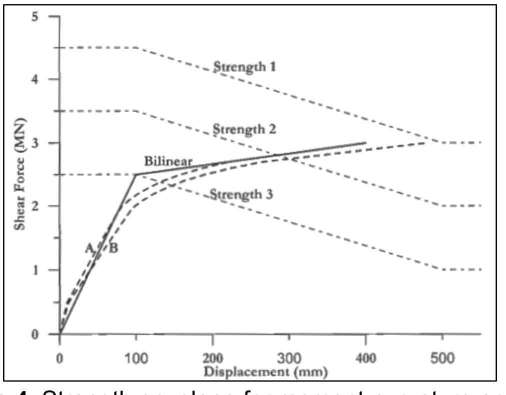


Figure 4. Strength envelope for moment-curvature analysis

The modified UCSD model is used as shear strength envelope since this approach provides a better agreement with experimental results (Kowalsky and Priestley, 2000). The model includes the shear strength provided by a concrete mechanism (V_c), transverse reinforcement mechanisms (V_s), and axial force mechanism (VP) as shown in this equation.

$$V = V_C + V_S + V_P \tag{1}$$

Concrete-shear resisting mechanism (V_c)

The strength of concrete shear-resisting mechanism is provided by aggregate interlock on the rough flexure/shear cracks. The strength reduces in the plastic hinge regions due to flexure/shear crack widen under ductility. The strength also dependent on the aspect ratio of member, M/(VD), where M and V are the moment and shear at the critical section and D is the total section depth and on the volumetric ratio of longitudinal reinforcement $\rho_l = A_{st}/A_g$.

$$V_c = k\sqrt{f'_{ce}} \cdot A_e = \alpha\beta\gamma\sqrt{f'_{ce}} \cdot (0.8A_g) \tag{2}$$

where: $1.0 \leq \alpha = 3 - M/(VD) \leq 1.5$, $\beta = 0.5 + 20\ \rho_l \leq 1.0$, γ is given in Figure 5 for concrete columns.

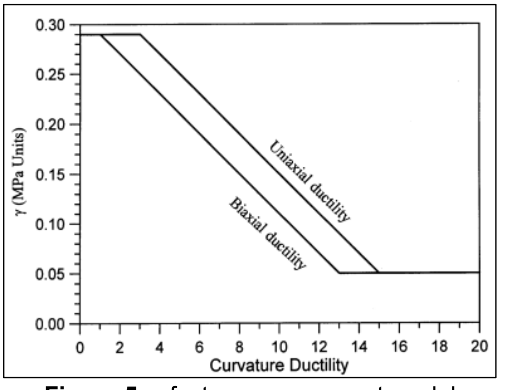


Figure 5. γ factor – assessment model

The prime variable is the curvature ductility demand, which is directly related to the width of flexure/shear cracks in the plastic hinge region. Secondary variable is the mode of ductility; the strength degrades more rapidly in biaxial ductility than sections subjected to uniaxial ductility. For concrete beams section, a 20% reduction of shear strength to compensate for the concrete confinement which are less satisfactory than for columns, rather the total reinforcement area be used to determine β.

Transverse Reinforcement Truss Shear-Resisting Mechanism (V_s)

The strength of transverse reinforcement truss mechanism is illustrated in Figure 6. It was assumed that the shear force is transferred through the diagonal crack and mobilize the transverse reinforcement along the crack. However, the crack is closed in the compression zone, therefore, the shear cannot be transferred. From Figure 6, it is apparent that a reduced column with of D-c-cov is appropriate for calculating the number of hoops mobilized by the cracks between the compression struts. The total resistance provided by the transverse reinforcement can be estimated as:

$$V_s = \frac{A_v f_{yh}(D-c-cov)\cot\theta}{s} \tag{2}$$

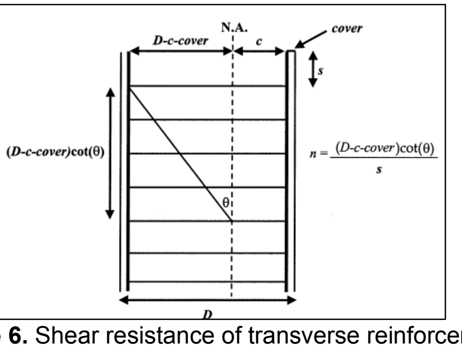


Figure 6. Shear resistance of transverse reinforcement

Axial Load Component (V_p)

The shear strength enhancement due to axial compression is considered as an independent component, resulting from diagonal compression strut, as shown in Figure 7. The average inclination of the strut involving the axial force is shown as angle ζ to the member axis. For column, which is restrained from rotation at the top and bottom, the axial force is effectively applied to column through the centre of the flexural compression at beam top and exits through the centre of flexural compression at the bottom. The horizontal component of this strut acts to resist the applied shear force, thus enhancing the column shear strength. The equation for V_p:

$$V_P = P \cdot \tan\zeta \tag{3}$$

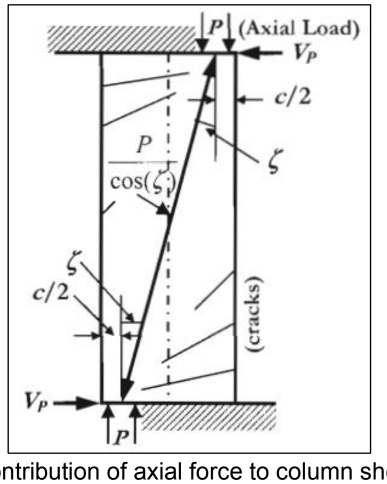


Figure 7. Contribution of axial force to column shear strength

STRUCTURE MODELLING

Structure failed in flexure-shear behaviour will fail after the yielding of flexural reinforcement, then fail in shear. Therefore, for the hinge model from FEMA 356 corresponding to Figure 3 used in the model for evaluation is shown in Table 4, and for theoretical hinge is shown in Table 5. Both properties are modelled as non-linear hinge in ETABS.

Table 4. FEMA 356 hinge properties

Member	a	b	c
Beam	0.010	0.015	0.2
Column	0.020	0.030	0.2

Table 5. Theoretical hinge properties

Member	a	b	c
Beam	0.042	0.066	0.71
Column	0.022	0.062	1.00

RESULTS AND DISCUSSIONS

The strength and ductility of beams and columns subjected to shear, moment, and axial loads were computed using a sectional analysis program, i.e., Response-2000 (Bentz and Collins, 2001). Shear, moment, and axial loads are considered simultaneously to find the full load-deformation response using research based on modified compression field theory. After that, the static nonlinear analyses were performed using ETABS Nonlinear v9.7.2 from CSI (11). The static pushover curves obtained from FEMA 356 and theoretical hinge approaches were then compared. The performance points of both approaches can be seen in Figure 8(a) and Figure 8(b):

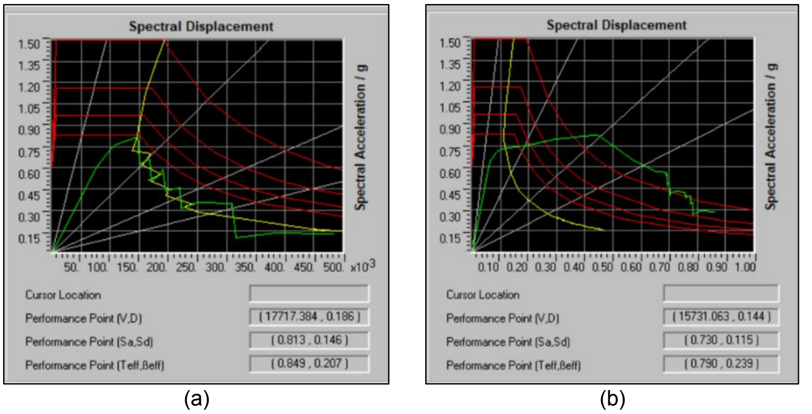


Figure 8. Capacity spectrum: (a) FEMA 356; (b) Theoretical hinge

It can be seen from Figure 8 that capacity spectrums obtained from both approaches were not that significantly different. Theoretical hinge gave a slightly more conservative result with smaller base shear and displacement. However, spectral acceleration obtained from FEMA 356 (0.813g) and theoretical hinge (0.73g) were bigger than the design spectral acceleration of Jayapura according to SNI 1726:2012 (0.6g). It means that this five-story RC structure can perform satisfactorily during earthquake events.

In Figure 9, a comparison between base force versus displacement from both approaches can be seen.

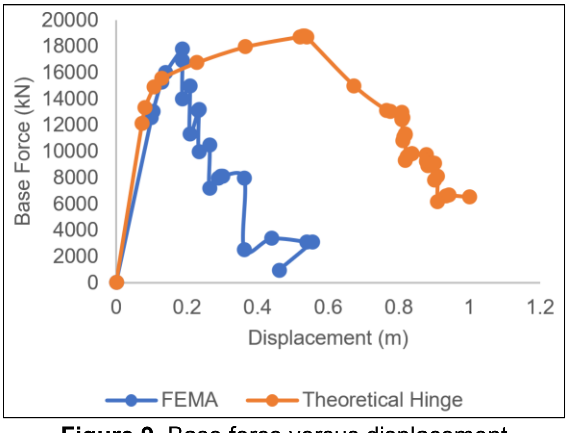


Figure 9. Base force versus displacement

Agreeing with the capacity spectrums in Figure 8, the performances of the structures were very similar. FEMA 356 had a higher base force, but with slightly larger yields displacement. It is because FEMA 356 uses smaller flexural rigidity than theoretical hinge in both beams and columns as shown in Table 1. From Figure 9, it also can be seen that theoretical hinge structure is slightly more rigid than FEMA 356 structure. To accurately evaluate RC structures, especially RC structures with deformed controlled behaviour, an appropriate stiffness should have to be determined first. FEMA 356 gives a constant reduction factor for beams and column under certain condition, but research by Priestley (2003) shown that member stiffness does not constantly and correlates with the reinforcement provided. It is also notable from Figure 9 that RC structures with theoretical hinge can deform further without losing their capacity. This behaviour could not be captured in FEMA 356 (i.e. losing their capacity after reaching the maximum base shear). Further research is needed to investigate RC structures with brick wall or concrete wall, since they will extend the stiffened area of the member.

REFERENCES

Anudai, S.A., Hamid, N.H. and Hashim, M.H., (2016). Experimental Study on Seismic Behavior Of Repaired Single and Double Unit Tunnel Form Building Under In-Plane Cyclic Loading, Malaysian Construction Research Journal, 19(2), 19-28.

ASCE (2000) *Prestandard and Commentary for the Seismic Rehabilitation of Building-FEMA 356*. Federal Emergency Management Agency, Washington, D. C.

Bentz, E. and Collins, M. P. (2001) *Response-2000*. User Manual. University of Toronto.

Bentz, E. C. (2000) *Sectional Analysis of Reinforced Concrete Members*. Ph.D. Thesis, Department of Civil Engineering, University of Toronto, Canada.

Habibullah, A. (2016) *ETABS 2016*. User's Guide. Computers & Structures, Inc.

Hsu, T. T. C. and Mo. Y. L. (2010) *Unified Theory of Concrete Structures*. John Wiley & Sons, Chichester.

Jaafar, M.S., Hamid, N.H. and Mahmood, N.H. (2016), The Behaviour of Rocking Multi-Column Pier of Unskewed Bridge Under Seismic Performance Using Quasi–Static Lateral Cyclic Loading, Malaysian Construction Research Journal, 19(2), 29-38.

Kam, W. Y. (2010) *Selective Weakening and Post-Tensioning for the Seismic Retrofit of Non-Ductile RC Frames*. Ph.D. Thesis, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Kowalsky, M. J. and Priestley, M. J. N. (2000) Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions. *ACI Structural Journal* Vol. 97, Issue 3. (May 2000), 388-396.

Priestley, M. J. N. (2003) *Myths and Fallacies in Earthquake Engineering, Revisited*. Rose School, Pavia.

Priestley, M. J. N., Verma R., and Xiao. Y. (1994) Seismic shear strength of reinforced concrete columns. *Journal of Structural Engineering* Vol. 120, Issue 8. (August 1994), 2310-2329.

SNI (2012) *Tata Cara Perencananaan Ketahanan Gempa untuk Struktur Bangunan Gedung dan Non Gedung-SNI 1726:2012*. Badan Standardisasi Nasional, Jakarta.

SNI (2013) *Persyaratan Beton Struktural untuk Bangunan Gedung-SNI 2847:2013*. Badan Standardisasi Nasional, Jakarta.

WATER PERMEABILITY OF CONCRETE CONTAINING COAL BOTTOM ASH AS PARTIAL REPLACEMENT OF SAND AND CEMENT WITH DIFFERENT GRINDING TIME

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Abstract
Coal Bottom Ash (CBA) is the hazardous waste that potential threats to people health and the environment. The increasing production of CBA need to find new alternative to use this material so that method exposed to landfill is prevented and could reduce the bad effect to environmental. Therefore, this paper presents an experimental investigation on the effect of CBA as sand and cements replacement on compressive strength and water permeability of concrete. For the optimum content for cement replacement, CBA was ground into three different grinding times as 20, 30, and 40 hours where the replacement levels are 10%, 20%, and 30%. Meanwhile, for the optimum content for sand replacement are 5%, 10%, 15% and 20. All the specimens were cured for 7 and 28 days. The result shows that at 20% replacement level of CBA in 30 hours grinding time has potential for cement replacement, whereas 15% replacement of CBA was the optimum for sand replacement. However, it was found that the strength of the combination of both optimum cement and optimum sand replacement with CBA shows the lowest strength. Besides that, there was an increment in water permeability test.

Keywords: *Coal bottom ash; grinding time; water permeability; workability; porosity; compressive strength*

INTRODUCTION

The challenge for the civil engineering community in the near future is to realize projects in harmony with the concept of sustainable development and this involves the use of recycled materials and wasted materials that can be manufactured at reasonable cost and lowest possible environmental impact. One of the possible ways to increase the productivity of concrete which is also can minimize the wasted material is by using a coal bottom ash. Steam (1978) stated that CBA was produced as a result of burning coal in a pulverized coal boiler from Electric Power Plant. Many investigations found that the CBA has some cementitious properties in which may increase the strength than normal concrete. Wan Ibrahim et al. (2016) found that the strength of concrete increase by the addition of CBA as sand replacement in concrete. Besides, Bakoshi et al. (1998) stated the compressive strength of CBA concrete generally increases with the increase in replacement from (10% to 40%) of fine aggregate and curing age. In this study, since the CBA was classified as a porous material, the grinding process has been done to obtain the micro-size of CBA that can be partial replacement of cement. Similar method has been done by Ramadhansyah et al. (2011) as they used ground rice husk ash at different grinding time. Jaturapitakkul et al. (2003) investigated that the compressive strengths of mortar containing 20 to 30% of ground CBA produce higher compressive strength than the cement mortar after 60 days. Furthermore, the addition of coal ash as a supplementary cementing material also causes an increase on both pozzolanic and physical properties that enhance the performance of concrete (Razak et al, 2008). Other than that, Kadam (2013) stated that the water permeability of up to 50% replacement of sieved CBA was decreased. The gel formation is clearly visible, with the micro-cracks occurring around the CBA particle, causing the strength of the concrete to reduce to less than that of the control concrete. Therefore, in this paper the effects of CBA as sand and cement replacement was investigated on the strength and water permeability test.

METHODOLOGY

Material Properties

CBA from Malaysia power plant and Portland cement type I (PC) water were used in this study. CBA were ground into 20 hour (20CBA), 30 hour (30CBA) and 40 hour (40CBA) to be used as cement replacement. For sand replacement, the CBA was sieved into 5mm in size. The sieve analysis on CBA sample in Figure 1 clearly shows that the percentage passing is distributed within the range and fully complied with overall limits except for raw CBA as given in BS 882:1992 (1992).

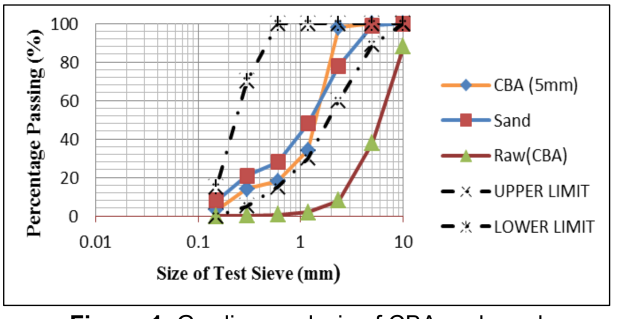

Figure 1. Grading analysis of CBA and sand

Mix Proportion and Preparation of Samples

Concrete with cube size (150mm x 150mm x 150mm) containing CBA with Grade 30 were produced by using different replacement percentage ranging from 10% to 30% by weight for cement replacement and 5% up to 20% for sand replacement. All the specimens were cured for 7 and 28 days. Mix proportion for combination of optimum cement and optimum sand was tabulated in Table 1.

Table 1. Mix proportion

Mixture	Cement Replacement		Sand Replacement	
	Ordinary Portland Cement (%)	CBA (%)	Sand (%)	CBA (%)
Control (M1)	100	0	100	0
Opt.Cement (M2)	80	20	100	0
Opt.Sand (M3)	100	0	75	15
Opt.Cement + Opt.Sand (M4)	100	20	75	15

At the first phase, the compressive strength test was conducted to identify the optimum content of bottom ash for each replacement. As the optimum content of bottom ash as partial replacement of cement and sand identified, the compressive strength and water permeability test was then conducted on the combination for both replacements as the second phase.

Water Permeability

The water permeability coefficient (Kp) was calculated by using Valenta's equation (Valenta, 1970).

$$\text{Kp} = \frac{d^2\text{U}}{2ht} \qquad \text{Equation 1}$$

Where Kp is a coefficient of water permeability (m/s), d represents depth of water penetration in concrete (m), U is the porosity of concrete, h denotes hydraulic head of water (m) and t is time under pressure (s). Meanwhile, Equation 2 is used to determine the value of porosity of concrete.

$$\text{U} = \frac{\text{m}}{Adp} \qquad \text{Equation 2}$$

Where U is a value of porosity, m is representing the gain in mass (g), A is cross-sectional area of the specimen (mm^2) and p is density of water ($1000kg/m^3$).

RESULT AND ANALYSIS

Workability

The slump test was conducted for all mix according to BS EN 12350-2: 2009. It could be seen that the slump was decreases as the replacement of CBA increased as shown in Figure 2. The result was found to be complement to a study conducted by Hamzah et al. (2015) and Jamaluddin et al. (2016). This can be concluded that the concrete incorporated with CBA has less workability compared to the control. It can be concluded that the CBA required a considerable amount of water to produce a workable mix of concrete depends on the water-cement ratio as the CBA have a porous texture and angular shape.

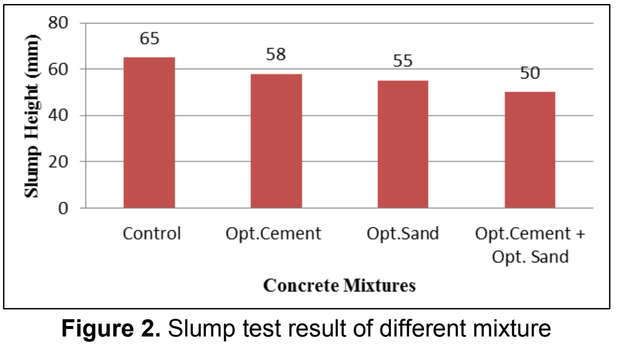

Figure 2. Slump test result of different mixture

Compressive Strength Test (Optimum Cement Replacement)

Compressive strength was tested according to BS EN 12390-3:2009 (2009). The optimum percentage of the cement replacement was obtained at 20% replacement level in 30 hours of grinding time which is 40.9 N/mm^2 as shown in Figure 3. The addition of coal ash as a supplementary cementing material causes an increase in both pozzolanic and physical properties that enhance the performance of concrete. Similar observation was reported by Abdulhameed et al. (2012) as he mentioned the concrete with a combination of CBA can produce strength above 30 MPa at 28 days of curing as he used ground CBA as partial replacement of cement. This increase in strength also was due to the increase in the availability of the C-S-H gel. During the initial period, the CBA in concrete was observed to gain strength at a slow rate, whereas after 28 days it was observed to have acquired strength at a faster rate due to the pozzolanic action of the CBA (2012). It can be concluded that the incorporation of 20% of CBA in 30 hours grinding time as partial replacement of cement contribute to produce the high strength concrete compared to other percentages at different grinding time.

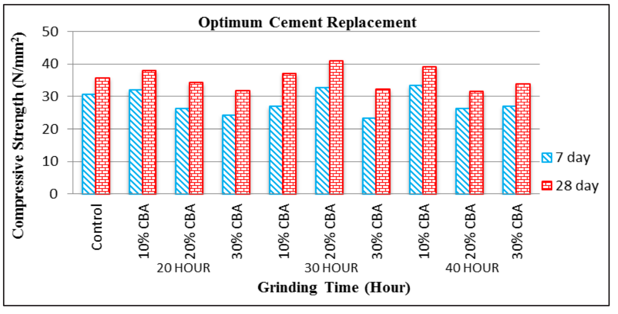

Figure 3. Compressive strength of optimum cement replacement

Compressive Strength Test (Optimum Sand Replacement)

Figure 4 shows that control samples give a comparatively lower compressive strength about 47% as compared to the 15% of CBA content. However, the compressive strength suddenly drops when the sand replacement up to 20%. This is due to the excessive amount of CBA had produced porous concrete since the CBA was classified as a porous material, hence contributing to reduce its compressive strength (Abu Bakar and Baharuddin, 2012). As a conclusion, the strength for 15% replacement of CBA is the most adequate as optimum sand replacement in concrete.

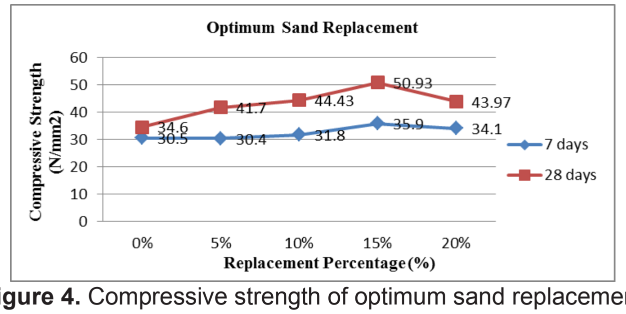

Figure 4. Compressive strength of optimum sand replacement

Compressive Strength Test (Combination of Optimum Content)

After the optimum content of cement and sand replacement obtained, the compressive strength test of both combinations was conducted. It can be seen from Figure 5, the maximum compressive strength was achieved by the mixes proportion of 15% replacement of sand with the compressive strength of 50.93 N/mm^2, while the minimum compressive strength of 24.6 N/mm^2 is from the M4 mixture where it contains 20% and 15% of CBA replaced with cement and sand respectively. It can be said that the combination of both optimum replacements gives a lower compressive strength compared to the other mix. This is due to the fact that the quantity of CBA leads to excess silica leaching out causing a deficiency in strength as it replaces some part of the cementitious material in concrete (Givi and Rashid, 2010).

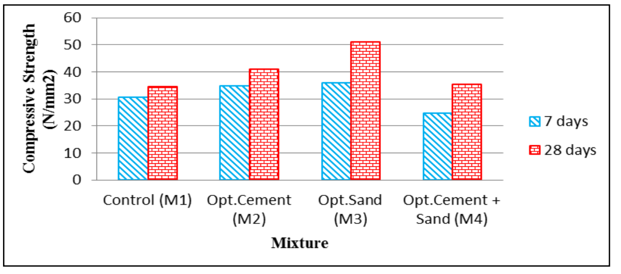

Figure 5. Compressive strength of both optimum contents

Table 1. Combination of both optimum contents

Mix Design/ Curing Day	Control (N/mm2) (M1)	Optimum Cement Replacement (N/mm2) (M2)	Optimum Sand Replacement (N/mm2) (M3)	Opt. Cement + Opt. Sand Replacement (N/mm2) (M4)
	0%	20% + 0%	0%+15%	20%+15%
7 days	30.5	34.90	35.90	24.6
28 days	34.6	40.85	50.93	35.2

Depth of Water Penetration

After 72 hours, the specimen was split to determine water penetration depth. Both combination of optimum cement and sand replacement mixture (M4) experience the highest penetration depth whereas, the lowest penetration depth was observed in control mixture as shown in Figure 6. The depth of water penetration increases about 3.2% from the M1 to the M4 mix. The reason for this observation is believed due to the existence of capillaries that filled with water in the concrete which containing the CBA. This is also due to the characteristics of CBA that has a porous or vesicular texture and absorbs water. However, the depth of water penetration decreases at day 28 for all mixtures. This is maybe due to the increasing of the formation of the C-S-H gel reaction in samples at day 28 compared to day 7 that formed the large crystals inside the voids and create dense microstructure that can increase the strength and reduces the permeability of the concrete.

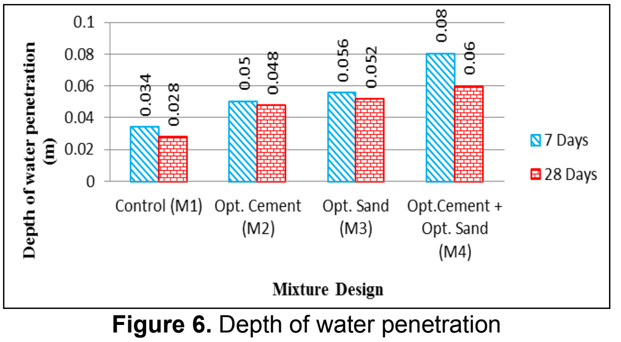

Figure 6. Depth of water penetration

Porosity of Concrete

Porosity is the measurement of the proportion of the total volume of concrete occupied by pores, and is usually expressed in percentage (Neville, 1995). The porosity value in the M4 mixture shows the highest porosity value which increased about 16% from the control sample (M1) as shown in Figure 7. This could be due to the addition of 20% and 15% of the coal bottom ash as partial replacement of cement and sand respectively, which increases the porosity in the concrete samples. Despites of the increases in the porosity due to the presence of CBA in the mixture, the silica content in the ground CBA particles will enhance the formation of C-S-H gel that responsible for the reduction of pores. As the conclusion, the addition of the CBA in concrete will increase the porosity value as the CBA volume increase.

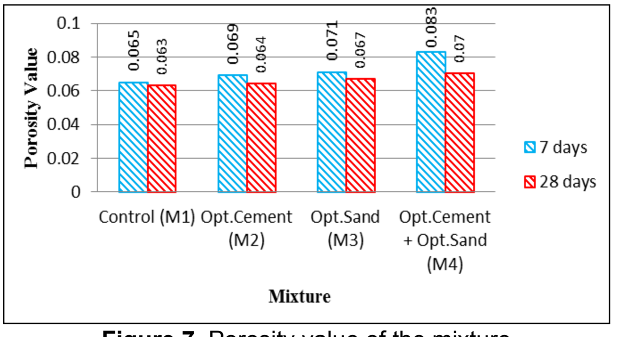

Figure 7. Porosity value of the mixture

Coefficient of Water Permeability

The coefficient of the water permeability is expressed by using Valentas's equation (Equation 1). Referring to the Figure 8, the coefficient of water permeability increases gradually from M1 to M4 mixture with the significant increment was observed from M2 to M3 mixture. This shows that the 20% replacement of cement with CBA has less water permeability as compared to 15% replacement of sand and combination of both 20% and 15% replacement for cement and sand respectively. This indicates that the sample containing both the cement and the aggregate replacement contains more pores that can permit liquid to pass through the sample. This result can be related to the porosity of concrete since the porosity value increases with the addition of the CBA. Higher porosity and coarse pore structure mean higher permeability of concrete (Neville, 1995).

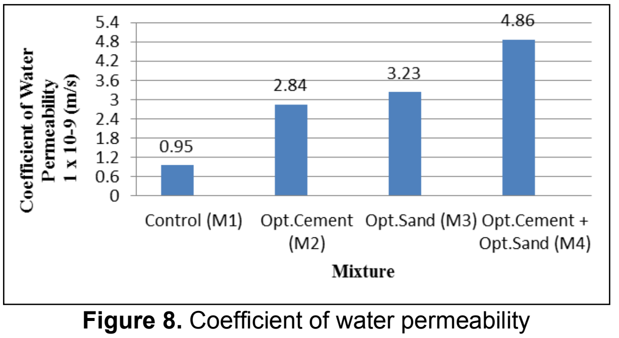

Figure 8. Coefficient of water permeability

CONCLUSIONS

As a conclusion, it was observed that the incorporation of 20% of CBA in 30 hours grinding time as partial replacement of cement contributed to produce the high strength compared to other percentages at different grinding time. The increase of strength was due to the increase in the availability and the formation of the C-S-H gel of CBA and its pozzolanic reaction. This indicates that 20% replacement level of CBA in 30 hours grinding time is the optimum cement replacement percentage and has potential to be a cement replacement material. Meanwhile, for the sand replacement, 15% of CBA as sand replacement in concrete is the optimum amount in order to get favourable strength and environment saving. For the water permeability test, as the water penetration depth and porosity value increased the water permeability coefficient was also increased since they have a good correlation. It can be concluded that the addition of CBA in concrete will increase the porosity value since the CBA was classified as porous material.

REFERENCES

Abdulhameed Umar Abubakar and Khairul Salleh Baharudin (2012), Properties of Concrete Using Tanjung Bin Power Plant CBA And Fly Ash, International Journal of Sustainable Construction Engineering & Technology (ISSN: 2180-3242) Vol 3, Issue 2.

Abu Bakar A.U and Baharudin K.S (2012), Potential Use of Malaysian Thermal Power Plants CBA In Construction, Dept. Of Civil Engineering, International Journal of Sustainable Construction Engineering & Technology (ISSN: 2180 3242) Vol 3, Issue 2.

Bakoshi T, K. Kohno, S. Kawasaki, N. Yamaji, (1998), Strength and Durability of Concrete Using CBA As Replacement for Fine Aggregate, ACI Spec. Publ. (SP-179) (1998) 159–172.

British Standards Institution (1992), Specification for Aggregates from Natural Sources for Concrete, London: BS 882. 1992.

British Standards Institution (2009), Testing of Hardened Concrete, Compressive Strength of Test, London: BS EN 12390-3:2009.

Faisal Sheikh Khalid, Mohd Irwan Juki, Norzila Othman, Mohd Haziman Wan Ibrahim. (2017), Pull-Out Strength of Polyethylene Terephthalate Bottle Fibre in Concrete Matrix. Malaysian Construction Research Journal, 21(1), 75-85.

Faisal Sheikh Khalid, Mohd Irwan Juki, Norzila Othman, Mohd Haziman Wan Ibrahim, (2016) Flexural toughness of ring-shaped waste bottle fiber concrete. MATEC Web of Conferences, pp 47, 01002.

Givi, A.N and Rashid, S.A (2010), "Assessment of The Effects of Rice Husk Ash Particle Size on Strength, Water Permeability and Workability of Binary Blended Concrete", Construction and Building Materials, 24(2), 2145–2150.

Hamzah, A.F., Ibrahim, M.H.W., Jamaluddin, N., Jaya, R.P., Arshad, M.F., Abidin, N.E.Z. (2015), Fresh Characteristic And Mechanical Compressive Strength Development Of Self-Compacting Concrete Integrating CBA As Partial Fine Aggregates Replacement, International Journal of Mechanical and Mechatronics Engineering, 15 (4), 61-67.

Jaturapitakkul C and Cheerarot R (2003), Development of CBA As Pozzolanic Material. Journal of Materials in Civil Engineering, Jan-Feb. ISSN 0899-1561. Vol. 15, 48-53.

Kadam M.P and Patil Y.D .(2013), "Effect Of CBA As Sand Replacement On The Properties Of Concrete With Different W/C Ratio", International Journal of Advanced Technology in Civil Engineering, ISSN: 2231-5721, Volume 2, Issue 1, 2013, 45-50.

Lovell CW, Ke TC, Huang WH, and Lovell JE (1991), Cba As Highway Material, 70 thAnnual Meeting of The Transportation Research Board" Transportation Research Board.

Jamaluddin, N., A.F. Hamzah and M.H Wan Ibrahim (2016), Fresh Properties and Flexural Strength of SCC CBA. International Conference on Civil and Environmental Engineering for Sustainability 1-6.

Neville, A.M. (1995) Properties of Concrete. Third Edition, New York, U.S.A, Wiley and Sons, pp. 433-443.

Razak, H.A., Seong, W.H., Kian, C.H. (2008), The Potential of Calcined Malaysian Kaolin As A

Pozzolanic Admixture for Concrete, Malaysian Construction Research Journal, 3(2), 21-36.

Ramachandran, V. S., and Beaudoin, J. J. (2001), Handbook of Analytical Techniques in Concrete. In: Concrete Science, New York: Chem Tec Publishing.

Ramadhansyah, P.J, Abu Bakar, B.H., Megat Azmi, M.J., Wan Ibrahim, M.H., (2011), Engineering Properties of Normal Concrete Grade 40 Containing Rice Husk Ash at Different Grinding Times, International Journal of Technology, Volume 2 (1), pp.10-19.

Siddique. R (2003), Effect of Fine Aggregate Replacement with Class F Fly Ash on The Abrasion Resistance of Concrete, Cement and Concrete Research, 2003, Vol. 33, pp. 1877-1881.

Steam (1978), CBA and Its Generation and Use. 39th edition. New York: Babcock &Wilcox.

Wan Ibrahim, M.H, Zainal Abidin, N.E, Jamaluddin, N., Kamaruddin, K. and Hamzah, A.F. (2016), CBA – Potential Use in Self-Compacting Concrete as Fine Aggregate, ARPN Journal of Engineering and Applied Sciences, Vol 11, No.2, pp 34-45.

GUIDE TO AUTHORS

Aims and Scope:

The Malaysian Construction Research Journal (MCRJ) is the journal dedicated to the documentation of R&D achievements and technological development relevant to the construction industry within Malaysia and elsewhere in the world. It is a collation of research papers and other academic publications produced by researchers, practitioners, industrialists, academicians, and all those involved in the construction industry. The papers cover a wide spectrum encompassing building technology, materials science, information technology, environment, quality, economics and many relevant disciplines that can contribute to the enhancement of knowledge in the construction field. The MCRJ aspire to become the premier communication media amongst knowledge professionals in the construction industry and shall hopefully, breach the knowledge gap currently prevalent between and amongst the knowledge producers and the construction practitioners.

Articles submitted will be reviewed and accepted on the understanding that they have not been published elsewhere. The authors have to fill the Declaration of the Authors form and return the form via fax to the secretariat. The length of articles should be between 3,500 and 8,000 words or approximately 8 – 15 printed pages (final version). The manuscripts should be written in English. The original manuscript should be typed one sided, single-spacing, single column with font of 11 point (Times New Roman). Paper size should be of Executive (18.42 cm x 26.67 cm) with 2 cm margins on the left, right and bottom and 3 cm for the top. Authors can submit the manuscript:

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CODIFICATION AND APPLICATION OF SEMI-LOOF ELEMENTS FOR COMPLEX STRUCTURES

Ahmad Abd Rahman[1,2], Maria Diyana Musa[2] and Sumiana Yusoff[2]
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[2] *Institute of Ocean and Earth Sciences (IOES), University of Malaya, Malaysia*

Abstract (Arial Bold, 9pt. Left and right indent 0.64 cm.)
Damage assessment …… ……(it should be single paragraph of about 100 – 250 words.)

Keywords: *Finite element analysis; Modal analysis; Mode shape; Natural frequency; Plate structure*

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Heading 2 (Arial Bold + Lower Case, 11pt)

Heading 3 (Arial Italic + Lower Case, 11pt)

Body Text: Times New Roman, 11 pt. All paragraph must be differentiated by 0.64 cm tab.

Units: All units and abbreviations of dimensions should conform to SI standards.

Figures: Figures should be in box with line width 0.5pt. All illustrations and photographs must be numbered consecutively as it appears in the text and accompanied with appropriate captions below them.

Figures caption: Arial Bold + Arial, 9pt. should be written below the figures.

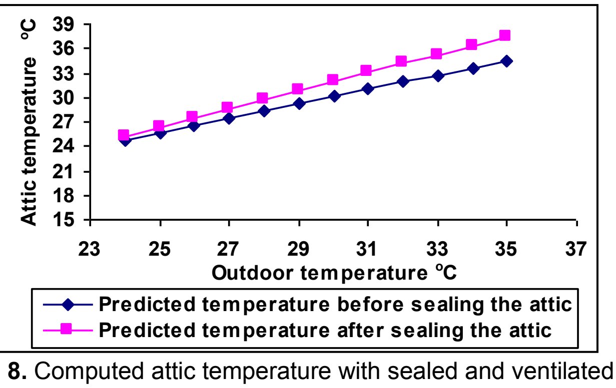

Figure 8. Computed attic temperature with sealed and ventilated attic

Tables: Arial, 8pt. Table should be incorporated in the text.

Table caption: Arial Bold + Arial, 9pt. Caption should be written above the table.

Table Line: 0.5pt.

Table 1. Recommended/Acceptable Physical water quality criteria

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

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

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

Journal

Sze, K. Y. (1994) Simple Semi-Loof Element for Analysing Folded-Plate Structures. *Journal of Engineering Mechanics*, 120(1):120-134.

Books

Skumatz, L. A. (1993) Variable Rate for Municipal Solid Waste: Implementation, Experience, Economics and Legislation. Los Angeles: Reason Foundation, 157 pp.

Thesis

Wong, A. H. H. (1993) *Susceptibility to Soft Rot Decay in Copper-Chrome-Arsenic Treated and Untreated Malaysian Hardwoods*. Ph.D. Thesis, University of Oxford. 341 pp.

Chapter in book

Johan, R. (1999) Fire Management Plan for The Peat Swamp Forest Reserve of North Selangor and Pahang. In Chin T.Y. and Havmoller, P. (eds) *Sustainable Management of Peat Swamp Forests in Peninsular Malaysia* Vol II: Impacts. Kuala Lumpur: Forestry Department Malaysia, 81-147.

Proceedings

Siti Hawa, H., Yong, C. B. and Wan Hamidon W. B. (2004) Butt Joint in Dry Board as Crack Arrester. *Proceeding of 22nd Conference of ASEAN Federation of Engineering Organisation* (CAFEO 22). Myanmar, 55-64.

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MCRJ Volume 30 | No.1 | 2020

Contents

ISSN 1985-3807
9 771985 380005

eISSN 2590-4140
9 772590 414000