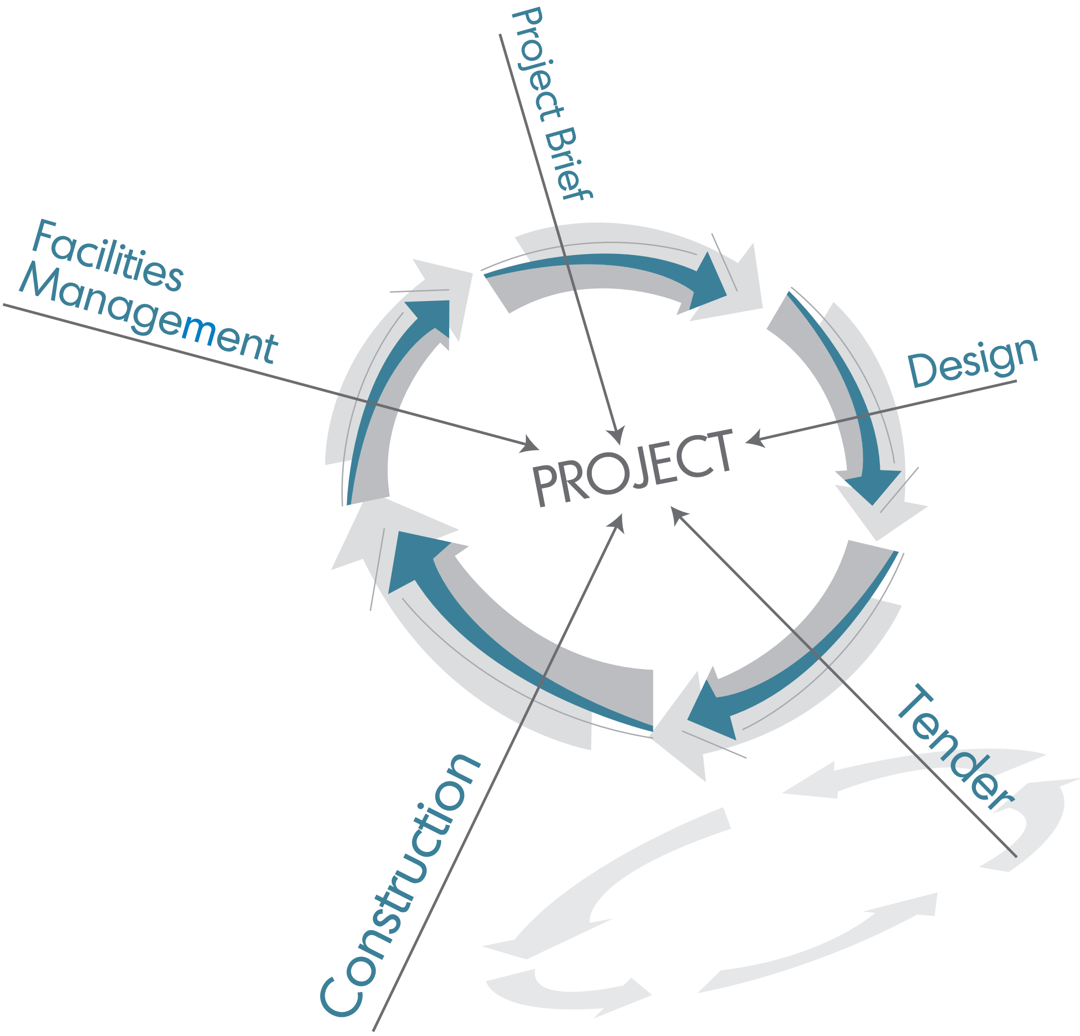
MCRJ Volume 31 | No.2 | 2020

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

Malaysian Construction Research Journal



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MALAYSIAN CONSTRUCTION RESEARCH JOURNAL (MCRJ)

Volume 31 | No. 2 | 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,
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MALAYSIA

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Contents

Editorial Advisory Board

Editorial

Welcome from the Editors

Welcome to the thirty-one (31st) issue of Malaysian Construction Research Journal (MCRJ). In this issue, we are pleased to include seven 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:

Herawati Zetha Rahman et al., investigated the potential of railway development in Cikarang, Indonesia by considering potential passengers and transportation mode choice among the people. The research employs mixed methodology which are quantitative and qualitative approaches through questionnaire survey and in-depth interview. The result shows that the shifting of transportation from private vehicles is varied on the previous experience in conducting journey. The rail-based transportation has potential passenger about 1.7 million in 2016 in Cikarang.

Mazlina Mohamad et al., analysed the ground response in offshore Terengganu. Four sites of stiff oil have been selected in this research. The analysis conducted is through correlation equations to determine the shear wave velocity. This study also conducts analysing local site effect through one dimensional nonlinear ground response analysis program, called NERA. The result shows that the occurrence of soil amplification ranging from 1.6 to 2.0 in the study areas.

Ou Yang Yu Xin et al., evaluated on the business prospect between insurance companies and Green Certified Commercial Buildings in Malaysia. This research used qualitative method through interview session with top management from insurance companies and Green Certified Buildings. The findings show that Green Certified Commercial Building (GCCB) and insurance companies have strong relationship between each other to create the business prospect. The researcher also conclude that it is a need for GCCB to transfer their risk to insurers completely.

Chong Fung Yun et al., reviewed the design and the test results of precast concrete wall panels with opening on both load bearing and shear walls. The factors affecting the failure mode and crack pattern as highlighted in this research are boundary condition, concrete strength, and slenderness ratio. The failure mode of reinforced concrete wall panels depends on the strength of the wall panels.

Darrien Yau Seng Mah et al., reported a computer modelling effort to guide the barrage operations in flood warning in terms of levels, rates of flow and other parameters. A hydraulic simulation used in this study represents the impact of wetting events on the river system. The researcher used data from January 2004 and January 2009 to give insights to a flood warning system in Kuching. It is suggested that in the absent of a flood flow forecasting, the riverside communities are to be alerted at lower river water levels between 0.8 and 1.0m MSL as compared to the current use 1.37m MSL for flood warning with reference to Batu Kawa.

Siti Nur Fatimah Zuraidi et al., proposed a new assessment method, derived from the current rating system to assess the building condition and rating the seriousness of each defect. This study used exploratory mixed method research through Delphi technique and Analytic Hierarchy Process (AHP). Three main criteria is assess which are the building structure, building fabric and building service of 3 museum in Kota Bharu. It is concluded that, to preserve the heritage building, condition assessment of building structure is essential to identify the potential existing heritage buildings for hazard mitigation, disaster preparedness and prior knowledge of potential hazards.

Yee Hooi Min et al., investigated the form-finding of membrane surface bordered by Costa through nonlinear analysis method. It is found that the form-finding to converge for the case of half Costa tensioned membrane structure models in XZ-plane, models with the larger size opening, smaller size opening and mathematically defined opening in the initially assumed shape.

Editorial Committee

IMPROVEMENT TRANSPORTATION CONNECTIVITY OF RAIL-BASED INFRASTRUCTURE AT CIKARANG, INDONESIA

Herawati Zetha Rahman[1], Jade Sjafrecia Petroceany[1], Perdana Miraj[1], Erna Savitri[1], Randika Dwirahman[2], Yusuf Abdurrachman[2], Adi Subandi[3], Ali Sunandar[4], Fadli Kurnia[1] and Gerci Fairio[1]
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Abstract
As part of Bekasi Regency, Cikarang has potential economic sector such as trade, services, and many scales of the industry. However, public access to Cikarang relies on road infrastructure, especially toll roads. Rail-based transportation is required to mitigate heavy congestion in the area and provide reliable transportation for the people. The research aims to investigate the potential of railway development in Cikarang area by considering potential passengers and transportation mode choice among the people. The research combines quantitative and qualitative approaches through a questionnaire survey and in-depth interview. The result shows that rail-based transportation has potential passenger about 1.7 million in 2016 in Cikarang area. The shifting from private vehicles is varied on their previous experience in conducting journey. Car user will switch to rail-based transport when the ticket price is reasonable. On the other hand, bus user will use the rail-based transport when it offers faster time than the bus. Last, motorcycle user will only use the rail-based transport when it proposes lower ticket price and faster travel time. Further investigation of alternative routes, financial simulation, and policy as well as regulatory framework must be conducted to propose the comprehensive implementation of rail-based transport in Cikarang area.

Keywords: *Cikarang area; connectivity; potential; rail infrastructure; transportation*

INTRODUCTION

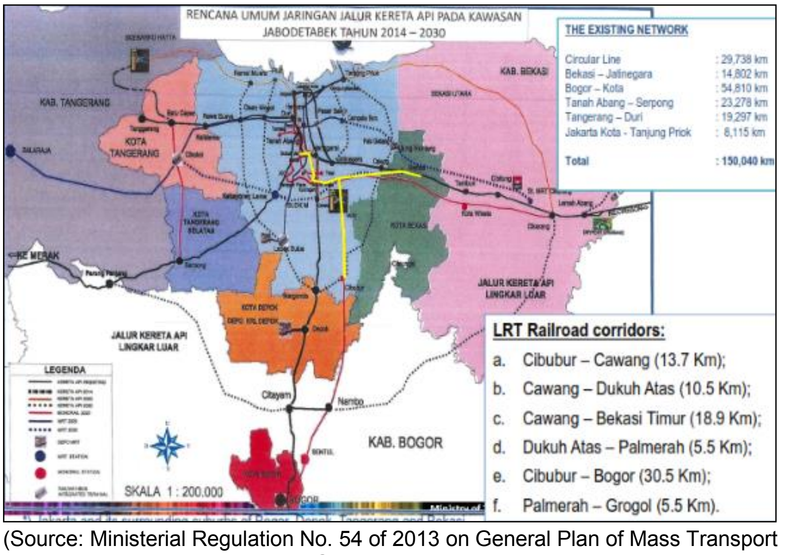
Development of urban areas in Jakarta, Bogor, Depok, Tangerang, and Bekasi (commonly called as Greater Jakarta) is significantly expanding during the past decades (Simone, 2014). Thus, borders are no longer an obstacle for people movement and social interaction. Greater Jakarta is home to 28 million people during the day and 10 million people at night (Winarso et al., 2015). They are commuting not only in one area but also among those cities. Jakarta Transportation Statistics data in BPS (2015) shows that 3,566,178 people in Greater Jakarta commute daily.

Bekasi is one of supporting area for the urban transportation and agglomeration in greater Jakarta. The municipality is also experiencing rapid growth, particularly in Cikarang area. As part of Bekasi Regency, Cikarang has potential economic sector such as trade, services, and many scales of the industry (BPS, 2014). As the economic activity growth rapidly, Cikarang is not supported by a reliable transportation system. Currently, access to and from Cikarang is mainly supported by road infrastructure, especially toll roads. The roads are experiencing a massive congestion at day and night due to a high volume of the private vehicle which leads to longer travel time and creates more stress for the people (Berawi et al., 2015). The volume-per-capacity ratio data for the toll roads around Cikarang shows a high number of passenger units per hour. It means the capacity of roads no longer able to accommodate vehicles adequately. Developing alternative infrastructure through a rail-based transportation is required to connect Cikarang to other areas in shorter travel time.

The aim of this research is to investigate the potential of railway development in Cikarang area by considering potential passengers and transportation mode choice among the people. The result will generate passenger demand from the city and potential of shifting modes from private vehicle users to the rail-based transportation.

Greater Jakarta Railway Network Plan 2014-2030

In the General Plan of Railways Network of Jabodetabek Area 2014-2030, there are several plans for rail-based transportation routes that connect several central of economic and social activities. Therefore, the Directorate General of Railways, Ministry of Transportation has undertaken to establish several routes for increasing transportation capacity in Jabodetabek as outlined in Ministerial Regulation No. 54 of 2013 on the general plan of the mass transportation network in the Jabodetabek urban area as shown in Figure 1.



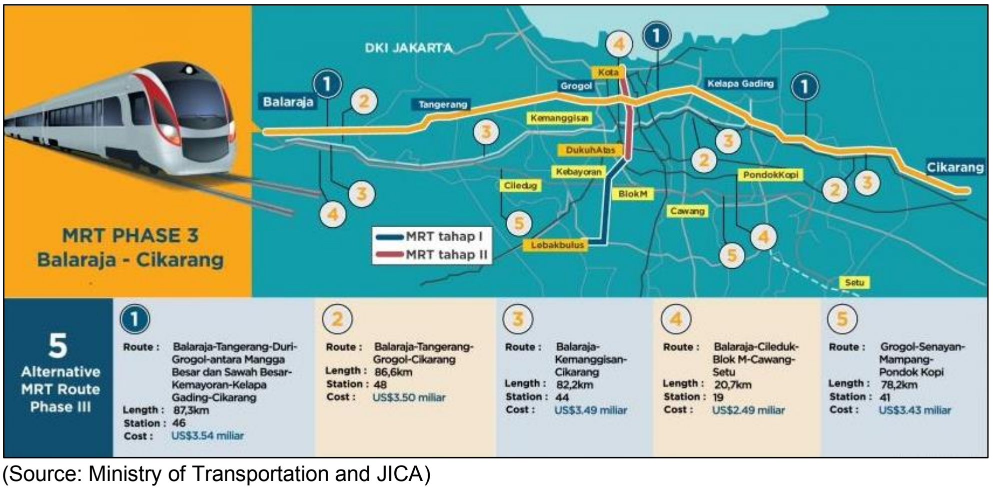
(Source: Ministerial Regulation No. 54 of 2013 on General Plan of Mass Transport Network in Greater Jakarta, Indonesia)

Figure 1. Jabodetabek Railway Service

Reviewing Rail-based Transportation System in Cikarang Area

Currently, rail-based transport in Cikarang area has been expanded from double track construction between Jakarta to Cikarang. It aims to make a commuter line runs independently apart from long distance rail service. The rail connects four stations consist of Bekasi Timur, Tambun, Cibitung, and Cikarang. The service expected to reduce travel time between Jakarta and Cikarang from 2 to 3 hours using private cars to one and half hours. The total financing is about 2.3 trillion rupiahs which divided between state-owned enterprise in railway (35%) and state budget (65%).

On the other hand, the government also aims to develop the second corridor of rapid mass transit (MRT) that connects Cikarang and Balaraja. The first phase is under construction and expected to finish in 2018. The first corridor is divided into two phases which are Lebak Bulus to Hotel Indonesia and Hotel Indonesia to Kampung Bandan. The route of the second corridor spans about 80 km of elevated lines and 7 km of underground when entering the capital city of Indonesia. The plan can be seen in Figure 2.



(Source: Ministry of Transportation and JICA)

Figure 2. Alternative Route of MRT Second Corridor

RESEARCH METHODOLOGY

This study uses a qualitative and quantitative approach to generate the targeted result (Karim et al., 2007; Woodhead and Berawi, 2008; Rahman et al., 2016). Firstly, the literature review was conducted by evaluating related regulation and policies, investigating previous studies related to the development of rail-based infrastructure in the Cikarang area, and benchmarking similar concept implemented in the world.

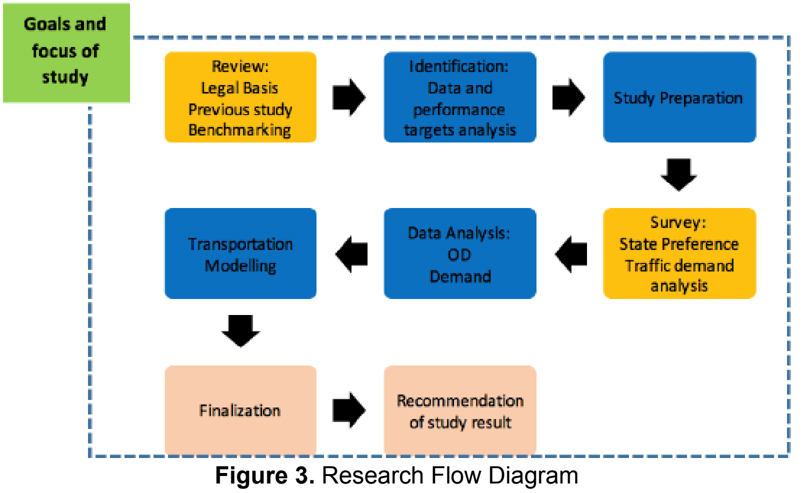


Figure 3. Research Flow Diagram

Firstly, reviewing legal basis on previous study benchmarking will suggest performance analysis for further evaluation. Secondly, a survey questionnaire was conducted to generate passenger movement in the area and to produce a user preference for traveling. Instrument survey was structured using multiple choice approach and deliver to 100 potential passengers. Finally, the data will be analysed using origin-destination (OD) matrix, trip demand analysis, and transportation modelling (Bell, 1991; Yang et al., 1992). The research flow is shown in Figure 3.

RESULT AND DISCUSSION

Characteristics of Movement Patterns and Travel Demand

The study considers two provinces such as Jakarta and West Java to investigate the magnitude of demand. The pattern of passenger movement is divided into two types, internal and external area. The internal zone consists of City/Regency close to the area such as Jakarta province, Bogor regency, Bekasi Regency, City of Bekasi and City of Depok. On the other hand, the external zone consists of Tangerang and middle part of West Java. The detail zoning for this study is shown in Table 1.

Table 1. Zoning of the Study Area

Zone Region	City/Regency/Part of Province	Remarks
Internal	DKI Jakarta	Includes Central Jakarta, East Jakarta, West Jakarta, North Jakarta and South Jakarta
	Bogor	
	Bekasi	Includes Bekasi Regency and City of Bekasi
	Depok	
External	Tangerang	Includes City of Tangerang and South Tangerang Regency
	Middle part of West Java	Includes Karawang Regency and Nearby District

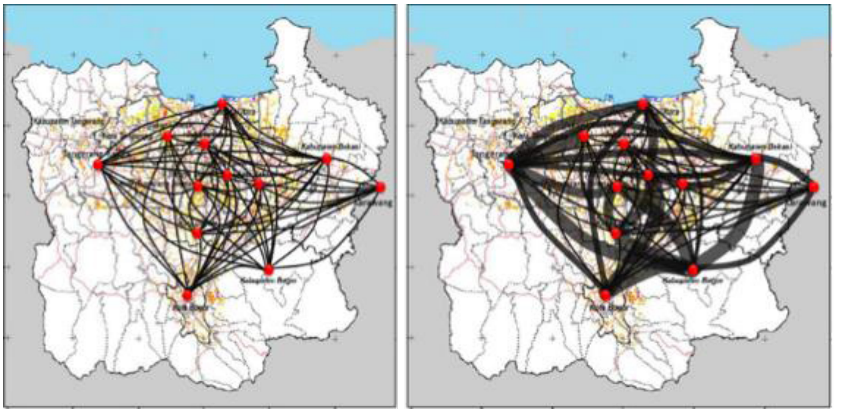


Figure 4. *Desire line* Passenger Movement in 2016 (left) and 2046 (right)

Origin-destination of national transportation plan in 2011 and the coverage of Cikarang zoning shows total passenger movement of Cikarang is about 679.43 million people in 2016. It includes users using both private vehicles and public transportation. Internal zoning estimates about 454.24 million people per year and 225.20 million people contribute to external zoning. Using the current calculation, projected passenger movement in 2046 can be generated as shown in Figure 4.

Mode Choice

The modal choice model is used to show respondents' preference in traveling among rail-based transport, private car, motorcycle, and public bus. When comparing the rail-based transport over the private car, the result shows that the probability of switching modes from private car users to rail-based transport mode is range from 55% to 70% as shown in Figure 5. A similar condition also applied to the others simulation such as rail-based transport over the motorcycle and rail-based transport over the bus.

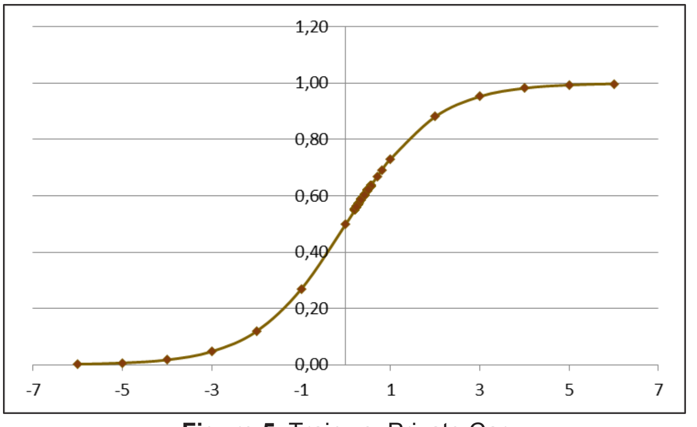


Figure 5. Train vs. Private Car

However, the analysis shows that the ticket price and time travel play a significant role in determining the choice of the user in traveling. When the ticket price is 20,000 rupiah (US$ 1.5) in a single trip, the motorcycle user tends to use their vehicle as shown in Figure 6.

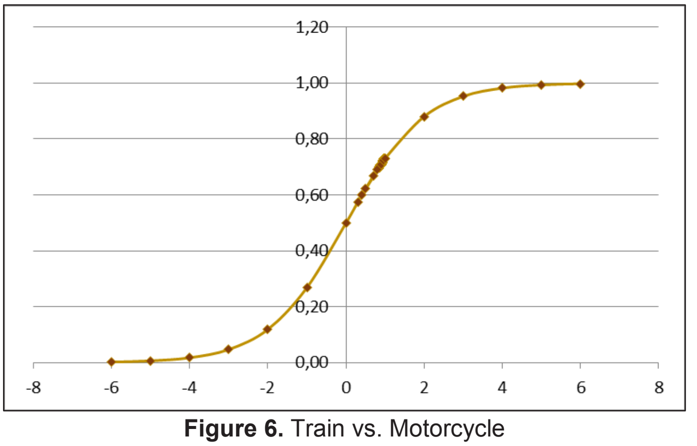


Figure 6. Train vs. Motorcycle

On the other hand, car user will switch to rail-based transport, and bus passenger remains to use the bus. However, when taking into account the similar ticket with faster time travel, all users except motorcycle user agreed to switch to rail-based transportation. The condition might happen because expenditure using the vehicle is much lower than other modes of transportation. A US$ 1.5 can be used for 2 to 3 days of the trip for motorcycle users depends on their home location and trip behaviour as shown in Figure 7.

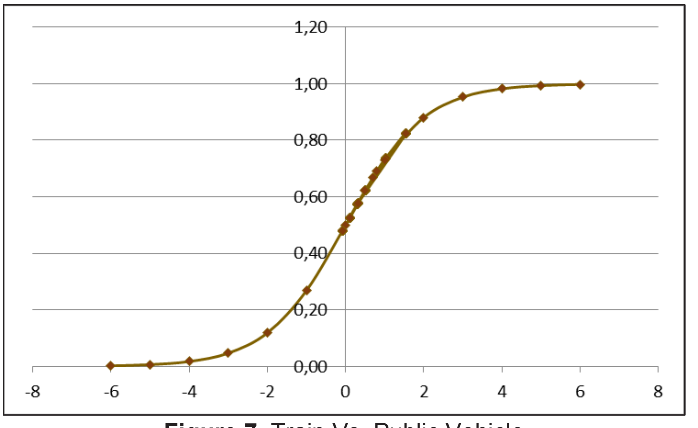


Figure 7. Train Vs. Public Vehicle

The survey shows the probability of shifting from private to public transportation is about varied depends on previous user experience in conducting journey. The people tend to use their current transportation when the ticket price is high and travel time is similar. Disintegrated transportation also leads in their resistance to using public transportation. Current rail-based transport set up 5,000 rupiah or equal to US$ 0.4, but they need to spend more than US$ 2 from railway station to their office.

Using the desired line of passenger movement in Figure 4, social-economic growth in the area and probability of shifting from the people, the rail transportation in the area estimated about 1.7 million passengers in 2016 and became 5.01 million passengers in 2046.

CONCLUSIONS AND RECOMMENDATIONS

Railway development argued one of the solutions to release traffic congestion and offers sustainable transportation for the people. The result shows that rail-based transportation has potential passenger about 1.7 million in 2016 in Cikarang area. The shifting from private vehicles is varied on their previous experience in conducting journey. Car user will switch to rail-based transport when the ticket price is reasonable. On the other hand, bus user will use the rail-based transport when it offers faster time than the bus. Last, motorcycle user will only use the rail-based transport when it proposes lower ticket price and faster travel time.

Further investigation of alternative routes, financial simulation, and policy as well as regulatory framework must be conducted to propose the comprehensive implementation of rail-based transport in Cikarang area.

ACKNOWLEDGEMENT

The authors wish to thank Jababeka Infrastructure for their generous support in conducting this research work.

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GROUND RESPONSE ANALYSIS IN OFFSHORE TERENGGANU

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Abstract
The amount of vibration experienced by any specific location with respect to bedrock in any seismic event can be predicted using a ground motion equation. Results from this analysis are important as it will be used in the design process of engineering structure to withstand a horizontal load from a seismic event. However, by depending on these values, the possibility of amplification or de-amplification of acceleration under the local soil effect has been ignored. The occurrence of amplification will result in higher acceleration on the surface and produce higher lateral load to fixed platforms in offshore Terengganu. These platforms have been designed without provision to seismic load. Therefore, this study focuses on ground response analysis in offshore Terengganu. Four sites have been selected where all sites are considered as stiff soil. Related input data has been extracted from the soil investigation report and Nias-Simeulue earthquake was used as earthquake profile. Four developed correlation equations have been adopted in determining shear wave velocity. One dimensional nonlinear ground response analysis program, namely NERA has been used to analyse local site effect. Peak surface acceleration in each site has been obtained. Results show that the occurrence of soil amplification ranging from 1.6 to 2.0 in the study areas.

Keywords: *Shear wave velocity; Malaysian water; ground response analysis; clay*

INTRODUCTION

The earth's surface is formed by several plates which are moving towards and away from each other. The physical activities such as diverging, converging, and sliding towards each other at the boundary will trigger a seismic event. During this seismic event, seismic waves will be dispersed at the focal point, and moving away in any directions. It can travel in long distance which can cause vibration that can be felt by engineering structure as well as living creatures. The degree of vibration received in any specific location will be depended on the amount of energy release, duration of seismic event and the medium where seismic waves passed through the soil (Monroe and Wicander, 2006).

Ground response analysis (GRA) especially in one dimensional is an established method that has been adopted to study local site effect due to seismic load in any specified location. Shear wave which also known as secondary wave is one of body waves available. The transmission of velocity prior to this wave which passes through any material except for liquid and gasses is known as shear wave velocity (Datta, 2010; Elnashai and Sarno, 2008; Erdey, 2007; Kramer, 1996; Monroe and Wicander, 2006). It is one of the key parameters in GRA that can be determined in-situ or using correlations equations when in-situ data are not available, for example when Cone Penetration Test with pore pressure (CPTu) or Cone Penetration Test (CPT) being used. In offshore area, cone penetration tests have been widely used and has been developed for decades (Lunne, 2012).

In onshore Malaysia, the GRA has been conducted by many researchers in order to improve the design or as a checking whether the existing engineering structure able to resist horizontal load from seismic events (Adnan et al., 2008; Marto et al., 2011; Majid et al., 2013; Tan et al., 2014). Amplification was found with a range of 1.2 to 2.6 at Kuala Lumpur city centre after analysing 12 existing soil data (Adnan et al., 2008). This has led to other important question such as the resistance of fixed jacket platforms in offshore Terengganu which located in Peninsular Malaysia Operation (PMO) to seismic load. Currently, more than 15 fixed jacket platforms operated by PETRONAS Carigali Sdn Bhd (PCSB) are already or almost exceeding their designed service life (Kurian et al., 2014) while others are still within the designed service life. In addition, all these platforms have been designed without provision to seismic load since Malaysian water previously fall in the seismic Zone 0 according to ISO seismic zone. However, based on study conducted in 2008, Malaysian water is actually fall in Zone 1 or 2 based on ISO seismic zone classification (D'Appolonia, 2008). Furthermore, local site effect which might resulted in soil amplification has been ignored in previous study. Therefore, this study is intended to determine ground response analysis in offshore Terengganu. Conducting GRA for these sites is required since the results can be used in further analysis of the platforms reliability under seismic event. The existence of soil amplification can be detected and included in the analysis, if any.

In addressing this issue, four sites in offshore Terengganu have been chosen for GRA. These sites are type D soil based on ISO 19901-2 (International Standard, 2004). Related input data required for Nonlinear Earthquake Sites Response Analysis for layered soil (NERA) have been gathered from soil investigation reports. Calculation is required for shear wave velocity and total unit weight of the study area. GRA has been conducted using NERA program. Results of peak surface acceleration (PSA) has been analysed and further discussed in results and discussion section.

LITERATURE REVIEW

Offshore Terengganu is selected as study area due to the existence of fixed jacket platforms which mostly are ageing and have been designed without provision to seismic load. In terms of ground response analysis, further review of literatures has been presented herein. This section comprises of ground response analysis in general and CPT-Vs correlation.

Ground Response Analysis

Ground response analysis (GRA) allows relevant parties to evaluate local site effect at any proposed or existing development area. This analysis can be conducted up to 3-Dimensional. However, 1-Dimensional GRA is widely used due to its compatibility in replicating local site effect with minimum requirement of input data. It can be divided into linear, equivalent linear and nonlinear methods (Afshari and Stewart, 2015; Basu and Dey, 2016; Govindaraju et al., 2004; Kramer, 1996; Kumar et al., 2014). Since it is 1-Dimensional, only thickness is accountable, the soil boundaries will be assumed horizontal and the length will be ignored (Irsyam et al., 2008). 2-Dimensional and 3-Dimensional analysis normally will be conducted upon request for engineering structure which required special attention. More input data will be necessary for these analyses (Datta, 2010; Marto et al., 2011; Wair et al., 2012).

There are a lot of programs being developed by previous researchers to ease GRA such as SHAKE and EERA for an equivalent linear method and NERA for nonlinear method. As up to date, the existing programs have been updated and new programs were created. In this study, NERA (Nonlinear Earthquake Sites Response Analysis of layered soil) has been adopted. It has been used to determine local site effect of Malaysia and also Indonesia. In addition, the advantages of NERA have been discussed by previous researchers (Adnan et al., 2008; Irsyam et al., 2008; Marto et al., 2011; Zamri and Adnan, 2016). Details on NERA program has been reported by Bardet and Tobita (2001). In this study, time history for seismic event has been obtained from Malaysian Meteorological Department. All the input data required in NERA were obtained from soil investigation report except for shear wave velocity. Further explanation on shear wave velocity has been provided in next subsection.

CPT-Vs correlation

Shear wave velocity (Vs) can now be measured in-situ by using the Seismic Cone Penetration Test (SCPT). However, this method required attention from expertise and higher cost which therefore led to the usage of other method such as Cone Penetration Test (CPT) or Cone Penetration Test with Pore Pressure (CPTu). Furthermore, SCPT is the newest method as compared to CPT and CPTu. However, the implementation of CPT and CPTu will result in the absence of Vs value. In solving this problem, correlations equations such as CPT-Vs have been developed to determine Vs values (Hegazy and Mayne, 1995; Mayne and Rix, 1995; Mayne, 2007; Mayne, 2006).

Wair et al. (2012) suggested that Vs can be determined by using two types of correlations which are soil type dependent correlation and also correlation developed generally for all types of soil. These correlations normally being developed based on period in geologic time scale (USGS, 2016). In this study, four CPT-Vs correlations developed for all soil (general) and clay (soil type dependent) in Quaternary period have been selected. The main parameters in these correlations are cone tip resistance (qc) and sleeve friction (fs). Details on the chosen CPT-Vs correlation are presented in Table 1.

Table 1. Details on selected CPT-Vs correlations for Quaternary period

Soil Type	Model	Equation for Vs	r^2	Paired Data
Clay	Hegazy and Mayne (1995)	$3.18\ q_c^{0.549}\ f_s^{0.025}$	0.78	229
	Mayne and Rix (1995)	$1.75\ q_c^{0.627}$	0.74	481
All Soil	Hegazy and Mayne (1995)	$(101 \log(q_c) - 11.4)^{1.47}\ (100\ f_s/q_c)^{0.3}$	0.70	323
	Mayne (2007)	$118.8 \log(f_s) + 18.5$	0.82	161

In summary, knowing local site effect by conducting GRA is required, especially for engineering structure, which is high rise, special project or highly cost. This structure must be designed to resist any kind of loading and failure is unacceptable. Offshore platforms fall in this category which provided a significant reason for conducting GRA for existing and also future sites. In this study, NERA has been used in conducting GRA for offshore platforms sites in offshore Terengganu. Four CPT-Vs correlation equations have been selected in providing input data for this analysis. Detail on the methodology adopted has been presented in the next section.

METHODOLOGY

This section focuses on methodology in determining peak surface acceleration (PSa) for four sites in offshore Terengganu as depicted in Figure 1. One borehole data for each site has been used in determining peak surface acceleration. The summary of the adopted research methodology is shown in Figure 2.

Figure 1. Sites location in offshore Terengganu

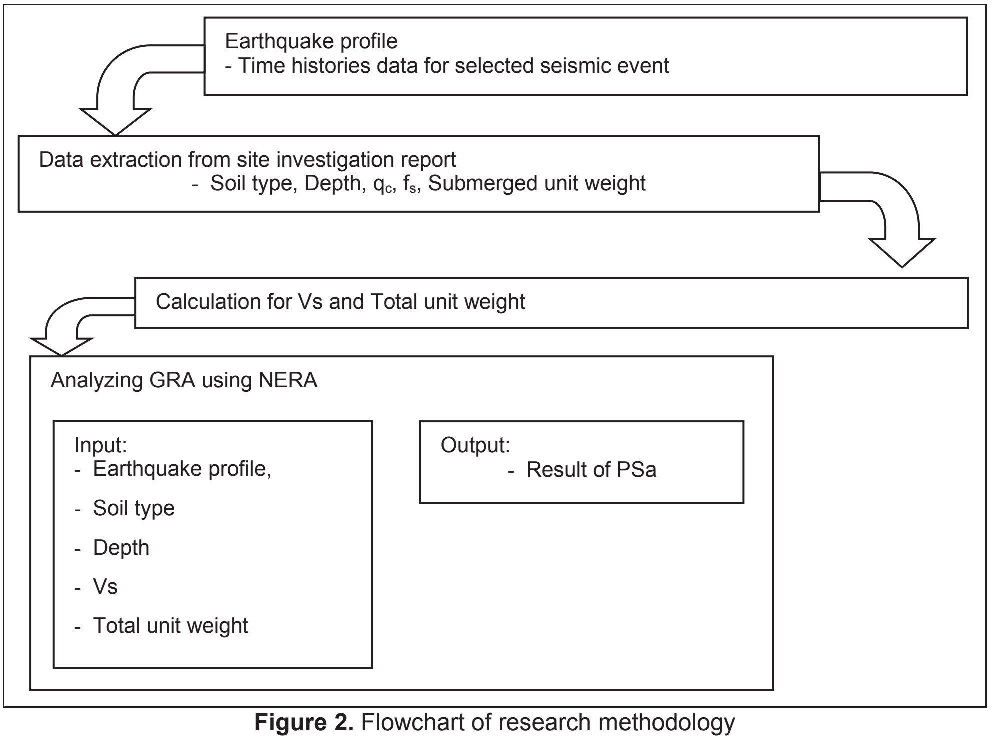

Figure 2. Flowchart of research methodology

In the first phase, data need to be extracted from soil investigation reports which have been conducted using CPTu method. Data such as soil type, depth, cone tip resistance (q_c), friction sleeve (f_s) and submerged unit weight are provided in the reports. Offshore Terengganu is fall in soil type D under ISO 19901-2 (International Standard, 2004) soil classification and in all sites, clay is the dominant soil.

Calculation of shear wave velocity (Vs) and total unit weight have been carried out in the second phase. In obtaining Vs values, four correlations of CPT-Vs as being discussed in the literature review have been used and the average has been taken as final Vs values in order to avoid under or overestimation of the values. Since only submerged unit weight values available in the report, total unit weight for these sites need to be determined. It must be highlighted that these sites are located in offshore area where the sites will be fully saturated. Thus, Equation 1 has been used to calculate the total unit weights for all sites. Since the soil in these sites are fully submerged in sea water, total unit weight is equal to saturated unit weight (Whitlow, 2004).

$$\gamma_{sat} = \gamma' + \gamma_{sw} \tag{1}$$

where; γ_{sat} = Saturated unit weight
γ' = Submerged unit weight
γ_{sw} = Unit weight of sea water

The third phase is analysing using NERA to determine PSa. All the input data such as earthquake time history, soil type, depth, Vs and total unit weight were filled in NERA excel spreadsheets. In this study, seismic event from Sumatera earthquake on 28 Mac 2005 known as Nias-Simeulue earthquake, recorded at KTM stations in both east (HGE) and north (HGN) has been chosen as the input in earthquake spreadsheet. In profile spreadsheet, the numbers and thickness of layers, soil material types, total unit weight and Vs are the input data.

There are three processes in conducting GRA using NERA namely process earthquake data, calculate step-by-step and calculate output. In process earthquake data, input of earthquake time history was read and processed. The earthquake was scaled to maximum desired scale that is 0.03g. During calculate step-by step, the input of profile and material curve were read, and then main calculation was executed based on selected number of iterations. In calculate output for acceleration, velocity and displacement was obtained from calculation time history of displacement, relative velocity and acceleration on surface and bedrock. Results on peak surface acceleration and peak ground acceleration are the output of this analysis. The same procedure has been repeated for all four sites where soil profiles are differed as it depends on borehole data for each site. Acceleration on bedrock and at the surface were used to calculate soil amplification of study areas. The results have been depicted in form of figures in next section.

RESULTS AND DISCUSSION

Results from the 1-Dimensional ground response analysis specifically on peak surface acceleration for sites A, B, C and D were presented in this section. These four sites are layers of clay which fall in stiff soil or soil type D in NEHRP soil profile type. All sites except site A have a depth of 50m borehole. Two-time histories which were recorded in east and north direction have been used in the analysis. Results have been presented in the form of a graph as depicted in Figure 3, 4, 5 and 6.

In Figure 3 and Figure 4, peak surface acceleration for Site A and B under seismic load from east and north direction can be observed at a depth of 0 m. Changes in the profiles and also PSa values can be seen with different seismic load direction. In Site A, load from the east direction produce lower PSa while in Site B, load from north direction produce lower value.

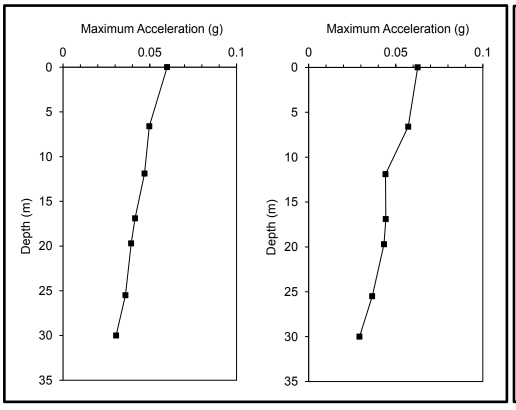

Figure 3. PSa for Site A (HGE and HGN)

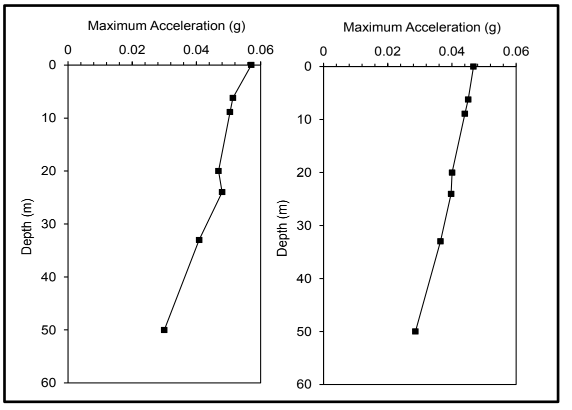

Figure 4. PSa for Site B (HGE and HGN)

Profiles of ground response analyses for sites C and D were presented in Figure 5 and Figure 6, respectively. The profiles show the changes of acceleration values as the seismic wave travel upwards. The rising of the acceleration values as it reaches the surface can be clearly seen. On Site D, there are slight changes in the profiles of both east and north directions. Overall, Site C present the highest PSa values in east direction while site A is higher in north direction. Calculations on the amplification factor have been performed and the results have been tabulated in Table 2.

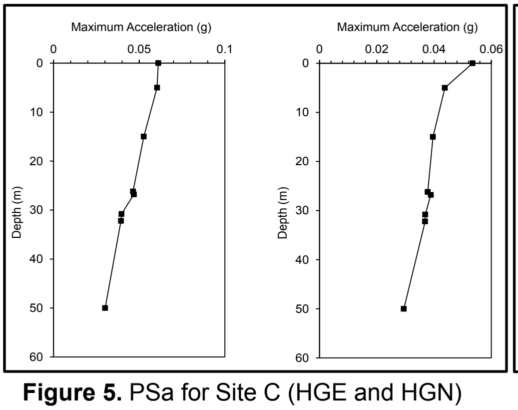

Figure 5. PSa for Site C (HGE and HGN)

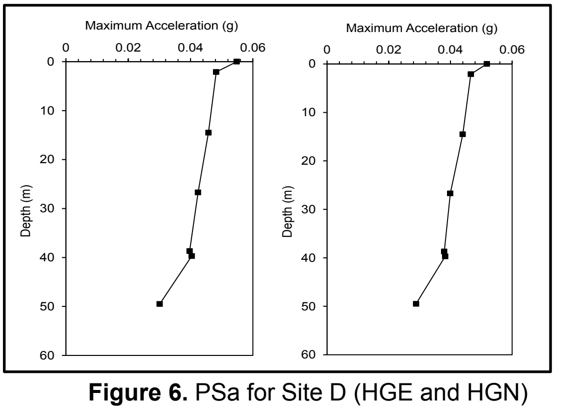

Figure 6. PSa for Site D (HGE and HGN)

Table 2 shows the values of soil amplification factors calculated are more than 1.0 for all sites in both east and north directions. This replicates that the acceleration at the surface which known as PSa has increase has as it travel upward when compared to the acceleration recorded at the bedrock of sites A, B, C and D. Highest increment was found at Site A in north direction where soil amplification factor obtained is 2.1. The lowest factor can be seen in the north direction of site B. The amplification due to soft soil can be seen in these sites. As highlighted by previous researchers (Huang et al., 2010; Iyisan and Khanbabazadeh, 2013; Wang et al., 2004), soil amplification is very important. For structure which have been designed without provision to seismic load and also approaching the end of designed life service such as the offshore platforms in offshore Terengganu, this study will be helpful. Knowing the prediction on how the site will react under seismic event by ground response analysis will help in preparing the appropriate mitigation (Adnan et al., 2008). Comparison of PSa values for all sites has been depicted in Figure 7.

Table 2. Amplification factor for study area in both directions

Site	Direction	Soil Amplification factor
A	HGE	2.0
	HGN	2.1
B	HGE	1.9
	HGN	1.6
C	HGE	2.0
	HGN	1.8
D	HGE	1.8
	HGN	1.8

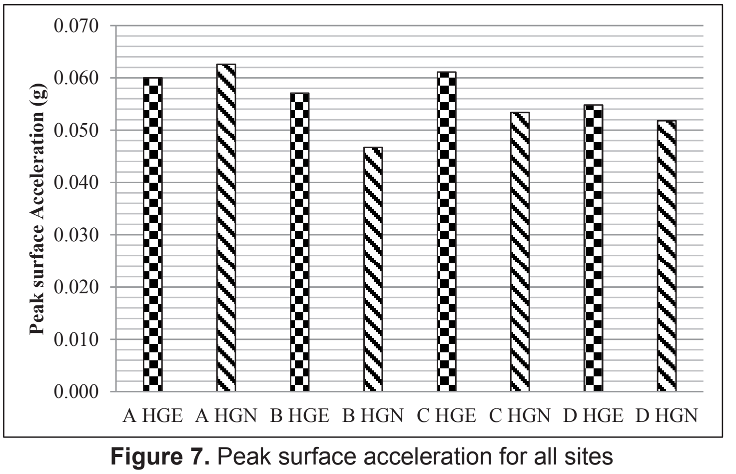

Figure 7. Peak surface acceleration for all sites

PSa resulted from different directions of seismic load has been presented side by side to ease the comparison. Based on the difference with respect to HGE results, the percentage differ for PSa between HGE and HGN has a range of 4.3% to 18.2%. Lowest percentage differs is shown in Site A while the highest percentage differs was found in Site B. Site C and D has 12.6% and 5.5%, respectively. The average of PSa values in eastern and north direction for sites A, B, C and D are 0.0613g, 0.052g, 0.057g and 0.053g respectively. PSa values in all sites range between 0.047g to 0.063g. Thus, these values show that these sites fall in zone 1 of ISO seismic zone classification as suggested by D'Appolonia (D'Appolonia, 2008). Even though these sites are quite closed to each other as can be seen in Figure 1, differences occurred in term of acceleration pattern in soil profile. Albeit they are in the same soil type, under NEHRP soil profile type D, these sites comprise of different Clay layers. The different Clay layers will produce different Vs values which directly affect acceleration transferred to the surface. Therefore, further analysis will be carried out to see local site effect on each site where more data will be analysed.

CONCLUSION

In conclusion, this study is focusing on ground response analysis of offshore Terengganu where PSa values for four sites have been obtained. Upward trend can be observed at all sites. However, mostly different profile can be detected in each site and also in the different load direction. Each site comprises of different combination of clay layers which affect the acceleration in upward direction up to the surface. Ground response analyses were conducted and PSa for each site has been obtained. Calculation of soil amplification factor resulted in amplification with the range of 1.6 to 2.1. Research findings show that amplification exists in offshore Terengganu, therefore more study on ground response analysis for other sites in offshore Terengganu is required. This finding will help in analysing reliability for existing platforms in offshore Terengganu which have been designed without provision to seismic load and also in designing new structures in the offshore area. This paper reported on the preliminary study. Number of data available is one of the limitations faced in this study, especially when the sites are in offshore areas. However, further study currently being conducted where more available data were included.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Kementerian Pengajian Tinggi Malaysia and Universiti Teknologi MARA, Shah Alam, Selangor for awarding research grants and technical supports.

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THE BUSINESS PROSPECT BETWEEN INSURANCE COMPANIES AND GREEN CERTIFIED COMMERCIAL BUILDINGS IN MALAYSIA

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Abstract
Malaysia start to adopt green development in construction when Green Building Index (GBI) was first launched in 2009. Although there are various types of green buildings such as residential, industrial, commercial and township, this paper only focuses on newly built Green Certified Commercial Buildings (GCCB). This is because unlike residential which is an individual owned property where the decision need to be done collectively, the commercial building is a shared property which handled by a management team; thus, the decision to maintain the green certification is done within the management team. Moreover, the Green Certified Commercial Buildings also will secure their green certification for their reputation to increase the market value for business purposes. Although the number of Green Certified Commercial Buildings is increasing rapidly in Malaysia, their insurance coverage remains the same as the conventional property insurance, which does not provide the actual coverage on the status of the GCCB. However, insurance companies such as Fireman's Fund from United States, Zurich Insurance Group from Switzerland and The Green Insurance Company from Scotland start to offer Green Insurance for the green certified buildings in their country. Therefore, this paper will focus on the evaluation of the business prospect between insurance companies and Green Certified Commercial Buildings in Malaysia. The research method used is a qualitative research method because of the novelty of the study and there is not any implementation of Green Insurance in Malaysia yet. Furthermore, the researchers require in-depth understanding on the property and insurance market. The qualitative data collected from interview session with top management from insurance companies and Green Certified Buildings will provide in depth opinion on the business prospect between these two parties. It is essential to evaluate the business prospect between the two parties to ensure that the implementation of Green Insurance in Malaysia is a successful business opportunity.

Keywords: *Green insurance, Green Buildings, property insurance.*

INTRODUCTION

Green certified buildings are designed to amplify the positive effect of the buildings and mitigate the negative impacts of buildings on the environment, natural resources, and human health throughout the entire life cycle of a building. Malaysia has adopted Green Building Index (GBI) which is a green rating tool in 2009 to encourage the development of green buildings in Malaysia. The green rating system has four different levels of certification, which is platinum (the highest), followed by gold, silver and certified.

Although there are six types of green certification building (Non-Residential New Construction (NRNC), Non-Residential Existing Buildings (NREB), Residential New Construction (RNC), Industry New Construction (INC), Industry Existing Building (IEB) and Township (T)) in Malaysia under the certification of GBI, this paper only focus on newly built Green Certified Commercial Buildings (GCCB) which falls under Non-Residential New Construction (NRNC) category. This is because the value of GCCB will appreciate over the years, which make it worth value to be covered comprehensively. Moreover, GCCB is a shared property handled by a management team where the collective decision can be obtained easily in a GCCB compared to the other types of building such as residential and industrial buildings.

Treischmann et al. (2005) state that building and personal coverage form (BPP) is a standard form of general insurance coverage to provide coverage for buildings and personal property. The researchers found that the conventional property insurance such as fire or flood are still largely dominating the Malaysian property market. Although the number of green certified buildings is increasing rapidly in Malaysia, their insurance coverage remains as the conventional property insurance, which does not provide adequate coverage for the status of green certified buildings. It is important for insurance companies to provide adequate coverage for a certified building to prevent the building from losing the green certification when damage occur. Furthermore, some of the features that covered under the conventional property insurance are not required in a Green Certified Building due to the unique green features in it. Therefore, insurance companies need to design a new policy that suits the green certification of the green buildings.

However, when GCCB in Malaysia still cover under the conventional property insurance, Fireman's Fund Insurance from United States, The Green Insurance Company from Scotland and Zurich Insurance Group from Switzerland have started to offer Green Insurance for the green certified buildings in their country. According to Echeverria (2012), Green Insurance is a type of insurance that offer additional coverages for green property owners, which the property has been certified by green rating authorities such as LEED, BREEAM, High Quality Environmental standard and so on.

Therefore, the researcher will evaluate the business prospect between insurance companies and GCCB in Malaysia in this paper. It is essential to evaluate the business prospect between two parties because Green Insurance is a new product which has not been implemented in Malaysia yet. These two prospects are the fundamental relationship between Green Certified Commercial Buildings and insurance companies because without a new business opportunity, insurance companies hardly will look into to provide new policies for GCCB. Moreover, the risk transfer must appear for the GCCB to transfer their unique risk to insurance companies so that the developers or owner of GCCB will have a peace of mind when owning a GCCB.

The rest of the paper is structured as follows: the first part consists of a literature review on green insurance, green building, and business prospect between insurance companies and GCCB in Malaysia. The second section is research methodology of the study, then followed by the third section which is a data analysis and discussion. The last section provides the conclusion of the study.

LITERATURE REVIEW

This chapter consists of three sections which are green insurance, green buildings and evaluate the business prospect between insurance companies and Green Certified Commercial Buildings in Malaysia.

Green Insurance

Kaplan University (2017), mentions that society's interest in environmentally friendly products and services become a critical issue for the insurance industry, especially in the field of sales, underwriting, risk management and claim handling. This is because these green products may present new or unknown risk exposures where the underwriter may have difficulties to measure and evaluate them. Therefore, insurance companies start to offer Green Insurance to provide coverage for green certified buildings in the market.

Green Insurance is a type of first party property insurance coverage, which it is used to cover a green certified building that recognized by any green rating tools such as LEED, BREEAM, High Quality Environmental standard and so on (Kaplan University, 2017). English (2016), further adds that there are two types of Green Insurance (1) covers buildings that already have green certifications (2) offers privilege to replace any losses incurred on the property with more environmentally sensitive materials. In line with the statement made by Kaplan University (2017), Wells (2006), mentions that Green Insurance will only access and provide coverage for buildings the meet the requirement of certain green rating tools and allow the buildings to be restored with green materials or features when covered events occur.

Green Buildings

Yudelson (2008), states that green building is a high-performance property that believes to be able to reduce negative impact on the environment and human health. Bahaudin et al. (2012) further suggest that green building is a green development that can reduce the negative impact of building and construction towards environment and occupants' health significantly. Moreover, Environmental Protection Agency (2016), posits that the practice of green building has done throughout a building's life Cycle from design, construction, operation, maintenance and deconstruction with the implementation of environmental responsibility and resources efficient processes.

Generally, the buildings that entailed for green certification are assessed based on six key criteria:

i. Energy efficiency
ii. Indoor Environment Quality
iii. Sustainable Site Planning and Management
iv. Materials and Resources
v. Water Efficiency
vi. Innovation

Buildings which intent to achieve the green certification have to comply with the requirements set by the green rating tools which based on these six key criteria.

Evaluate the Business Prospect between Insurance Companies and Green Certified Commercial Buildings in Malaysia

There are two business prospects between insurance companies and GCCB. It is essential to examine these two business prospects before the implementation of Green insurance in Malaysia to ensure the success rate of it. This paper only looks into these two business prospects because it is the fundamental business relationship to identify the potential of implementation of Green insurance towards Green Certified Commercial Buildings.

Create New Business Opportunity for Insurance Companies in Malaysia

Moreso (2009), mentions that implementation of government policies has driven the development of green buildings in the market. Moreover, its minimum operating cost is also becoming of the recognize reason for the increasing number of green buildings in the market. Wells (2013), states that the increasing of the green certified building has attracted insurance companies to cash into the business. This is because although green certified buildings are designed to use energy efficiently, less natural resources are being used and provide a healthier environment for occupants, it does not mean that it is free from risks. Therefore, it creates a new opportunity for the insurance industry.

Risk Transfer from Green Certified Commercial Buildings to Insurance Companies

Culp (2004), concludes that risk transfer is a precise process where the shareholders of a firm altered the inimical impacts of a risk to another individual or shareholder of other firms. Risk transfer often can be accomplished through an insurance policy. This is further supported by Vaughan & Vaughan (2008), who mentions that the purchase of an insurance policy is the main approach of risk transfer. This is because in insurance industry there are four main characteristics where risk transfer is one of the characteristics. Generally, the insured will transfer the pure risks to the insurer who typically in a stronger financial status to pay the loss than the insured (Rejda, 2011; Albahori et al., 2017; Abdul Tharim et al., 2017). Hence, people start to have demand and purchase insurance to protect against those risks that are insurable and transfer the financial loss to insurance companies.

RESEARCH METHODOLOGY

Research design is a fundamental planning that the researcher can apply to answer the research questions Saunders et al. (2012). In this paper, the researchers applied exploratory design to carry out the research because the initial intention of this research is to search for new connections, mediation, patterns, themes and codes. In line with the statement made by Law (2017), that exploratory research design is used when the researcher does not have much data to support the theory of the research, this paper is focused on investigating the business prospect between insurance companies and Green Certified Commercial Buildings (GCCB) where insurance companies have not yet promoted any Green Insurance to the industry. Moreover, the researchers had to continuously engage in searching, updating and review the latest information (secondary data) and to obtain primary data through focus - group interviews, in-depth interview, and overall triangulation due to the dynamic evolve of the technological change.

The qualitative research method was adopted in this research project. This is mainly because qualitative research is the recommended method to associate an interpretive philosophy where the researchers are required to get on of the subjective and socially explicit about the phenomenon being studied (Denzin & Lincoln, 2005). Since the researchers required in depth understanding regarding the green insurance industry and Green Certified Commercial Buildings in Malaysia, qualitative research method is selected to conduct this study. Furthermore, Saunders et al. (2012) mentions that qualitative research design can be used to determine participants' meaning and relationship between them. It is in-line with the reason of the researchers selecting this research method because the researchers are investigating the business prospect between insurance companies and GCCB in Malaysia.

The qualitative data were collected through semi-structured interviews. Through semi-structured interviews, the researchers manage to cover a theme or possible key questions for the research study. Moreover, the researchers conducted the interview session by interviewing one-to-one participants to ensure the validity of the data collected. Although Saunders et al. (2016) state that there are several types of question (open questions, probing questions, specific and closed questions and other means to further the questioning) can be formulated in the interview session, the researchers selected open question as the type of question in order to capture a better understanding about the business prospect between insurance companies and GCCB in Malaysia. The sample questions that asked in the interview session are as below:

Question for interview session:

1. What is your position and experience in construction industry/insurance industry?
2. What do you know about Green Buildings and insurance coverage for these buildings?
3. Do you think it is essential for the Green Buildings to have their own insurance coverage?
4. How do you evaluate the business prospect between Green Buildings and insurance companies?
5. How far do you think Malaysia is ready for implementation of Green Insurance?

The respondents selected for the data collection are focused on top management from general insurance companies and GCCB management team through judgmental sampling method. This is in line with the statement made by Saunders (2012), that judgmental sampling is based on the judgement to select cases that will best enables to answer the research questions. Therefore, the researchers selected these respondents because they are the expert in the field of property and insurance industry. Their expert opinion would help to answer the research questions. There were nine respondents in this research where five from general insurance companies and four from GCCB management team. According to Patton (2002), there are no rules for sample size in qualitative inquiry because the sample size depends on what the researcher want to know, the purpose of the inquiry, what's at stake, what will be useful, what will have credibility and what can be done with available time and resources. Hence, the researcher stops at 9 respondents because the respondents have answered what the researcher wants to know. The designation of each of the respondents is shown in the table below.

Table 1. The designation for each of the respondents

Position	Company's Name
Deputy Branch Manager 1	Allianz General Insurance Malaysia (Allianz)
Assistant Executive 1	MSIG Malaysia (MSIG)
Agent 1	AIG Malaysia (AIG)
Admin and Technical Officer 1	Allianz General Insurance Malaysia (Allianz)
Branch Manager 1	RHB Insurance Malaysia (RHB)
Chief Operating Officer 1	Green REHDA
Project Officer 1	Green Building Index (GBI)
Project Officer 2	Construction Industry Development Board (CIDB)
Maintenance Executive 1	SP Setia Bhd Group (SP)

The researchers adopted explanation building technique to analyse the data obtained and build an explanation for the data. The theory was triangulated with the primary and secondary data to verify the theory. The explanation should involve the comparison of any statements or proposition that has been generated when explanation building is adopted in the analysis process.

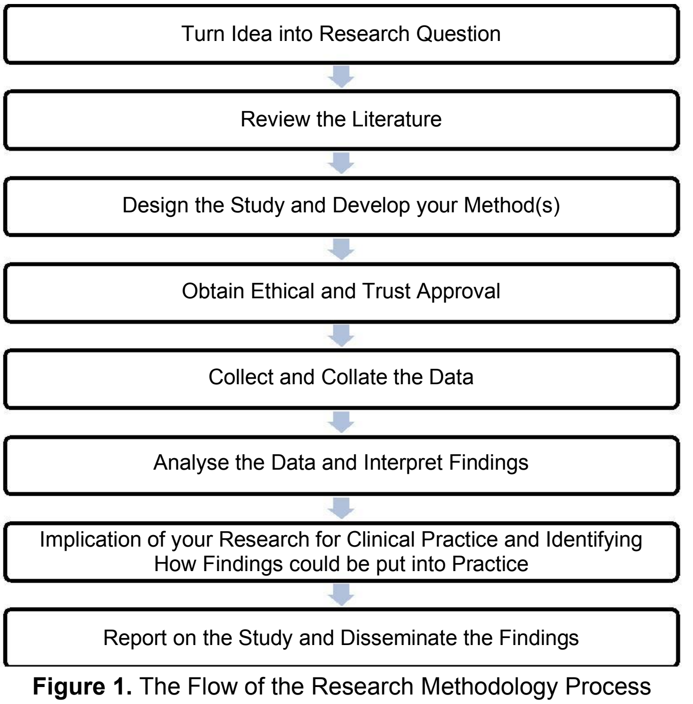

Figure 1. The Flow of the Research Methodology Process

ANALYSIS AND DISCUSSION

This section presents the data collected on the business prospect's aspect through the interview session and explanation about GBI because it is one of the pioneer green certification providers in Malaysia. The researchers discuss about GBI in the first section and followed by the evaluation of the business prospect in second and third section.

Green Building Index

Green Building Index (GBI) was first launched in 2009 to promote green development in Malaysia. Therefore, it is a well-recognized green rating tool in Malaysia's construction industry. Glazette (2012), mentioned that (GBI) is a green rating tool to promote sustainability in the built environment and raise awareness among developers, contractors, planners, and engineers about the negative impact of constructions towards environment and occupants' health.

Bahaudin et al. (2012) show that GBI provides 100 points for each building which has interest to obtain the green certification to fulfil. The buildings must comply the requirements set the GBI to obtain the points for the green certification level. Bahaudin et al. (2012) further add that different level of green certification has different points to achieve.

Table 2. The rating points of green buildings

Points	GBI Rating
86+ points	Platinum
76 to 85 points	Gold
66 to 75 points	Silver
50 to 65 points	Certified

Summarized from the GBI rating system: (Green Building Index, 2013).

Generally, GBI considers six criteria when rating a building. The six main criteria are as follows (Green Building Index, 2013):

i. Energy efficiency
ii. Indoor Environment Quality
iii. Sustainable Site Planning and Management
iv. Materials and Resources
v. Water Efficiency
vi. Innovation

Although there are six types of green certification building (Non-Residential New Construction (NRNC), Non-Residential Existing Buildings (NREB), Residential New Construction (RNC), Industry New Construction (INC), Industry Existing Building (IEB) and Township (T)) in Malaysia under the certification of GBI, this paper only focusses on newly built Green Certified Commercial Buildings which falls under Non-Residential New Construction (NRNC) category.

Create New Business Opportunity for Insurance Companies in Malaysia

Although green buildings are designed to use energy efficiently and reduce the usage of natural resources which generally are more economical to operate, it does not mean that they are free from risk (Moreso, 2009). Moreover, the number of green buildings also increasing due to the implementation of government policy and its low operating cost. Therefore, Wells (2013), states that insurance companies start to seek green construction, practice as a new business opportunity.

Assistance Executive 1 from MSIG mentioned that the increasing number of GCCB in Malaysia due to the National Initiatives had become a huge and potential opportunity for insurance companies to implement Green Insurance in Malaysia.

Table 3. The Number of Non-Residential New Construction

Points	No. of Non-residential New Construction
Platinum (86+ points)	10
Gold (76 to 85 points)	50
Silver (66 to 75 points)	21
Certified (50 to 65 points)	113
Total Certified	194

Sources: (greenbuildingindex, 2017)

The Star Online (2016), added that the Construction Industry Development Board (CIDB) Malaysia and the Real Estate and Housing Developers Association (REHDA) Malaysia had signed a memorandum of collaboration (MOC) to promote low carbon and sustainable development among developers and construction industry in Malaysia. The researchers believe that this cooperation between private and public sector would stimulate better focus on sustainable development and enhance more GCCB development in Malaysia.

Moreover, Agent 1 from AIG, Admin and Technical Officer 1 from Allianz and Branch Manager 1 from RHB also claimed that GCCB is a completely new business opportunity for insurance companies to explore if they are to handle this type of buildings. This is because insurance companies only provide conventional property insurance for GCCB in the current practice. Thus, the insurance coverage offered does not secure the status of the GCCB as a green certified building. They further added that it is a preferred business because generally GCCB has lower building risk which eventually leads to lower claim rate and increase the profit margin of the insurance companies. This is because the GCCB will still pay the premium, but the possibility to claim has been mitigated due to the characteristics of the lower building risk.

Chief Operating Officer 1 from Green REHDA highlighted that green insurance for GCCB will be a new market because it has not practiced in Malaysia yet. This is mainly because the history of developing green buildings in Malaysia still quite short (since 2009 when Green Building Index first launched in the market). Chief Operating Officer 1 further added that if green insurance for green buildings can be implemented in oversea such as Fireman's Fund Insurance Company from U.S. and Zurich Insurance Group from Switzerland, it has the potential and possibility to implement in Malaysia. It could be another initiative to encourage and enhance the development of green certified buildings in Malaysia (Maintenance Executive 1 from SP).

Furthermore, according to Douglas G. Hotchkiss, owner of the Dallas, Texas-based Hotchkiss Insurance agency, the market has more than 50 percent of the builders are building green that open up an opportunity for the insurance agency to look into the business potential (Wells, 2006). Therefore, Fireman's Fund Insurance Co. Released a new product to provide coverage specially for certified green building replacement and green upgrade coverages in October 2006. Moreover, based on the article by Walker (2013), it believes that approximate 40-48 percent of new non-residential construction, when measured by value will be environmentally friendly. This situation equates to a $120 billion to $145 billion opportunities for the insurance market. Therefore, the researchers believe that the insurance companies in Malaysia can absorb the experience in oversea context to start to consider implementing green insurance to GCCB in Malaysia.

Based on the discussion, the researchers believed that GCCB is an upcoming trend in Malaysia and the number of GCCB will continue to increase tremendously with the encouragement of National Initiatives in greening the construction development. Hence, it is a positive and a potential market for insurance companies in Malaysia to explore. GCCB will open up a relatively new business opportunity for insurers in Malaysia because of its unique characteristics that lead to lower claim rate. Therefore, the researchers stated that the pioneer insurer who start to implement Green Insurance for GCCB has the potential to obtain most of the market share of GCCB; thus, the economic and financial status of the company can be enhanced to a higher level. Directly, it will increase the confidence level of the insured who choose to be insured under the insurance company. It will be a breakthrough in term of sales and product if insurer start to offer green insurance policy for GCCB in the market.

Risk Transfer from Green Certified Commercial Buildings to Insurance Companies

Purchase of an insurance policy is the main approach of risk transfer (Vaughan & Vaughan, 2008). This is because there are four main characteristics in the insurance industry where risk transfer is one of the characteristics. Furthermore, Rejda (2011), suggests that by purchasing an insurance policy, the insured will transfer the pure risks to the insurer who are generally stronger in term of financial status than the insured to compensate the loss.

Agent 1 from AIG stated that although GCCB has lower building risk due to the installation of advance green features, it does not mean the GCCB is free from risks. This is because risk occurs unintentionally in daily life which it applies to GCCB as well, although most of the risks have been mitigated from the green design. Agent 2 from AIG and Deputy Branch Manager 1 from Allianz further added that GCCB which has lower building risks does not show that it does not need any insurance coverage because when covered event occurs to a GCCB, it might suffer a larger loss in term of financial compared to a conventional commercial building (CCB) due to the installation of advance green features to achieve the green certification.

> "Although not all the green aspects are covered in the current conventional property insurance, GCCB should continue transfer their risk to the insurer because at least part of the risk is covered comprehensively by the insurance policy. The impact of loss in term of financial is not as serious as without any insurance coverage."
>
> (Branch Manager 1 RHB)

In line with the statement made by Agent 1, the researchers agree that although GCCB presents a lower building risk, it does not mean that GCCB does not need to transfer the risk to insurers to minimize the financial lost when covered events occur. Moreover, the green features that used in GCCB might suffer from a larger loss compared to CCB due to economic uncertainties or expensive price of the green features when covered event occurs. Thus, it is better for insurers to absorb the loss rather than GCCB's owners have to bear themselves.

Project Officer 1 from GBI posited that having Green Insurance for GCCB is a good idea to transfer the unknown risks or unique risks to insurance companies by paying a certain amount of premium. The respondents further added that although currently GCCB is covered under conventional property insurance, GCCB which is relatively new in the market might expose to risks that have not encountered before by any parties (insurance or construction) in the industry. Moreover, Chief Operating Officer 1 from Green REHDA mentioned that GCCB would not like to lose their green certification, which might affect their reputation when lost it or pay huge amount of reconstruction sum when covered events occur. Therefore, the effective way to mitigate the possibilities is to transfer the risks completely to the insurers.

> "GCCB can consider retaining their risk instead of transfer to insurer if they have confidence in the buildings' risk. Even if they want to transfer, the amount of coverage may differ because the risks are well mitigated by the green technologies and designs."
>
> (Project Officer 2 from CIDB)

Based on the discussion, it showed that it is necessary for GCCB to transfer the risks completely to insurance companies although generally they have lower building risks. This is because risks occur unintentionally and unexpectedly. Moreover, by transferring the risk through Green Insurance can avoid the possibility of GCCB to lose their green certification when covered events occur to the GCCB. It also can prevent the GCCB owner or management team suffer from the huge amount of reconstruction sum because the insurers will bear the risks when GCCB has relocated the risks completely to them. Therefore, the researchers agree that Green Insurance is a good method for GCCB to transfer their risks comprehensively to insurers to avoid loss of certification or out-of-pocket issues to happen when an encounter with covered events.

CONCLUSIONS AND RECOMMENDATION

The researchers conclude that Green Certified Commercial Buildings (GCCB) and insurance companies have strong relationship between each other to create the business prospect. From the findings in this study, the researchers agree that GCCB is a new business opportunity for insurance companies to explore. This is because the number of GCCB in Malaysia is increasing rapidly over the time, thus, it opens a relatively new and potential market for the insurers. Moreover, GCCB currently is covered under the conventional property insurance, which does not provide the actual coverage on the status of the GCCB. Furthermore, GCCB which equipped with advance green features might expose to certain unknown risks that caused the GCCB management team to face unique demands in insurance coverages. Therefore, insurance companies should start to consider offering specially designed Green Insurance to provide adequate coverage for GCCB.

Based on the findings in this study, it shows that it is a need for GCCB to transfer their risk to insurers completely. Although there are studies proven that GCCB generally have a lower building risk compared to conventional commercial buildings, it does not mean that GCCB is free from risks. Some of the buildings which go for higher certification such as gold or platinum will install certain advance green features which might have unique or unknown risks. Hence, the researchers agree that it is essential for GCCB to transfer their risk to insurers comprehensively to avoid loss of green certification when covered events occur on the green features.

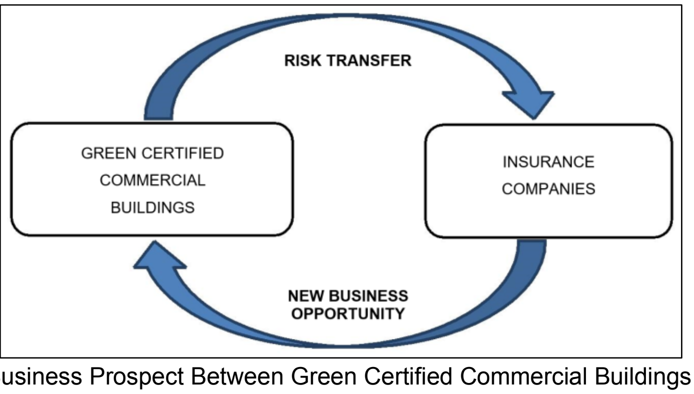

Figure 2. The Business Prospect Between Green Certified Commercial Buildings and Insurance Companies

The researchers believe that the first insurance company that launch Green Insurance in Malaysia will have a positive increase in the reputation because it shows that the insurer has started to appreciate the effort taken by building and construction industry in reducing the negative impact on environment and occupant's health. Besides, it also provides the GCCB an opportunity to transfer their risk comprehensively to the insurance company. This will help the GCCB to secure their green certification even when covered events occur; hence, provide a peace of mind to the GCCB owner or management team. Therefore, the researchers conclude that it will be a breakthrough in term of sales and products if insurers start to promote Green Insurance in the market.

Limitations of a study will become a barrier and affect the conclusions of the study. The limitation of this study is the honest answers provided from the respondents, and the researcher assumes the respondents have adequate knowledge about this topic, albeit this is an exploratory research.

In the future recommendation, the researchers would suggest implementing green insurance to the other types of building such as residential, industrial, and institutional. This is because the number of green certifications for these buildings is increasing at a promising level. Thus, it will soon open a business opportunity for insurance companies. Moreover, these green certified buildings also will encounter unique risk with the application of green features in the buildings. In addition, Moreover, it can be suggested to minister departments such as Kementerian Tenaga, Teknologi Hijau dan Air (KeTTHA), Construction Industry Development Board (CIDB) and Malaysia Green Building Council (MGBC) to cooperate with insurers to implement GI to safeguard the Green Buildings in Malaysia.

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A REVIEW ON REINFORCED CONCRETE WALL PANELS WITH OPENING

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Abstract
Industrialised Building System (IBS) is a term used in Malaysia to denote precast construction method arising from the demand in shifting the realm of conventional construction method into the era of automation. Precast concrete wall panels with opening are important structural elements in buildings because openings are created in the wall panels for the provision of doors, windows as well as services purpose. This paper reviews the design and the test results of precast concrete wall panels with opening on both load bearing and shear walls. The design parameters highlighted are the boundary conditions, geometric characteristic and the configuration of size and the location of the opening on wall panels. In general, the openings in a wall panel affect its structural behaviour.

Keywords: *Concrete wall panel; opening; design parameters; behaviours.*

INTRODUCTION

Industrialised Building System (IBS) is a term used in Malaysia for a technique of construction where-by components are produced in a controlled environment, either at site or off site, placed and assembled during construction works. Over the years, precast concrete wall panels are widely use in the construction industry. The demand for precast product particularly on precast concrete wall panels is continuously increasing in IBS. Reinforced concrete wall panels are usually used as load-bearing structural elements because these concrete products are able to resist high compressive stresses. Therefore, a precast reinforced concrete wall panel is acceptable as an important structural element apart from slabs, beams and columns. This acceptance is due to the increase in research undertaken by the researchers on the behaviour of concrete wall panels to improve and uplift their performance in construction industry.

Eurocode 2 (2014) defines a wall as reinforced concrete wall panel with a length to-thickness ratio of 4 or more, in which the reinforcement is taken into account in the strength analysis. Reinforced concrete wall panels have several advantages when they are used in IBS. By using this type of walls, some structural components such as beams, and columns can be eliminated. Consequently, it may help to reduce the dead loads on the building which in turn reduce the cost of footings and foundations. Reinforced concrete wall panels are easy to design and use in fast track project. The reduction in time of construction will save the cost of construction at the same time.

In order to satisfy the demand in construction, functional modifications need to be made on the precast concrete wall panels. These modifications refer to the door and window openings, and also openings for ventilation system, electricity and water pipe. Figure 1 shows a cut out opening for a window on the concrete wall. Therefore, modification on reinforced concrete wall panels is a necessity. It is more beneficial both environmental and economic to make these modifications of existing building than demolishing them and build up a new building. Therefore, the concrete wall panels with opening have become important structural elements in construction works. The opening in the reinforced concrete wall causes local cracking around the opening which leads to decrease in load carrying capacity (Mohammed et al., 2012).

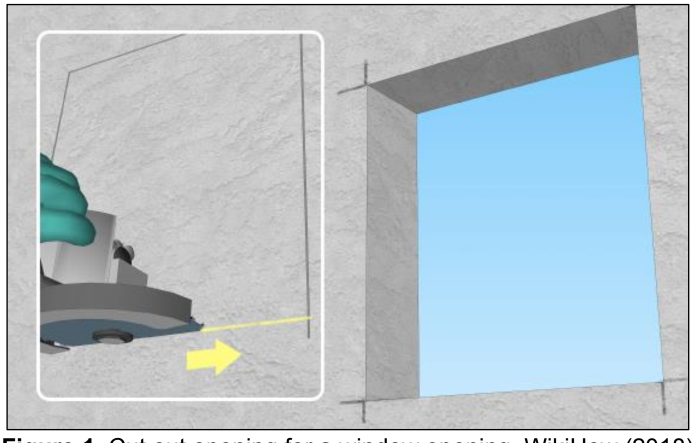

Figure 1. Cut out opening for a window opening, WikiHow (2018)

Generally, there are three types of openings in walls, which are, existing openings, enlarged existing openings and newly created openings (Popescu et al., 2015b). Existing opening is an opening that has been deliberately created in the factory. Once the location of the opening has been identified, the reinforcement can be placed around the opening according to the design method. An enlarged existing opening is the opening that cut the surrounding wall to enlarge the opening. Newly created opening is an opening that is modified or created in an existing reinforced concrete wall. A cut-out opening has no design method. The opening is cut-out from an existing concrete wall panel. Previous researchers have studied on the behaviour of reinforced concrete wall panel with openings with different boundary condition, aspect ratio, size and position of opening and loading conditions (Saheb and Desayi, 1990a; Lee, 2008; Doh et al., 2010; Guan et al., 2010; and Fragomeni et al., 2015). However, the study on cut-out opening on reinforced concrete wall is less explored and the research on the subject is very limited. The cut-out opening on the concrete wall effects the structural capacity and stress distribution of the concrete wall panel due to the cut-off and reduction of reinforcement. Popescu et al. (2015a) studied the behaviour of cut-out opening on concrete wall panel and found that fibre-reinforced polymer can increase the strength of the wall panel as externally bonded reinforcement. The change in behaviour of the reinforced concrete wall panel with opening influenced the load carrying capacity of wall panels, but none came to conclude regarding the ideal behaviour and the optimal size of existing opening of reinforced concrete wall panels. Recently, Jaseela and Pillai (2017) found the optimum shape of cut-out opening on concrete structural wall panel. Majority of the previous researches studied the effect of boundary conditions in one-way and two-way wall but the study on the effect of load eccentricity and reinforcement ratio are less pronounced.

This paper is to review the designs and the results of precast concrete wall panels with opening in both one-way and two-way actions which are subjected to centrally and eccentrically applied axial loads and seismic loads. This paper also reviews the effect of sizes and the locations of the opening to the properties of the shear wall and compression wall.

COMPRESSION WALL

Concrete wall is a structural member with a length to thickness ratio of 4 or more and which the reinforcement is taken into account, Eurocode 2 (2008). There are two types of concrete walls, which are load bearing walls and shear walls. If the loading is applied vertically on a wall, it is a load bearing wall. If lateral load acts, on a wall it is a shear wall. The results in the previous researches are summarised in terms of boundary condition, configurations of opening, reinforcement ratio, loading, finite element modelling, slenderness and aspect ratio of wall panels.

Effect of boundary condition on strength

A concrete wall can either be one-way wall or two-wall wall depending on the support conditions. A concrete wall panel that is supported laterally and restrained along top and bottom edges are referred to as one-way action panels. However, the wall panels restrained along three or four sides are referred to as two-way action panels. The supports of the concrete wall panel refer to the support from other wall, roof, floor slab and other forms in real structures as shown in Figure 2.

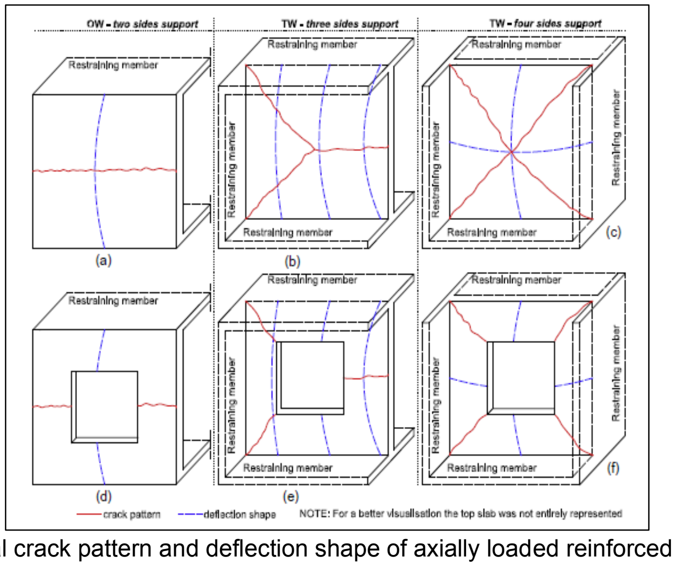

Figure 2. Typical crack pattern and deflection shape of axially loaded reinforced concrete walls (Popescu et al., 2015b)

Boundary conditions have a main influence on the failure mode and cracking pattern of the concrete wall panels (Saheb and Desayi, 1990a; Lee, 2008; Doh et al., 2010; Fragomeni et al., 2012; Doh et al., 2014; and Guan et al., 2010). Eurocode 2 (2008) and AS3600 (2009) are the only major codes that consider lateral restraint along the side edges of a concrete wall.

Lee (2008) carried out both experimental study and LFEM analysis for high slenderness ratio wall panels. The experimental results showed that the strength for two-way wall is 2.5 to 3.5 times higher than one-way walls. LFEM analysis also showed the same results that the two-way walls increased strength due to side restrains. These results revealed that the strength of two-way concrete wall decreases when the opening area on the wall panel increases. Additionally, Guan et al. (2010) also conducted a similar study in LFEM analysis. The results also showed that two-way walls gave an improved axial strength. Both researches concluded that the effect of the boundary condition can be ignored when the opening ratio is large, that is, $A_0/A \geq 0.67$, where A_o = area of opening; and A = wall area without opening.

Doh et al. (2010) studied and compared on six half-scale reinforced concrete walls with slenderness ratio of 30 supported on two, three or four sides with various opening configurations. All panels were subjected to a uniformly distributed axial load at an eccentricity. The results show lateral supports have changed the ultimate strength of wall panels with various opening location. Additional side support increases the strength of the wall. If the opening is near to an unsupported edge, it behaves like a two-way supported wall panel. The two-way wall panel has higher ultimate strength than the one-way wall panel. Doh et al. (2014) conducted the study using the same properties of wall panel in Doh et al. (2010) by comparing the strength ratio of solid panel and panel with central opening. The results concluded that there is an increase in axial strength of 40% to 70% by providing two-side and three-side supports. Fragomeni et al. (2012) also concluded that two-way walls have about 2 to 4 times higher failure load than one-way walls in an experimental study.

On the other hand, an experimental study carried out by Saheb and Desayi (1990a) revealed the opposite results as compared to the other researches. The deflection of two-way wall panels was reported to be 40.5% to 78.7% smaller than one-way wall panels. However, the cracking loads for one-way wall panels showed neglected effects with 7.3% to 16.2% higher than two-way wall panels and similarly to the ultimate loads with only 3.0% to 7.3% higher than two-way wall panels. It was concluded that the effect of the boundary condition is small because the cracking loads and ultimate loads for one-way and two-way wall panels were almost equal.

The effect of opening on strength

The opening in concrete wall panels refers to the provision of windows, doors and ventilation system. The ultimate strength of the reinforced concrete wall panel will be affected due to the creation of openings. The effect of small opening is not considered because the reinforced concrete walls have the ability to redistribute stress. However, the large opening created in a wall may change the stress distribution within the wall and adversely affects its behaviour due to considerable amount of concrete and reinforcing steel being removed. Figure 3 shows the maximum principal stresses in a wall panel of a computer simulation. It can be seen that the presence of the opening in precast wall panels will lead to stress concentrations resulting in cracking.

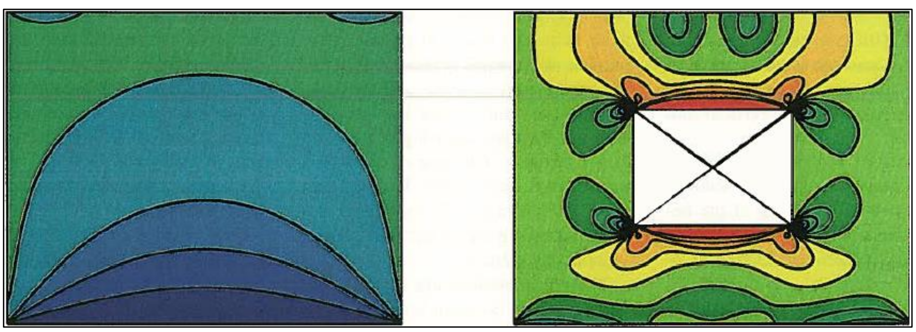

Figure 3. Stress contours in the wall panel with opening and without opening (Michael et al., 2002)

The varying configuration of the opening in reinforced concrete wall panel will also change the behaviour of the reinforced concrete wall panel. The change in location of the opening will change the eccentricity of the reinforced concrete wall panels and tend to influence the ultimate strength of wall panels, Popescu et al. (2015b).

The effect of opening size

There is no design guideline or recommended detailing in code of practice for concrete wall with opening. However, Eurocode 2 (2008) and AS3600 (2009) have recommended the limit for the height of the opening to be a third of the height of the wall or the area of the opening is less than 1/10 of the area of the whole wall so that it could be designed as a solid wall. The effect of small opening is not significant because the reinforced concrete walls have the ability to redistribute stresses. However, a large opening created in a wall may change the stress distribution within the wall and adversely affects its behaviour because there is considerable amount of concrete and reinforcing steel removed. The varying configuration of the opening in reinforced concrete wall panel may change the behaviour of the reinforced concrete wall panel as well. These changes being size and location of the opening will change the eccentricity of the reinforced concrete wall panels and tend to influence the ultimate strength of the wall panels (Saheb and Desayi, 1990a; Lee, 2008; Guan et al., 2010; Fragomeni et al., 2012). Mohammed et al. (2013), Popescu et al. (2015a) and Jaseela and Pillai (2017) studied on the behaviour of cut out opening on reinforced concrete wall panels. On the other hand, the effect of the number of opening to the concrete wall panels was carried out by Fragomeni et al. (2012).

Saheb and Desayi (1990a) carried out a test on twelve concrete walls with opening in both one and two-way action. Each panel had a window or a door opening at different location. The wall panels used in the experiment have the slenderness ratio equal to 12 and aspect ratio equal to 0.67. The test parameters of the opening are number of opening, size and location of the opening. The walls were tested with uniformly distributed eccentric loads. The conclusion to these tests was that the failure of the concrete panels with opening was induced by the buckling of the slender column strips adjacent to the opening.

Lee (2008) and Fragomeni et al. (2012) studied the effect of the number of opening on the ultimate strength of precast concrete wall panel. Both studies stated that the failure load decreases when the number of the opening increases. Lee (2008) conducted the experiment with seventeen two-way concrete wall panels with opening. The walls were tested with both one and two window openings and concluded that the failure load decreases with the increasing number of opening. The decrease in axial strength ratio of wall was 77% for one opening and 54% for two openings when the opening ration, A_o/A, was increased from 0.25 to 0.5. Lee (2008) also studied on the door-type openings and concluded that the axial strength decreased by 26% in two-way action. It was also found that an asymmetric opening location led to a decrease in axial strength ratio. However, both experimental studies did not have a control sample of solid wall without opening. Therefore, there is no relevant results to show how the openings affect the failure load of the walls.

Mohammed et al. (2013) and Popescu et al. (2015a) studied the effect of cross-sectional area of the cut-out opening to the wall panels. Both experimental studies concluded that the increase in cross sectional area of the opening will decrease the failure load. Jaseela and Pillai

(2017) continued the investigation of cut-out opening and study on the optimum shape of cut-out opening on concrete structural wall. A static structural linear analysis using ANSYS software to find the influence of cut-out opening in one-way and two-way wall panel was conducted. The results of the study were accomplished by twenty-eight wall panels with different opening shape. The shape is varied from rectangular with narrow height (wide rectangular opening) to narrow width (slit rectangular opening) and a comparison was also made with square opening. The study concluded that the optimum shape of the opening in a one-way wall panel is slit rectangular wall opening at the middle of the wall, whereas square opening is the most optimum shape that can be cut on the two-way wall panel. The behaviour of the wall panel is in ambiguity because the conclusion was made only on total deformation of the wall panel but not on ultimate strength, cracking pattern and failure mode.

Mohammed et al. (2013) tested one-way concrete wall panel with cut-out openings of size varying from 5 % to 30 % of the solid wall panel. This study found that the disturbed zones formed due to the cut-out opening caused high stresses in the concrete. Discontinuities in the disturbed zones formed cracks at the corner if it is improperly reinforced. The results show that the failure load decreases as the cross sectional of the opening increases. However, this experimental study also did not include the control solid wall, so no conclusion can be drawn on how the opening size affects the failure load.

Pospescu et al. (2015a) carried out an experimental study on high strength two-way walls and solid wall. The concrete wall panel with small opening which had cross sectional area of 25% registered a decrease in failure load by 36%. However, the concrete wall panel with large opening which had cross sectional area of 50% decreased in failure load by 50%. Pospescu et al. (2015) explains the decrease in the failure load could be affected by the piers of the concrete wall, where a pier is defined as the column-like cross section of the remaining the wall area. The larger the opening, the smaller the piers. The decrease in the aspect ratio of the piers led to the restraints being more active and therefore the material strength is utilized more effectively. The decrease in the failure load does not decrease linearly with the increase of the size opening. This is because the results show that double the cross-sectional area in the wall from small to large opening does not give half the failure load. Pospescu et al. (2015a) and Jassela and Pillai (2017) found that the strength of the wall panel with opening was affected by the pier or the size of the remaining wall. The larger opening causes the piers width of the remaining wall to become narrow.

Linear finite element analysis (LFEM) was carried out by Lee (2008) and Guan et al. (2010) with the purpose to study how the dimensions of opening affect the axial strength of the wall. Guan et al. (2010) did a linear finite element analysis of two-way walls with window and door openings. The properties of the wall panel were identical with those in the experimental study conducted by Saheb and Desayi (1990a). A parametric study on the effect of the height and length of opening on wall panel was investigated. The results show that the strength of the wall panel decreases by 86% when the opening area of 25% was increased to 67%. The increase in length of the opening to the total length of the wall, L_o/L, from 0.25 to 0.67 decreases the load carrying capacity by 50% for two-way walls. However, the increase in height of the opening to the height of the wall from 0.25 to0.67 reduces the ultimate load carrying capacity by 14% for two-way walls. Guan et al. (2010) found that the response of the ultimate load capacity to the variation in the opening size, height and length is approximately linear and that increasing the height together with the length of the opening has the most critical effect on the axial strength of the wall. This conclusion is in agreement with the LFEM analysis made by Lee (2008). Increasing only the height of the opening has a small effect on the ultimate load carrying capacity.

The effect of number of opening

Fragomeni et al. (2012) studied the behaviour of concrete wall panels with one or two openings. There was a reduction in ultimate strength in the walls with two openings compared with identical walls with one opening. It was also reported that the failure loads of two-way panels with openings are about 2 to 4 times those of similar one-way panels with openings. In general, the test results indicate that failure loads decreased when the number of openings was increased from one to two.

Reinforcement

According to BS8110 (1997), AS3600 (2009), Eurocode 2 (2010) and ACI318 (2011), reinforcement is provided in the concrete wall panels to offset creep and shrinkage effect when the reinforced concrete walls are subjected to axial load. It is generally proved that the contribution of steel reinforcement in a compression wall can be neglected but is necessary to provide the minimum requirement of reinforcement for each wall panel to control cracking. From the various design codes, the reinforced concrete wall panels can be designed by using simplified design method or method based on column theory. Simplified design method is used when only a single layer of reinforcement is provided.

Eurocode 2 (2010) categorises concrete walls to reinforced concrete walls or unreinforced and light reinforced concrete walls. A reinforced concrete wall should be designed according to a strut-and-tie model. Therefore, both horizontal and vertical reinforcement need to fulfil the minimum reinforcement requirement. Eurocode 2 (2010) specifies a minimum vertical reinforcement ratio of 0.2% in wall, and the maximum vertical reinforcement ratio is 4%. When the reinforcement ratio exceeds 2%, strut-and-tie should be used in the wall. The clear distance is mentioned in the design code and half of the amount of reinforcement should be placed at each face. This requirement is used to satisfy structural and durability of walls. For the unreinforced and light reinforced concrete wall, it has less amount of reinforcement than the minimum reinforcement. The unreinforced and light reinforced concrete walls only have a single layer of reinforcement to control cracking due to shrinkage and creep effects in the walls. This single reinforcement does not account for the bearing capacity in the wall.

Experimental studies were carried out by Seddon (1956) and Pillai and Parthasarathy (1997) on single layer reinforcement concrete wall. Both results show the single reinforcement placed in the centre of the wall panel has negligible influence on the strength of the wall. Moreover, same conclusion was obtained from the finite element analysis. Indeed, one of the 3D finite element analysis studies conducted by Somiah et al. (2014) attempted to compare the stresses in concrete wall panels with minimum level of steel reinforcement and plain wall panel revealed that the stress in the reinforcement can be ignored. The results proved that the steel reinforcement in load bearing wall does not contribute to the strength of the wall since the principal stress only increased by 0.05% in the presence of steel reinforcement.

On the other hand, there are also some researches which found out that there is a significant effect of reinforcement to the strength of the wall. There are certain cases that the reinforcement provides additional strength to the wall panel. Seddon (1956) proved that double layers of reinforcement have more effect on the strength of the wall panel. Saheb and Desayi (1990b) conducted an experimental study on two-way wall panel. This study shows that the load capacity of the wall panel increased by 54% if the amount of the vertical reinforcement was increased from 0.33% to 0.85%. It was also concluded that the horizontal reinforcement did not contribute to the load carrying capacity of the wall panel.

Pospescu et al. (2015a) studied the effect of centric reinforcement placed in one layer in an experimental study. The study shows that the reinforcement only contributed to the strength at higher load. When the wall was about to fail, no yielding occurred in the vertical bar while only some horizontal bars yielded. The increase in geometric nonlinearities also led to large deformation of wall panels. The large deformation activated the yielding of the reinforcement and delayed the failure of the wall. So, the failure of the wall only occurred in the pier with lower deformation.

Doh (2002) carried out a parametric study with LFEM of two-way walls with centric single-layer reinforcement. The results show that load capacity of the wall increased for low slenderness wall panel. However, the reinforcement can be negligible for high slenderness wall panel, that is, walls with height (H) to thickness (t_w) ratio of more than 40. For lower slenderness-ratio wall panels, the reinforcement placed in two layers can increase axial strength of the wall panel, instead of centric one-layer reinforcement. Fragomeni et al. (2012), Lee (2008) and El-Metwally et al. (1990) also deduced the same conclusion because the contribution of reinforcement in the tension zone increased the strength of the wall. Fragomeni and Mendis (1997) and Saheb and Desayi (1990b) also made the same conclusion that when the reinforcement is placed in two layers, a significant increase in ultimate strength can be achieved.

Another study on reinforcement in wall panels was carried out by Guan et al. (2010) through a nonlinear finite element analysis. The spacing of the bars differed depending on the opening arrangement for each wall. The spacing of the bar was closer when it was nearer to the opening, and the spacing became larger when it was away from the opening. This method was used to create a reinforced frame system around the opening to control cracking. The results show the strength of the one-way wall models reduces by 92% as the opening dimension increases from 25% to 67% of the wall dimension.

Factors affecting the failure mode and crack pattern

The cracks on a wall panel will lead to failure. Boundary conditions and slenderness ratio were found to be influencing the failure mode of concrete wall with opening (Lee, 2008; and Fragomeni et al., 2012). The difference in concrete strength also to different failure mode (Doh et al., 2010; Fragomeni et al., 2012; and Doh et al., 2014).

Boundary condition

An experimental study carried out by Doh et al. (2010) showed the majority of cracking propagated diagonally from the restrained corners to the opening and then horizontally from the opening to unrestrained edges, indicating typical two-way behaviour close to the restrained ends and one-way behaviour between unsupported edges. Similar investigation was conducted by Doh et al. (2014) with wall panels tested on three-side lateral supports. The results revealed that majority of cracking propagates diagonally from the restrained corners to the opening and then horizontally from the opening to the unrestrained edge. Both researches concluded that the cracking pattern would start from the corner of the opening and extend to the unrestrained edge of the wall panel.

Fragomeni et al. (2012) carried out an experimental study with forty-seven reinforced concrete walls with opening with both one-way and two-way actions. It was found that one-way wall panels with opening showed single curve bending failure and horizontal cracking at the centre of opening. However, the two-way wall panels with opening showed typical curvature bending failure and diagonal cracking from corner edges of opening to the corner of the wall panel.

Concrete strength

Concrete strength is also another factor that affects the cracking pattern and failure mode of the concrete wall panel with opening (Doh et al., 2010; and Doh et al., 2014). Doh et al. (2010) found out that the high strength concrete panels developed a single large crack, commencing at restrained corners at the tension face and then horizontally towards the unrestrained edge. This indicates a brittle failure mode, with possibly some yielding of reinforcement taking place. In contrast, the normal concrete strength panels tested exhibited more ductile behaviour with a number of parallel cracks. Biaxial curvature was evident as a result of the wall being supported on three sides.

Doh et al. (2014) compared the normal strength and high strength concrete wall cracking patterns and illustrated the differences in failure mode of two-way action with three-side lateral supports. The high strength concrete panel developed a single large crack, commencing at restrained corners at the tension face and then splitting in two separate parts near unrestrained region. This indicates that the high strength concrete panels possessed a more brittle failure mode, with some yielding of reinforcement taking place before concrete failure. In contrast, several minor cracks propagated in normal strength concrete panel.

Slenderness ratio

Slenderness ratio of a wall is defined as a function of the effective height divided by either the effective thickness or the radius of gyration of the wall section. The effect of slenderness ration on the ultimate strength of reinforced concrete walls with opening was investigated by Fragomeni et al. (2012) by testing forty-seven reinforced concrete wall panels with opening. In this investigation, it was found that the axial strength ratio for one-way panels gradually decreases when slenderness ratios were increased from 30 to 40. Same conclusion was deduced by Doh and Fragomeni (2006), where the investigation results show that axial strength decreases with increase in slenderness ratio. This conclusion was also proved by Philip and Guan (2007) from a parametric study on fifty-four one-way and two-way concrete wall panels with opening with slenderness ratios between 10 and 50. The results revealed that that axial strength ratio becomes lower for the more slender wall.

In general, walls with a low slenderness ratio may fail by crushing on the compressed face and bending on the tension face, while those with high slenderness ratio may additionally fail through buckling. Brittle types of failure have been observed in all experimental studies performed on low slenderness-ratio walls (Fragomeni et al., 2012; and Doh and Fragomeni, 2006).

Eccentric Loading

Most of the researches subjected the concrete wall panel to a uniformly distributed axial load with an eccentricity of $t_w/6$. The loading with an eccentricity is used to simulate the imperfection of installation in construction. The wall develops tension within a certain zone when the load is applied on the wall with an eccentricity more than $t_w/6$. The portion of the wall cracks slightly at each corner in the section with the assumption of a no tension material. The geometry of the crack section changed with the varying value of eccentricity loading applied as depicted in Figure 4 (Kuddus, 2010).

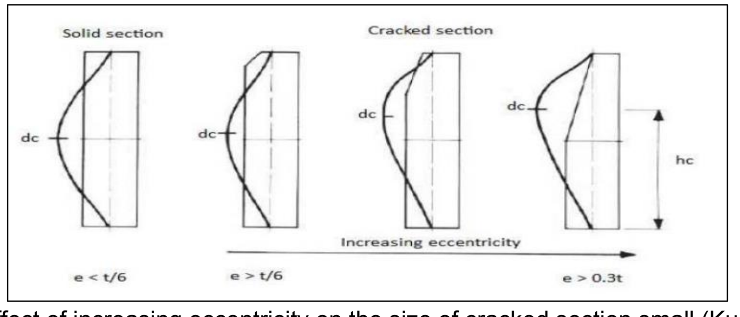

Figure 4. Effect of increasing eccentricity on the size of cracked section small (Kuddus, 2010)

Eccentric load affects the ultimate load and the displacement of the wall panels. The eccentric axial load acting on the wall panel will provide an increase in bending moment resulting in out of plane curvature due second order effect. When the displacement of the wall panel is out of plane, and the eccentric load is continuously increasing, the second order effect occurs as shown in Figure 5.

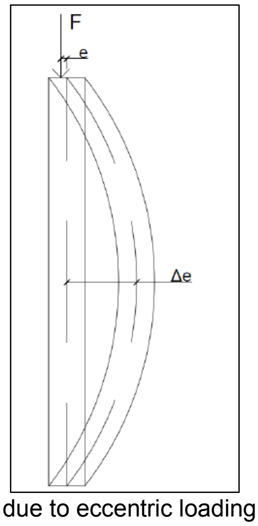

Figure 5. Second order effect due to eccentric loading (Johan and Sebastion, 2016)

Doh (2002) mentioned that only eccentric load acts on wall panels in practice. The centrically load can only be use in theory. Therefore, eccentric loading must be treated in the design procedure because eccentricity affects the failure type of the wall panel. A large curvature occurring in a slender wall and high eccentricity gives a catastrophic failure mode. Doh (2002) provided the results showing failure load for the centric loading was more than double compared to the eccentric loading. LFEM results showed the failure mode for lower eccentricity is crushing whilst buckling failure was recorded for higher eccentricity loading on wall panels.

Design equations for wall panel with opening

Many researches had carried out studies and investigations to improve the existing design method in the design code. There are different design formulae for one-way and two-way wall panels. However, study on the design formulae for one-way and two-way wall panel with opening is scarce (Saheb and Desayi, 1990a; Doh and Fragomeni, 2006; Philip and Guan, 2007; and Guan et al., 2010). Most of the researchers studied on wall panels with low slenderness ratio (< 40) except for Philip and Guan (2007) who studied high slenderness-ratio wall panels with opening. Basically, the effect of the size and location of the opening is taken through a dimensionless parameter, α, accounting for the opening width. Only Guan et al. (2010) changed the reduction parameter from $\propto_x$ to $\propto_{xy}$ to include for the opening height. For the formulae contributed by each researcher, N_{uo} is the ultimate load (kN) for wall with openings; N_u is the ultimate load (kN) for solid wall; A_g is the gross cross sectional area of wall in plane (mm^2); f_{yv} is the yield strength of steel (MPa); A_{sv} is the area of vertical steel in wall (mm^2); x is the opening geometry parameter; x_{xy} is the opening parameter defined by the opening size and location in both horizontal and vertical directions and the spacing between opening; s_x is the opening spacing parameter for walls with two openings located side by side (mm); x_x and x_y is the opening parameter with respect to the influence of opening size and location in the horizontal and vertical direction respectively.

Saheb and Desayi (1990a)

Saheb and Desayi (1990a) conducted an experiment on six two-way wall panels with different types of opening. Each wall panel was acted upon with eccentric load of eccentricity equals to $t_w/6$. The wall panels were insert with two layers of reinforcement symmetrically, one in each face of the wall. Figure 6 shows the definition of parameters in the design equations.

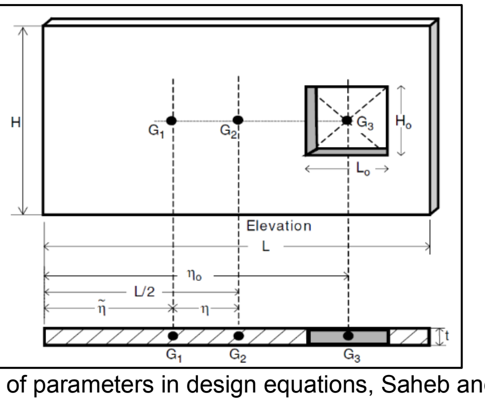

Figure 6. Definition of parameters in design equations, Saheb and Desayi (1990a)

The design formula for ultimate load is given by

$$N_{u0} = (k_1 - k_2 \cdot x) \cdot N_u \tag{1}$$

Where N_u for one-way wall panel

$$N_u = 0.55(A_g\, f'_c + (f_{yv} - f'_c)A_{sv})\left(1 - (\frac{H}{32t_w})^2\right)\left(1.20 - \left(\frac{H}{10L}\right)\right) \text{ for } \frac{H}{L} < 2.0 \tag{2}$$

$$N_u = 0.55\big(A_g\, f'_c + (f_{yv} - f'_c)A_{sv}\big)\left(1 - (\frac{H}{32t_w})^2\right) \text{for } \frac{H}{L} > 2.0 \tag{3}$$

And N_u for two-way wall panel

$$N_u = 0.67A_g f'_c \left(1 - (\frac{L}{120t_w})^2\right)\left(1 + 0.12\left(\frac{H}{L}\right)\right) \text{ for } 0.5 < \frac{H}{L} < 2.0 \text{ and } \frac{L}{t_w} < 60 \tag{4}$$

The constant k_1 and k_2 were obtained from test results by using curve-fitting techniques. It was found that k_1= 1.25 and k_2= 1.22 for one-way wall panels, while k_1 = 1.02 and k_2 = 1.00 for two-way wall panels. The non-dimensional quantity,x is a geometrical opening index which defined in Saheb and Desayi (1990a) is given by

$$x = \frac{A_0}{A} + \frac{\eta}{L} \tag{5}$$

where the area of the wall panel, $A = Lt_w$; the area of the opening, $A_0 = L_0 t_w$; $\frac{A_o}{A}$ accounts for the cross-sectional area of openings in the horizontal plane; $\frac{\eta}{L}$ corresponds to the location of the opening in the horizontal direction; η is the centre of gravity of wall panel with openings with respect to the wall centre, which $\eta = (\frac{L}{2} - \tilde{\eta})$, and $\tilde{\eta} = \frac{\frac{1}{2}t_w L^2 - t_w L_0 \eta_0}{Lt_w - L_0 t_w}$.

Doh and Fragomeni (2006)

Doh and Fragomeni tested wall panels with opening under one-way and two-way actions. The formula proposed for the ultimate load with openings, N_{uo} is same with the formula proposed by the Saheb and Desayi (1990a). The formula proposed for the ultimate load with openings is given by equation (1).

In this case, k_1= 1.188 and k_2= 1.175 for one-way wall panels, while k_1 = 1.004 and k_2 = 0.933 for two-way wall panels. obtained from experimental results; the non-dimensional quantity $\propto$ is defined in Saheb and Desayi (1990a); and the design axial strength per unit length of the wall, $N_u$was same as the formula defined in Doh and Fragomeni (2005), which is

$$N_u = 2.0 * f'_c{}^{0.7}(t_w - 1.2e - 2e_a) \tag{6}$$

in which f'_c = the characteristic compressive strength of concrete from experiment; e = eccentricity of load; and e_a = additional eccentricity due to the out-of-plane deflection of the wall during loading (the P-delta effect) given as $e_a = \frac{(H_{we})^2}{2500t_w}$; where $H_{we} = \beta H$.

For one-way wall panel, the effective height factor, β

$$\beta = 1 \text{ for } \frac{H}{t_w} < 27 \tag{7}$$

$$\beta = \frac{18}{\left(\frac{H}{t_w}\right)^{0.88}} \; for \; \frac{H}{t_w} \geq 27 \tag{8}$$

For two-way wall panel, the effective height factor, β

$$\beta = \propto \frac{1}{1+\left(\frac{H}{L}\right)^2} \text{ for } H \leq L\text{, and} \tag{9}$$

$$\beta = \alpha \frac{L}{2H} \text{ for } H > L \tag{10}$$

In these equations, the eccentricity parameter,

$$\alpha = \frac{1}{1-\frac{e}{t_w}} \quad \text{for } H/t_w < 27\text{, and} \tag{11}$$

$$\alpha = \frac{1}{1-\frac{e}{t_w}} \times \frac{18}{\left(\frac{H}{t_w}\right)^{0.88}} \quad \text{for } H/t_w \geq 27 \tag{12}$$

Philip and Guan (2007)

Philip and Guan (2007) studied on concrete wall panels with opening with slenderness ratio greater than 30. They studied the empirical formulae by linear finite element analysis. The formula for one-way wall panel with opening is same with the formula given by Doh and Fragomeni (2006) and Saheb and Deasayi (1990a) because this formula was proven to give an accurate result with the LFEM. However, Philip and Guan (2007) introduced a restraint factor, λ for wall panel with two-way support, where the proposed design formula for ultimate load for wall panels with openings is given by

$$N_{u0} = \lambda(k_1 - k_2 \cdot x)N_u \tag{13}$$

where λ = restraint factor which is equal to 1 for one-way wall panels with opening, whereas for two-way wall panels with opening, λ is given by

$$\lambda = 0.038 \left(\frac{H}{t_w}\right) + 1.04 \tag{14}$$

The reduction parameter, x which considers the size, number and location of opening was same with the formula in Saheb and Desayi (1990), where x = 0.25 for one-way wall panels with opening and x = 0.5 for two-way wall panels with opening. Factors k_1 and k_2 are 1.188 and 1.175, respectively, which are constants derived from a calibration process.

Lee (2010)

From the previous researcher, the x parameter is used to compensate for the effects of opening, but the effect of opening neglects opening variations in the vertical cross-section of wall panels. Lee (2010) proposed the new formulae to simultaneously account for opening variations in both horizontal and vertical cross section of wall panels. In order to solve this, the existing parameter in Equation (5) is now referred to as x_x and a new parameter is proposed as x_y. The x_y is proposed as follows: $x_y = \left(\frac{A_{oy}}{A_y} + \frac{\eta_y}{H}\right)$ in which $\eta_y = \left(\frac{H}{2} - \tilde{\eta}_y\right)$ with $\tilde{\eta}_y$ being the distance from the left vertical edge to the centre of gravity of the cross-section of the panel with openings, or $\tilde{\eta}_y = \frac{\frac{1}{2}t_w H^2 - t_w H_0 \eta_0}{H t_w - H_0 t_w}$; and $A_{0y} = H_{0y} t_w$. The proposed new design equation for ultimate strength is as follow:

$$N_{uo} = \{k_1 - k_2(x_x + \lambda x_y)\} N_u \tag{15}$$

Where N_u was calculated as in Eq (6). The constant k_1 and k_2 are obtained from experimental results, and they are 1.19 and 1.30 for one-way action, and 0.83 and 0.2 for two-way action respectively.

Guan et al. (2010)

Guan et al. (2010) identified that increasing both the height and length of an opening has the most significant effect on the load carrying capacity. The analysis was conducted using a nonlinear finite element method. Guan et al. (2010) proposed new opening parameter, x_{xy}, which cover the both function of x_x and x_y in the effect of both the opening length and height. The proposed design formula for ultimate load is

$$N_{uo} = (k_1 - k_2 x_{xy}) N_u \tag{16}$$

where

$$x_{xy} = \frac{x_x + \lambda x_y}{1 + \lambda} \tag{17}$$

and

$$x_y = \frac{A_{oy}}{A_y} + \frac{d_y}{H} \tag{18}$$

in which $\frac{A_{oy}}{A_y}$ accounts for the opening size in the vertical plane, $\frac{d_y}{H}$ corresponds to the opening location in the vertical direction, and dy is the distance between centres of gravity of the panel with and without an opening respectively, in the vertical plane. In Eq. (17), λ (0≤λ≤1) is the weighting ratio indicating the percentage of x_x in relation to x_y. However, λ together with the constants k_1 and k_2 can be determined by a standard regression analysis through a calibration process.

SHEAR WALL

Some buildings are subjected to both vertical and horizontal loads, i.e., buildings subjected to wind and earthquake loads. Shear walls are used in this type of buildings, mostly located at the seismic vulnerability areas or high-rise building. Besides, shear walls are also generally located at the side of buildings or arranged at the core that houses the stairs and lifts. Shear walls have become a very important element in a building and many researchers have studied on the behaviour of reinforced concrete shear wall panels with opening with different aspect ratio, opening configuration and loading condition.

Opening

Openings created on a shear wall disturbs the stress flow in the wall panel. Investigation on shear wall panels with opening is scarce due to the fact that shear wall panels are mostly related to high rise buildings (Yanez et al., 1992; Sharmin et al., 2012; Mosoarca, 2013; Aarthi and Senthil, 2015; Raflik et al., 2012; and Ashok et al., 2016).

Yanez et al. (1992) studied on the reinforced concrete walls with square opening of different sizes and arrangement under seismic loading. It was concluded that the stiffness of concrete wall panels with opening is dependent of the size of opening but not on the location of opening. The results show that the stiffness of the concrete wall with opening smaller than 10% of the wall area was the same as the stiffness of the concrete wall without opening. However, other investigations (Sharmin et al., 2012; Mosoarca, 2013; Aarthi and Senthil, 2015; Raflik et al., 2012) contradicted with Yanez at al. (1992) in regard to the effect of location of opening on the load carrying capacity of shear walls.

Mazen (2013) studied on the size of opening of shear walls by nonlinear finite element analysis using solid 65 elements. It was found that the small opening has minor effect on the load carrying capacity, cracking pattern, maximum drift, and flexural stresses along the base level of shear wall, which is in agreement with the experimental results of Yanez et al. (1992). However, Mazen (2013) concluded that a 1 m × 1 m opening size is considered as a small opening in life-size building. The load carrying capacity is reduced in the shear wall with the increase in opening size larger than this. For the 1 m × 3 m opening size, the load carrying capacity decreased by 70% compared with the solid shear walls.

The effects of opening in concrete wall panels in a six-story shear wall were reported by Sharmin et al. (2012). With the application of finite element method, the results reveal that the stiffness of a reinforced concrete shear wall is affected by the size and location of the opening. It was concluded that the larger the opening, the more the displacements registered, and this trend increases with increase in storey level. At lower storey level, displacements are almost equal but as soon as story level increases, displacements increase for the concrete shear wall with opening for door. At topmost storey level, displacement of reinforced concrete shear wall with opening in the middle is slightly less than those with opening placed at left side or right-side pf the shear wall.

Raflik et al. (2012) revealed that the shear strength and crack pattern was different depending on the opening location for lower three-storey wall panels of a typical six-storey reinforced concrete building. Cyclic reversed horizontal load and vertical axial load were used in the experiment and software analysis. The vertical axial loads were applied to the column to represent the loadings from upper three storeys. The results show differences between positive and negative directions due to the opening location and the damaged condition. The existence of an eccentric opening affected the behaviour of the concrete strut and caused the sliding failure of the wall panel. For wall panel without opening, the wall panel cracked at the boundary between the wall and the beam. However, for wall panel with opening, the flexural crack occurred in the column in tension at first storey and cracks were observed at the upper corner of the opening. For the opening located at the centre of the wall panel, there were only a few shear cracks over two storeys because of the difficulty in composing the compressive strut over two storeys as compared to the wall with opening at the side. The comparison between experimental and software analysis results showed a respectable agreement for the maximum peak load.

Mosoarca (2013) proved that the location of the opening affects the reinforcement ratio in the concrete shear wall with opening. With the same amount of reinforcement ratio, the concrete wall with staggered openings developed a ductile failure, whereas the concrete wall with regular openings developed a brittle failure. This investigation concluded that shear walls with staggered openings are stiffer, hence requiring less reinforcement.

Aarthi and Senthil (2015) studied on reinforced concrete shear wall with staggered openings by finite element analysis. The results reveal that the reinforced concrete shear wall with staggered openings performed better in time period and stress distribution within the wall compared with shear wall with vertical openings. The stress in shear wall around the staggered openings has much lesser intensity compared with the stress pattern around the shear wall with vertical openings. Hence, staggered openings in shear wall is highly advantageous in resisting the lateral loading as compared to shear wall with vertical openings.

Mashiko et al. (2001) revealed propagating from the corner of the openings that the experimental results for concrete wall panel with circular opening can support more loads compared to concrete wall panel with square opening, with or without reinforcement. Besides, the cracking strength was also higher for concrete wall panel with circular opening. The circular-opening wall panel without reinforcement and square-opening wall panel without reinforcement started to crack at load of 254 kN and 146 kN, respectively. There was an increase in ultimate strength of wall panels in around 4% to 19% when sleeve and plate reinforcements were added.

Mihai and Valeriu (2010) tested a precast reinforced concrete wall panel with a large door and window cut-out opening made at the ground floor of a five-storey building in both the gravity and seismic capacity. The opening size is 40% of the solid wall panel in the experimental studies. The results show that the concrete crushed at the both corner of the door opening and the window opening. The cracks were observed visibly extending up from the top corner of the opening to the top corner of the concrete wall panel. The large diagonal cracks also extended between the piers of two openings to the bottom end of the wall panel. These large diagonal cracks led to the loss of the bearing capacity and certainly led to the failure of the wall panel. The load bearing capacity was decreased around 28% compared with the wall panel without opening of Demeter et al. (2010). However, the drift ratio was increased by 50% because the opening made the wall panel more ductile in behaviour.

Ashok et al. (2016) compared the seismic performance of a 15-storey building with shear walls consisting rectangular and square openings by finite element modelling. The results show that the frames with shear wall are only affected by the location of the opening when the opening area or size is more than 15% of the solid wall panel. The height and width of openings do affect the load carrying capacity of the shear wall as well.

Reinforcement

Christopher et al. (1998) revealed that shear plays an important role in the behaviour of wall panels with opening compared with wall panels without opening when a horizontal force is applied. Different types of reinforcement which are boundary longitudinal reinforcement, transverse reinforcement and horizontal shear reinforcement were used to evaluate the shear requirement for slender wall with openings. The results of the experiment show that the wall which was designed by using the combination of displacement based and strut and tie approach gave a stable and ductile behaviour and the simplified code equation is not recommended for the design of concrete walls with opening. Finite element modelling has indicated that an increase in either horizontal or vertical web steel alone does not significantly improve the behaviour of walls with opening. Also, the strut and tie models developed for the test specimens do not directly indicate a need for vertical web reinforcement. However, as noted previously in the preceding section, vertical steel that is provided at the same spacing as the horizontal steel based on ease of construction as opposed to diagonal reinforcement that is effective in reducing crack widths. However, the experimental results prove that this arrangement is effective.

Mashiko et al. (2001) studied on the behaviour of steel plate reinforced concrete (SC) panels with an opening which is loaded in cyclic in-plane shear. The researchers tested six SC panels having an opening and one SC panel having no opening in an experimental program. Two different shapes of opening which are circular opening and square opening were created in SC panels. There were variations of reinforcement used which are sleeve reinforcement and plate reinforcement. The results reveal that the responses of SC panels under loading were ductile and less cracking pattern shown on the surface of the panels. The results also reveal that plate reinforcement is more effective than sleeve reinforcement and it can support more load with less deformation. The strain near the opening of the panel increases linearly as the load increases, but the value is smaller when the panels are provided with steel plate reinforcement.

Slenderness ratio

Muthukumar and Manoj (2014) studied on the dynamic behaviour of both squat shear wall and slender shear wall with various opening location, size and number of openings. Shear walls with aspect ratio between 1 and 3 are generally considered to be of squat type and shear walls with aspect ratio greater than 3 are considered to be of slender type. The results revealed that it is better to provide a large number of small openings instead of least number of large openings. The displacement has been found to be higher in the case of slender shear walls than the squat shear walls.

FUTURE PERSPECTIVE OF CONCRETE WALL PANELS WITH OPENING

Based on this literature survey, there are some important points that can be concluded. For wall panels subjected to axial load, most of the researches focused on the concrete wall panels with opening by designed, but not in the wall panels with cut out opening. Therefore, further investigation on concrete wall panels with cut out openings is necessary. The shape of the opening on the concrete wall panel also becomes an important factor due to the services purpose and aesthetic value. Hence, further investigation is also needed to study the effect of the shape of the opening on the behaviour of concrete wall panels. Most of the researches are not coordinated, so the results are not reflected in the design guidelines or codes of practice. Therefore, there is also a need to coordinate these research findings to facilitate the design provisions in codes of practice.

CONCLUSION

In this review of literatures on reinforced concrete wall panels with opening, it can be concluded that the boundary condition, aspect ratio and slenderness ratio are the main factors affecting the load carrying capacity of reinforced concrete wall panels with opening. The decrease in load carrying capacity of the wall panels due to opening depends on its size and location. Support condition affects the cracking pattern and therefore the load carrying capacity. The failure mode of reinforced concrete wall panels depends on the strength of the wall panels. For wall panels subjected to lateral load, e.g., seismic load, the previous researches focused on the arrangement of the openings because shear walls are normally used for high rise buildings. To date, there is still no design guideline or code of practice describing the size and shape of opening for the cut-out opening in reinforced concrete wall panels.

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DIGITIZING FLOOD PROCESSES FOR SARAWAK RIVER OPERATIONS

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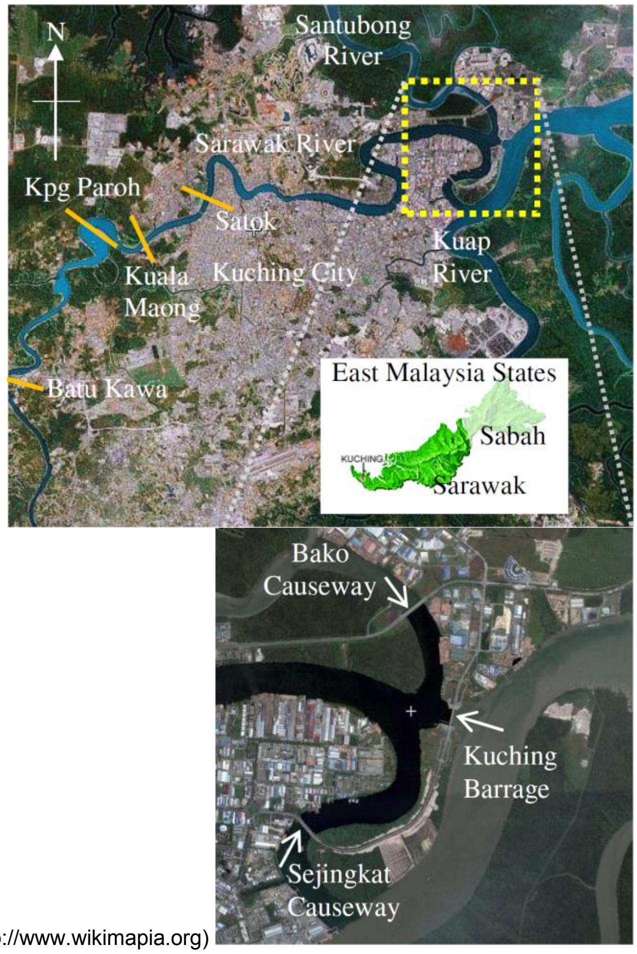
Abstract
Kuching city of the Sarawak State, Malaysia is located on a flat alluvial plain 30 km away from the sea. The low-lying city is vulnerable to high tides, where the city had experienced recorded spring tides as high as 3.5 m MSL. In 1998, a barrage is established near estuary to protect the city. The structure is the property of the State Government under the care of Sarawak Rivers Board, but the operation is managed by a private contractor. The operation team extensively regulates the hydrological data of the river system with its telemetry system. However, the operator is lacking a flood forecasting system to predict high flow from upstream. This paper reports a computer modelling effort to guide the barrage operations in flood warning. We present a way of warning based on computerized flood mappings. The behaviours of the river in terms of levels, rates of flow, and other parameters are simulated. The riverside communities are to be alerted at lower river water levels between 0.8 and 1.0 m MSL as compared to the currently used 1.37 m MSL for flood warning with reference to Batu Kawa.

BACKGROUND

A tidal barrage was established since 1998 across Sarawak River, the major river flowing through the capital city of Kuching in the State of Sarawak, Malaysia (see Figure 1). A barrage is an artificial obstruction at the mouth of a tidal watercourse. Kuching Bay is subjected to spring tides as high as 3 - 3.5 m MSL (A. Memon and M. Murtedza, 1999). Annual rainfall within the catchment is typically around 4000 mm. Therefore, the city is at risk to fluvial and tidal flooding events. The Sarawak River flow was modified from a naturally tidal regime to regulated gates system constructed just downstream of Kuching city under the Sarawak River Regulation Scheme (KTA Consulting Engineers, 1994). Kuching Barrage was the only outlet of the catchment while another two river courses to the sea were blocked. It was designed to reduce the incidence of floods in the Kuching city area, principally by curtailing the effects of tides upstream of the barrage (J.J. Sharpand and Y.H. Lim, 2000).

Operations of the Kuching Barrage to control river water levels involve frequent opening and closing of five (5) mechanical radial gates plus a ship lock. During periods of low river flow, all the gates must be closed to completely stop the flow and maintain high water levels. During floods, the gates may all be opened depending on tide levels. For intermediate conditions, constant regulation is required to prevent the entry of salt water and to keep the water level upstream at the required level. For daily operations, Kuching Barrage is carrying out flushing operation during low tide and alternate weekdays flooding-in operation during high tide (P.L. Law et al., 2007). Flooding-in operation is to specially provide the required water levels for shipyard maintenance activities located at the immediate upstream of the barrage.

Previously, only seven (7) telemetry stations were installed. The extreme floods in January 2003 and February 2004, however, have prompted a more pro-active flood warning system along the Sarawak River since December 2005. Twenty-four (24) telemetry stations utilizing Australian expertise and technology are now providing early warning for Kuching residents. Each of the station is equipped with siren to warn people living within one km radius. The barrage is entrusted as the centre of the whole operation where the operation team is responsible to monitor the system round the clock. The current flooding alert levels utilized by the barrage operator along the river based on water level / frequency relationships (see Figure 2).



(Sources : http://www.wikimapia.org)

Figure 1. Imageries of Kuching City, its river system and Kuching Barrage

Figure 2. Monitoring of river water level trends by Kuching Barrage operator

MOTIVATION

Reliable flood forecasting and early detection of flood conditions are critical components of an effective local flood warning program. However, the barrage operator is lacking a flood forecast system. This includes predictions of future flows into the area impounded by the barrage so that the gates in the structure are operated effectively (Y.Y. Oh et al., 2019; H.H. Iqbal et al., 2017; H.C.C. Auricht et al., 2018). The barrage has the capability of blocking the variation of tidal surges. Nevertheless, the land drainage outfalls and impoundment remain a concern because flows are designed to discharge only during low tides.

The river corridor is taken as a reservoir designed to accommodate 100-year flood flow. Such measure would have forced the floodwater behaves in certain ways. Kuching Barrage operation contractor has been operating the barrage for over 10 years. The assumption made here is that hydrological data collected from telemetry stations would provide insights on the flood conditions. As reported by M.G. del Tánago et al., 2016, representing the effective runoff production and processes at a catchment scale are key to improved flood simulation. Computer modelling has a long-standing tradition in this sense and one of the most important outputs is indicative flood maps. Flood mapping has been used for flood assessment and visualization but rarely been explored as a form of flood warning tool.

Therefore, as the main objective of this modelling effort, we argue that a computer river model would be capable to provide a way of flood warning system based on flood mapping. By knowing the river water level at a reference point and correlating this level to the flood extent elsewhere, impact indicators can be developed to guide early warning as when to sound the alarm along the city stretches, etc.

CORRELATING FLOOD EXTENT

A Wallingford Software model, InfoWorks River Simulation (RS) is utilized for modelling Sarawak River. InfoWorks RS involves tight coupling of GIS functionality and hydrodynamic flow simulation. Such model relies profoundly on the accuracy of topographical data. A ground model is derived from the contour map, the river corridor survey data and spot height in the floodplain. As for the contour maps, T835 series are restricted topographic map of Sarawak at 1:25,000 scale and sufficient for the river catchment. The data comes with 20 m interval contour lines in Borneo RSO (Timbalai) projection. The bed levels along the river are extracted from a river corridor survey exercise done in year 2000. Both sets of data are combined in ESRI ArcView GIS to form a base Triangulation Irregular Network (TIN) ground model. In the absence of more advanced earth surface observation datasets, the available topographical data was the best at the moment. These were adequate for InfoWorks RS hydrodynamic modelling as previously proven by reports in modelling Sarawak River systems (D. Kim et al., 2018).

InfoWorks RS model has to be subjected to sensitivity analysis using field data and then validated over a time period using different data sets. The model calibration is carried out on running the January 2004 and January 2009 flood, both 100-year events. The results are shown in the subsequent Figure 3. The model has been validated in existing conditions to at least 80% of confidence. The analysis indicated that for Sungai Sarawak, a Manning's n of 0.05 and 0.12 were appropriate for the river channel and floodplain respectively. Once the calibration of the model has been successfully completed, the modelling framework is then suitable for use in making predictions of future conditions.

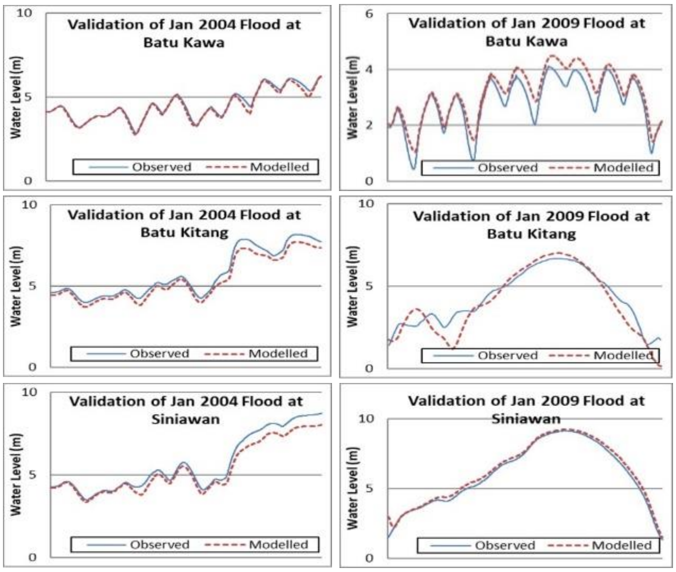


Figure 3. Model calibration of January 2004 and January 2009 flood events at Batu Kawa, Batu Kitang and Siniawan

The first ground taken into consideration is that the barrage operator has built up a familiarity of barrage gate operations accustomed to upstream and downstream conditions through their over 10 years of operation modes. This modelling effort recognizes this fact and intends to maintain the usual gate operations system. Representative of 1 in 100-year floods of January 2004 and January 2009 storm surges are run through the computer model. Both events coincided with the highest spring tides over this region, known as the King Tides downstream. The general scheme of building the model is displayed in Figure 4.

The upstream-ends at Buan Bidi and Kpg Git are treated as Flow-Time Boundaries. The Kuching Barrage was modelled as radial gates. The gauges at the barrage had recorded tidal fluctuations that were fed to Muara Tebas end (downstream of Kuching Barrage) as tidal Stage-Time Boundaries.

Upstream Barrage at first glance may seem to be an ideal reference. Simulation results of running the January 2004 flood event are shown in Figure 4 at the levels of -1 m, 1 m and 3 m MSL at Upstream Barrage. However, barrage operations maintain river level between -1 to 3 m MSL as normal day-to-day routine and no overbank flooding observed. It indicates that the upper catchments flow has a dominant influence in Sarawak River flooding.

The general ground level in Kuching city is at about 5 m MSL. The maximum level that can be accommodated at Kuching Barrage is 4.6 m MSL. Kuching Barrage is confronted by spring tides every fortnightly, sometimes up to 3.5 m MSL highest in Southeast Asia region. During normal days, the tides are between -0.5 to 0.5 m MSL high. Daily operation of Kuching Barrage maintains the Upstream Barrage at high water level between -1 – 3 m MSL is to equate the water forces of downstream sea water to lessen the pressure on the narrow strip of barrage structures.

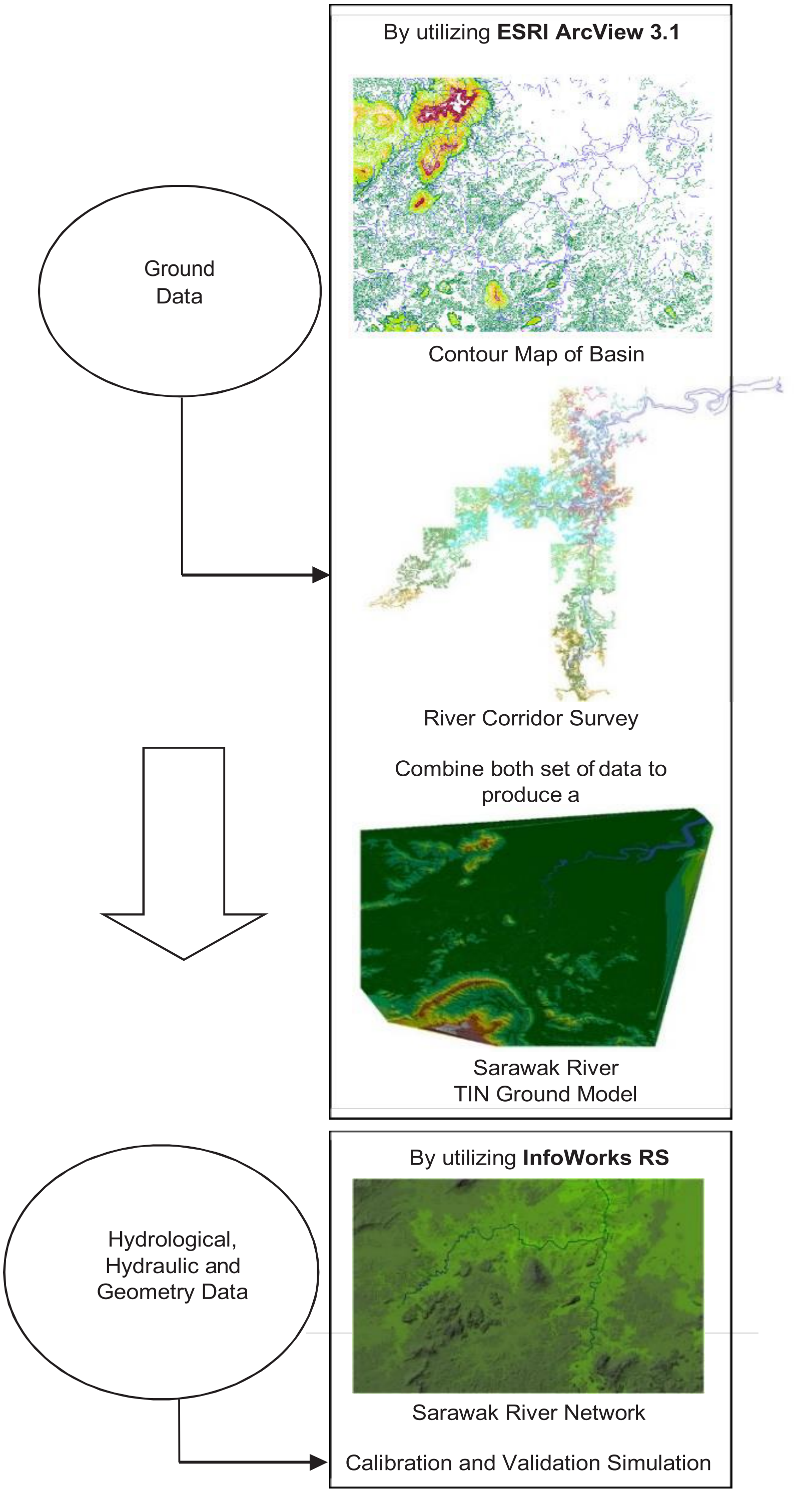
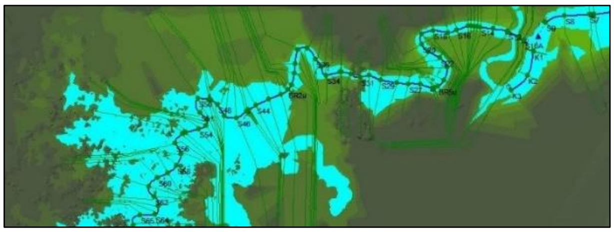
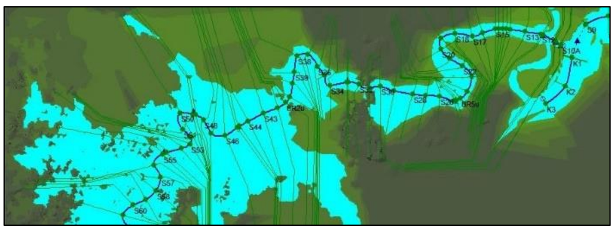


Figure 4. Hydrodynamic model development

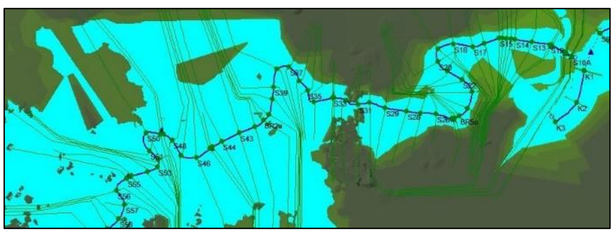
Upstream Barrage at first glance may seem to be an ideal reference. Simulation results of running the January 2004 flood event are shown in Figure 5 at the levels of - 1 m, 1 m and 3 m MSL at Upstream Barrage. However, barrage operations maintain river level between -1 to 3 m MSL as normal day-to-day routine and no overbank flooding observed. It indicates that the upper catchments flow has a dominant influence in Sarawak River flooding.



a) When river water level at Upstream Barrage is -1 m



b) When river water level at Upstream Barrage is 1 m



c) When river water level at Upstream Barrage is 3 m

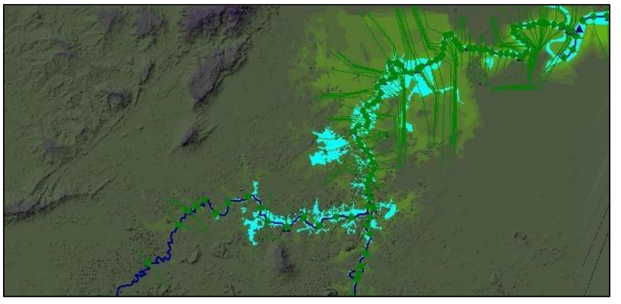
Figure 5. Simulation of January 2004 flood event at Lower Sarawak River

The general ground level in Kuching city is at about 5 m MSL. The maximum level that can be accommodated at Kuching Barrage is 4.6 m MSL. Kuching Barrage is confronted by spring tides every fortnightly, sometimes up to 3.5 m MSL highest in Southeast Asia region. During normal days, the tides are between -0.5 to 0.5 m MSL high. Daily operation of Kuching Barrage maintains the Upstream Barrage at high water level between -1 – 3 m MSL is to equate the water forces of downstream sea water to lessen the pressure on the narrow strip of barrage structures.

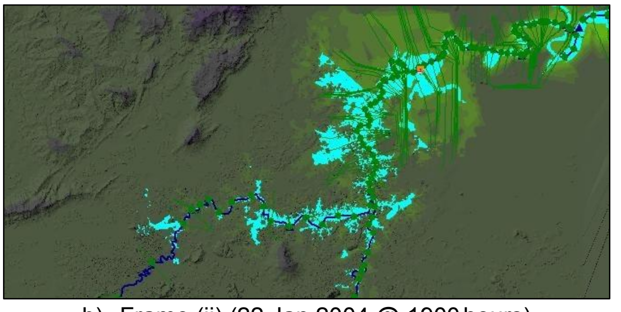
TRANSITION REGION

In the lower part of Sarawak River, water levels are controlled by barrage operations. In river upstream of confluence, water levels are controlled by overland flow. The interaction zone between these two is then explored, taking Batu Kawa as reference.

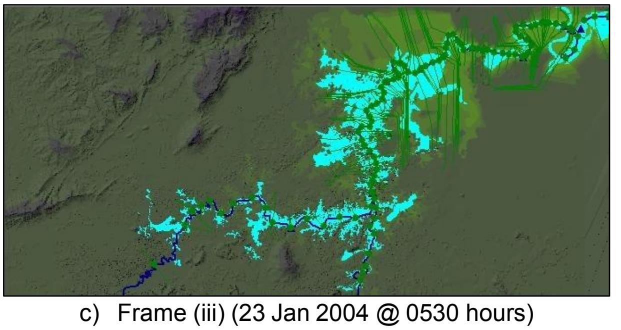
Simulation results of running the January 2004 flood event are depicted in Figure 6. It seems that flooding in upstream areas occurred even as water levels at Batu Kitang and Batu Kawa below bankfull levels (see Frame (i) to (iii)). When the water levels approaching bankfull, the flooding in downstream areas accelerating (see Frame to (vi)). Normal water level in Batu Kitang is between -0.55 – 0.35 m MSL, and Batu Kawa in -0.40 – 0.65 m MSL. From Frame (i) to Frame (ii), flooding is getting worse when water levels in Batu Kitang and Batu Kawa are low. Therefore, flood warning is to be alerted before reaching the bankfull levels as reference to Batu Kawa. This finding is in contra with the alert levels currently practiced by barrage operator.



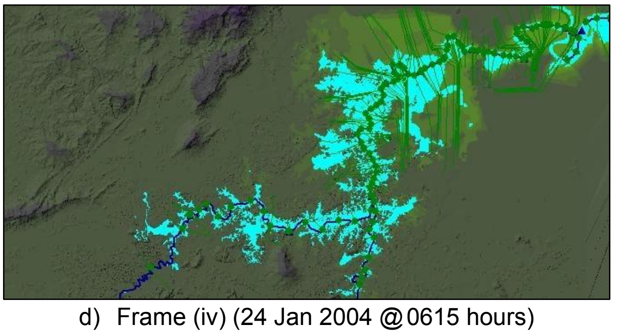
a) Frame (i) (22 Jan 2004 @ 1830 hours)



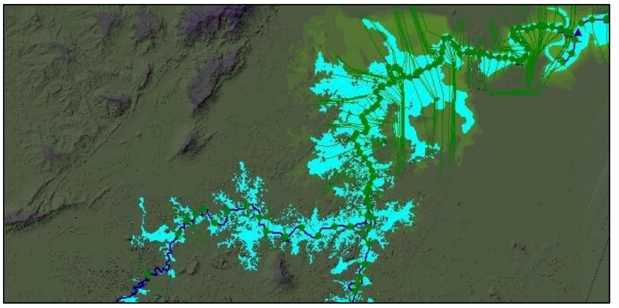
b) Frame (ii) (22 Jan 2004 @ 1900 hours)



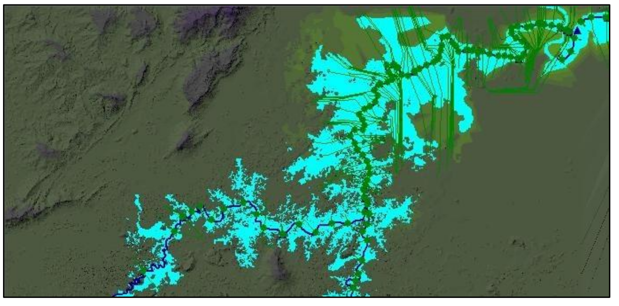
c) Frame (iii) (23 Jan 2004 @ 0530 hours)



d) Frame (iv) (24 Jan 2004 @ 0615 hours)



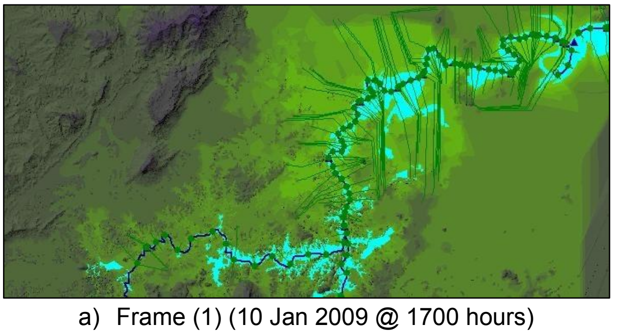
e) Frame (v) (24 Jan 2004 @ 1100 hours)



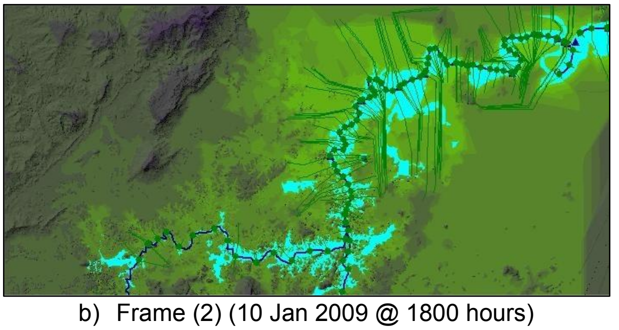
f) Frame (vi) (24 Jan 2004 @2100 hours)

Figure 6. Simulation of January 2004 flood event at different inundation frames

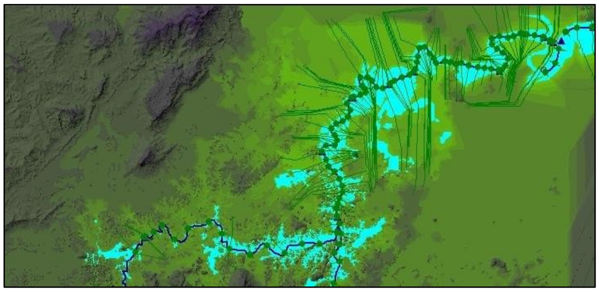
Another set of simulation on January 2009 flood event is featured in Figure 7. The extent of this flood is comparatively smaller than the previous flood. As the floodwater reaching its peak, the downstream tides were subsiding allowing water to be discharged from the river system. The simulation has provided similar understanding of the flooding from Frame (1) to (3) as of Frame (i) to (iii). It put a confirmation to the flooding behaviours of Sarawak River.



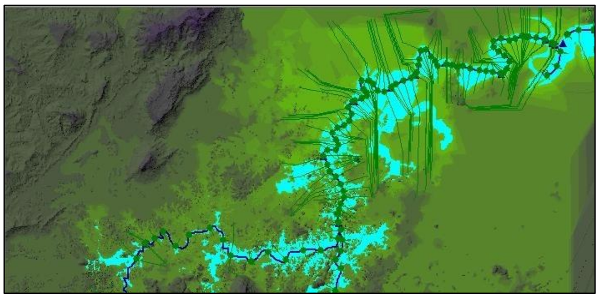
a) Frame (1) (10 Jan 2009 @ 1700 hours)



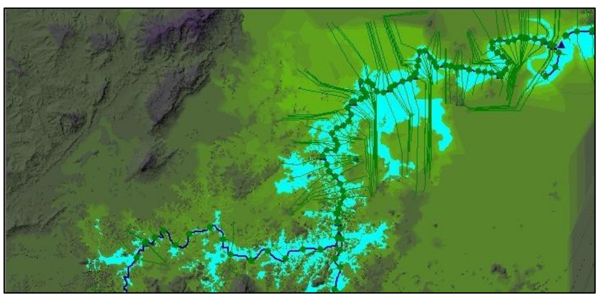
b) Frame (2) (10 Jan 2009 @ 1800 hours)



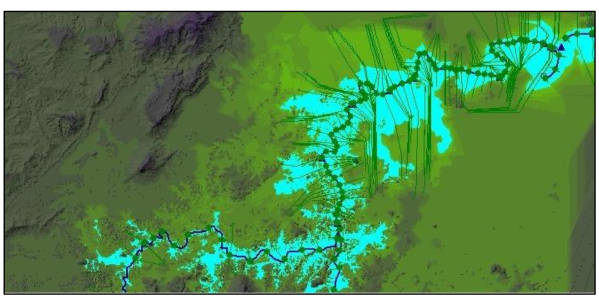
c) Frame (3) (11 Jan 2009 @ 0400 hours)



d) Frame (4) (11 Jan 2009 @ 0600 hours)



e) Frame (5) (11 Jan 2009 @ 1100 hours)

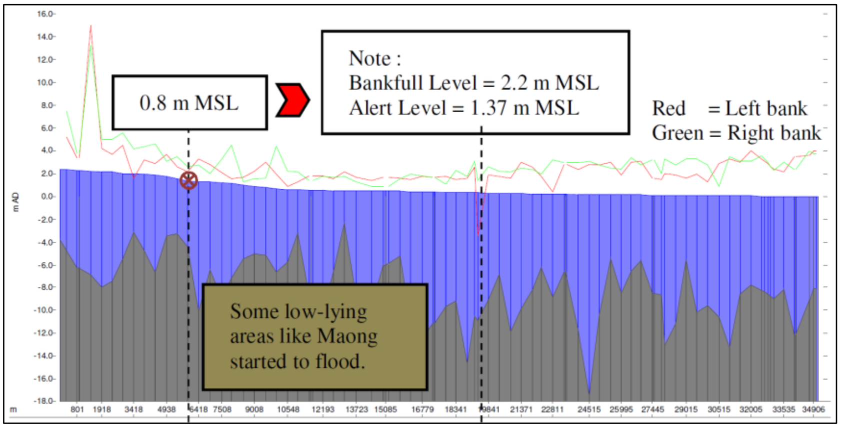


f) Frame (6) (11 Jan 2009 @ 2200 hours)

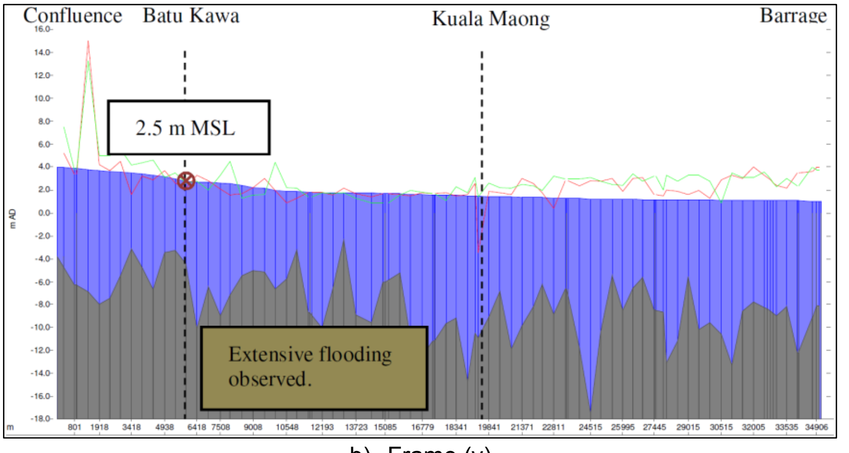
Figure 7. Simulation of January 2009 flood event at different inundation frames

LONG SECTION PROFILES

The long profile of a river shows a steep gradient at the source, gradually becoming lower and less steep. The long sections of Sarawak River are presented in Figure 8 as an attempt to correlate the river flood levels. Overlaying longitudinal profiles from different time frames are used to compare water level changes in in-stream and over- bank relationships as determinant of flooding patterns to warn. We suggest the alert level to be trimmed down to between 0.8 and 1 m MSL as reference to Batu Kawa.



a) Frame (iii)



b) Frame (v)

Figure 8. Long section profiles of Sarawak River for January 2004 flood event

CONCLUSION

A Sarawak River basin-wide model is developed using the InfoWorks software. InfoWorks RS, namely a hydrodynamic model for river system, incorporating ground models that extend across the catchment, and a hydraulic simulation, representing the impact of wetting events on the river system. The behaviour of the river in terms of levels, rates of flow, and other parameters are simulated. Flood mapping of extreme events, like the 100-year return period January 2004 and January 2009 floods, have been computed to give insights to a flood warning system in Kuching. The modelling results have suggested that in the absent of a flood flow forecasting, the riverside communities are to be alerted at lower river water levels between 0.8 and 1.0 m MSL as compared to the currently used 1.37 m MSL for flood warning with reference to Batu Kawa.

ACKNOWLEDGEMENT

This project was funded by DPP Grant Scheme F02/DPP64/1445/2016/11 rendered by the Universiti Malaysia Sarawak.

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CONDITION ASSESSMENT OF HERITAGE BUILDING: A CASE STUDY IN KOTA BHARU KELANTAN, MALAYSIA

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Abstract
Building assessment is one of the key components of building maintenance. Usually, heritage buildings are expected to serve their functional requirements throughout their operation and maintenance. The primary purpose of performing a building assessment is to evaluate the building's condition. Without assessment, it is difficult to determine a built asset's current condition, so failure to inspect can contribute to the asset's future failure. This study proposes a new assessment method, derived from the current rating systems, for assessing the building's condition and rating the seriousness of each defect identified. To achieve this, the study used a two-phase, sequential exploratory mixed methods research that was initiated with a qualitative phase (Delphi technique) and followed by a quantitative phase (Analytic Hierarchy Process technique). This condition instrument assesses three main criteria: the building structure (e.g. foundation, column, beam, truss and stair); building fabric (e.g. ceiling, floor, internal wall, external wall, roof, door, window and arch); and building service (e.g. electrical, air-condition, fire protection, sanitary and plumbing). These three assessment criteria are then multiplied to find the building's score. Instead of a longhand description of a building's defects, this assess requires concise explanations about the defects identified, thus saving on-site time during a building inspection. The full score is used to give the building an overall rating: Very Good, Good, Fair, Poor or Very Poor. The application of the proposed model is illustrated by a case study. Our overall findings reflect the reliability of the model.

Keywords: *Evaluate; building condition assessment; heritage building.*

INTRODUCTION

Condition assessments are an important aspect of effective maintenance planning. The incorporation of condition assessments as part of maintenance processes ensures that there is a structured, objective process for identifying the demand for condition-based maintenance works to meet strategic and operational priorities (Olanrewaju et al., 2015). Such works should form part of any comprehensive program of maintenance in conjunction with preventative, statutory and reactive (unplanned) maintenance work, over the immediate, medium, and long term (Kelly, 2006).

The assessment of building condition is a technical inspection conducted by a competent assessor to evaluate the physical state of building elements and services and to determine the maintenance needs of the facility (Dejaco et al., 2017). A professional building surveyor can examine building conditions comprehensively, and the generated detailed report can protect the interests of new home buyers, especially with respect to technical aspects such as workmanship and material. Moreover, property developers can consider this report in managing building defects (Ani et al., 2014).

Condition assessment generally comprises: physical inspection of a building to assess the actual condition of the building and its individual elements and services; identification of maintenance works required to bring the condition of the building and its services up to, or maintain it at, the specified condition standard; ranking of maintenance works in order of priority; and determination by the assessor of actions to mitigate any immediate risk until remedial works (or other actions) can be taken to address problems (Hill & Bowen, 1997; R. Talib et al., 2014; Akadiri et al., 2012). Thus, the assessment of building condition is important in evaluating building quality. To indicate building condition, assessed buildings must be rated. A rating is a set of categorization scales designed to expound on the quantitative or qualitative attributes of an object (Ani et al., 2014).

The purpose of conducting a building assessment is to evaluate the building's condition. The assessment is a key means of identifying a building's defects. Defects usually display their symptoms before getting worse and causing building failure. It is therefore crucial for building inspections to be performed many times in an asset's life cycle (Alwaer & Clements-Croome, 2010). This is also supported by the philosophy of Dasar Pengurusan Aset Kerajaan (DPAK), the Malaysian Government Asset Management Policy and Total Asset Management (TAM) Manual (Ifran Che-Ani et al., 2011).

LITERATURE REVIEW

In Malaysia, the thrust of heritage conservation of many of the nation's landmark buildings were undertaken by the public sector (Tan et al., 2016). However, the funds are limited for preventive measures to maintain all relevant buildings (A.F. Mohd-Isa et al., 2011) as well as the lack of public awareness of internationally recognized guidelines such as the Burra Charter (R. Abdul Rashid and A.G. Ahmad, 2011). The public is now starting to be aware of the value of heritage buildings as a limited resource that forms part of the urban scenery as well as a tourism landmark and attraction (N.F. Nik-Azhari and E. Mohamed, 2012). This is where the private owners come in, but they require the right guidance and approach to their property conservation independent of the oversight of the National Heritage Department (N.F. Nik-Azhari and E. Mohamed, 2012). The introduction of the National Heritage Act 2005 is a step in the right direction though it remains to be seen if building owners perceive it to be so. However, most conservation management lacks the skill to interrogate the building's history due to the lack of archival materials. Most of the heritage conservation literature focus on the organizational management, programmatic and awareness approach but the technical application and building survey approach is lacking (S. Macdonald, 2011).

Heritage building and monuments are the most important part of the cultural heritage and human civilization and it is the human imperative to protect those structures for the future generations. Besides their artistic value those structures are open to the public and to the large assemble of people (R. Abdul Rashid and A.G. Ahmad, 2011). The number of heritage buildings in many state capitals that serve as sources of livelihood and prestige to the owners and nation (Binda L. and Saisi A., 2005). While some of the heritage buildings are declared 'national heritage', a number are owned by private individuals or associations which are yet to be registered but contribute no less to the heritage and tourism landscape (C.L. Cartier, 1996). These notable building typologies include the museum building of Southeast Asia and were used for various purposes including education, residences, offices, community halls, businesses, and warehouses (W.H Wan Ismail and S. Shamsuddin, 2005). However, many of these heritage building has been left neglected, leading to scores of dilapidations and abandonment. This represents an economic loss to various stakeholders (S.N. Harun, 2011).

During their long life, heritage building has experienced many actions occurred over long periods of time; endured long term deteriorating effects and earthquake loads. Since heritage importance, cultural value and exposure of aggressive environmental loads there is no fixed criterion for evaluating safety of heritage structures. Study on the defects of a heritage building necessitates an interdisciplinary team of specialists and requires specific techniques (Vatan M., 2011). Condition assessment of the buildings against natural disasters and human induced hazards is very important. In the context of cultural heritage preservation, it is very important to assess the potential seismic risk in the hazard prone areas for hazard mitigation and limiting the disaster impact. It is very difficult to make precise quantitative risk assessment for heritage building structures. There are technical codes and guidelines for new buildings however the approaches for new constructions are not applicable to the heritage building structures (Binda L. and Saisi A., 2005).

Making precise assessment of heritage buildings is a very difficult task. Condition assessment based on qualitative and quantitative data is necessary before making any intervention decision. Mostly researcher get the qualitative data from a visual inspection of building structural damages, decays and deteriorations, research on archive material and literature. Obtaining the quantitative data requires rather complicated methods which necessitate specialists and are time and money consuming. Consequently, such techniques are mainly used in the last step of the diagnosis and condition assessment which can be performed only on a limited number of buildings (Roca, 2007). According to this fact, it is very important to use simpler methods to evaluate the potential risk of many buildings as a first step of cultural heritage preservation.

The objective of this study is to identify the criteria and attributes of defects as well as appraise the causes of these defects. The general significance of the study is that, by understanding the causes of defects through the life cycle of the building, a general guideline for heritage buildings maintenance can be framed for these buildings to be usable instead of being demolished for redevelopment. This is significant to provide reference to neighbouring owners and those responsible for the conservation of heritage buildings with similar conditions to identify and prioritize critical defects in relation to the building life span to determine its condition. A part of the presented data is based on PhD study of the researcher. Condition survey of Museum in Kota Bharu is given as a case study.

Condition Survey of Heritage Building in Kota Bharu, Kelantan

Kota Bharu city is situated in the north eastern part of Peninsular Malaysia and occupies a strategic location at the mouth of the Kelantan River. It has become a Royal City and an administrative centre for the state and has become an important place for trade and business as well as a cultural melting pot (Nasir, 2012). Early Malay towns usually evolved around the sultan's palace and the mosque because these were the two centres of cultural and religious activities. Traditionally, there were no fixed boundaries in a Malay town, although the distance up to which the prayer calls of the town mosque could be heard sometimes determined the town's limits (Fee, 1998). It is recorded that Sultan Muhammad II founded the city of Kota Bharu on the banks of the Kelantan River in 1844. Since then, Kota Bharu has remained the administrative centre for Kelantan. There are royal palaces, museums, mosques and historical buildings at the heart of the city centre. Various landmarks and commercial buildings also give a strong image to the city centre (Nasir, 2012).

Malaysia has always been open to ideas and influences from both East and West. It is thus blessed with an architectural heritage of great diversity, artistry, and sophistication, showing many cross-cultural influences, and ranging from the simplest built shelters to mega projects (Fee, 1998). Although very few timber palaces over a hundred years old have survived in Malaysia, in feudal Malay society the palaces of the sultans were of paramount importance, not only as places of residence but also as centres of administration, learning and culture. In Kelantan, especially in the heritage zone of Kota Bharu, there still remain the vernacular traditional structures of Kelantan Malay architecture, namely palaces and mosques (Fee, 1998; W.W. Lim et al., 2014).

However, due to the inability of building owners to extensively repair a building while it is in use, as this would disrupt business operations, many such heritage buildings' ground floor would be functional while the first floor would be left abandoned once it was deemed unsafe to use. The problem faced by many owners was a lack of technical based studies conducted to understand the types of physical defects present in such buildings and to conclude with appropriate solutions.

RESEARCH METHODOLOGY AND METHOD

This study used a two-phase, sequential exploratory mixed methods research that was initiated with a qualitative phase (Delphi technique) and followed by a quantitative phase (Analytic Hierarchy Process technique). Analytic Hierarchy Process (AHP), provides a framework to structure individual and group subjective judgments can refer from paper S. N. F. Zuraidi et al., 2018a. In this study, the Delphi method has been conducted prior to AHP so that not only can the objectives to be considered in analysis be determined, but the opinions of all decision makers can also be incorporated in problem formulation. This section outlines the steps of Delphi Technique and Analytic Hierarchy Process, and the next section, by means of experimentation, illustrates can be used to elicit objectives from decision makers, obtain their weighting of those objectives, and then derive priorities among them (S. N. F. Zuraidi et al., 2018b).

Data Collection

This work is based on the methodological structure which is organized as follows: project aims, method, data collection techniques, and data analysis techniques by S. N. F. Zuraidi et al., 2018c. With regard to the aims, the research method for this paper includes a building survey approach: (a) taking photographs of all building elevations, (b) dilapidation survey by visual observation, (c) determine building elements, and (d) observation of site context that may attribute to building condition. The case study process also requires an understanding of, (i) age of the building, (ii) the construction of historical buildings, (iii) interpretation of the architectural style, (iv) historical record on the urban development, (v) introduction of modern utilities, and (vi) history of its functional use. This building survey study will serve as a guideline to manage the building maintenance for modern use. The research is considered applied. As for the method used in the work, this is a case study, since it seeks improvement opportunities for a defect element on building, more specifically in the heritage sector.

The data obtained from the new instrument at 3 museums in Kota Bharu, Kelantan. The score for each three criteria were totalled to obtain the condition of the building. Based on the model's equations, according to Eq. (1) to (5), and the structure of the hierarchical decision tree by S. N. F. Zuraidi et al., 2018b, a model simulation with three museums for evaluating outsourced defects was created. Museum A, B, and C in Kelantan assembly was taken as a case study. The, scores on a 1- 5 scale were established with an increasing degree of importance for each attribute, and thus each museum will be measured.

RESULT AND DISCUSSION

This study was carried out on three heritage museums in Malaysia, consisting of palace and offices in Peninsular Malaysia. Data required for the evaluation of building condition obtained by building inspection works. Data collection and analysis conducted based on new instrument. There are 3 heritage museums in Kota Bharu (Vatan, 2011), and the sampling criteria used are based on building age, which refer to the first building constructed for the museum. Museum age is range from 80 year to 100 years above. The range of buildings within the study has been restricted by several parameters, such as building operation and age to provide a coherent and comparable data set. The buildings selected for the study were also required to be in 100% operation. Due to limited budget for this research, only 3 buildings were used for the data collection. The condition of building component is evaluated using a Building Condition Assessment. These code and protocol are a guideline to the Building Condition Assessment (BCA) to assess any defect of building based on priority and condition. This matrix has its own scoring system (Roca, 2007) to facilitate the examiner to assess the condition of heritage museum carefully and entirety. All defects identified are assessed and recorded on-site with the evidences (photos). The score obtained from the scoring system determine the level of defects such as very good, good, fair, poor and very poor. Besides, the possible cause of the defects also identified. This information recorded in Defect Sheet, and then it was compiled in the Schedule of Building Condition. A summary of finding such as the number of defects, total score and building condition rating based on instrument is produced. Table 1 show the descriptions of the buildings used in the case study.

Table 1. Descriptions of the buildings used in the study

	Building Type	Built	No. of Storey
Museum A	Palace	1840	2
Museum B	Residence	1902	2
Museum C	Palace	1939	1

During decision making or evaluating the condition assessment of the building it is very important to consider building as a whole. It is needed to associate the whole data and to decide final remarks. In this case study the data on hand are: defects state of the structural elements (individually), results of the evaluation criteria's calculations.

Table 2. Score of building condition for Museum A

			Total All Condition Assessment (a)	Weightage (b)	Priority Assessment (c)	Matrix Analysis (d)	Defect No. (e)	Total Matrix Analysis (f)=(d)/(e)
Building Structure	A1	Foundation	160	0.044	0.220	7.040	40	0.176
	A2	Column	230	0.021	0.106	4.858	60	0.081
	A3	Beam	217	0.020	0.099	4.297	59	0.073
	A4	Truss	7	0.007	0.037	0.052	2	0.026
	A5	Staircase	7	0.004	0.019	0.027	2	0.014
	Total attribute for criteria building structure (g1) = ∑ of (f)							0.369
	Finale score criteria (h1) = [(g1) / (∑ of (c))]							0.768
Building Fabric	B1	Ceiling	12	0.007	0.035	0.084	4	0.021
	B2	Floor	23	0.026	0.128	0.589	7	0.084
	B3	Internal Wall	63	0.015	0.076	0.958	18	0.053
	B4	External Wall	36	0.015	0.075	0.540	12	0.045
	B5	Roof	7	0.010	0.049	0.069	2	0.034
	B6	Door	30	0.004	0.020	0.120	8	0.015
	B7	Windows	24	0.003	0.013	0.062	8	0.008
	B8	Arch	5	0.002	0.009	0.009	1	0.009
	Total attribute for criteria building structure (g2) = ∑ of (f)							0.269
	Finale score criteria (h2) = [(g2) / (∑ of (c))]							0.665
Building Service	C1	Electricity	25	0.012	0.060	0.300	7	0.043
	C2	Air Condition	12	0.006	0.032	0.077	3	0.026
	C3	Fire Protection	12	0.003	0.016	0.038	3	0.013
	C4	Sanitary & Plumbing	4	0.001	0.006	0.005	1	0.005
	Total attribute for criteria building structure (g3) = ∑ of (f)							0.086
	Finale score criteria (h3) = [(g3) / (∑ of (c))]							0.755
	Total All Criteria (g) = (∑ of g)			0.725				
	Score Percentage (%)			72				
	Priority			4				
	Overall Museum Condition			Good				
	Description			Minor defects/damages, needs for monitoring, repairs, replaced to prevent serious defect/ damages				

The findings are shown in Table 2 score of building condition for museum A. The total number of defects for building structure was 163, with a total score of criteria is 0.369. While, the total number of defects for building fabric was 60, with a total score of criteria is 0.269. Besides that, the total number of defects for building service was 14, with a total score of criteria is 0.086. The sum of defects scores was divided by the number of defects to obtain the total score. In this study, the total score was 0.725, which merits a "Good" overall museum rating. The description maintenance action is minor defects but needs for monitoring, repairs, replaced to prevent serious defect or damages.

The same procedure was performed to simulate the museum B and C. The results reveal that in Museum B the total number of defects for building structure was 227, with a total score of criteria is 0.375. While, the total number of defects for building fabric was 89, with a total score of criteria is 0.244. Besides that, the total number of defects for building service was 27, with a total score of criteria is 0.076. The sum of defects scores was divided by the number of defects to obtain the total score. In this study, the total score was 0.695, which merits a "Good" overall museum rating. The description maintenance action is minor defects but needs for monitoring, repairs, replaced to prevent serious defect or damages.

However, score of building condition for Museum C is the total number of defects for building structure was 201, with a total score of criteria is 0.385. While, the total number of defects for building fabric was 157, with a total score of criteria is 0.286. Besides that, the total number of defects for building service was 29, with a total score of criteria is 0.086. The sum of defects scores was divided by the number of defects to obtain the total score. In this study, the total score was 0.756, which merits a “Good” overall museum rating. The description maintenance action is minor defects but needs for monitoring, repairs, replaced to prevent serious defect or damages.

Table 3. Finale score for case study in Kelantan

Museum Name	Museum A	Museum B	Museum C
Building Structure	0.369	0.375	0.385
Building Fabric	0.269	0.244	0.286
Building Service	0.086	0.076	0.086
Total All Criteria	0.724	0.695	0.757
Score Percentage (%)	72.4	69.5	75.7
Priority	4	4	4
Overall Museum Condition	Good	Good	Good
Scale	Routine	Routine	Routine

Table 3 shows the finale score for case study in Kelantan obtained by the new instrument for evaluating the criteria and attribute of condition in heritage building. Museum C has the highest score, thus making it the museum with the best evaluation according to the objective of achieving the maximum value in the objective function. Therefore, according to the established criteria, museum C has the best conditions for providing with minor defect, constantly seeking to improve performing as intended. The score of museum C is 75.7% higher score than another museum. The maintenance action needs are minor defects/damages, needs for monitoring, repairs, replaced to prevent serious defect or damages.

CONCLUSION

The results of condition assessment of 3 museum in Kota Bharu supported the fact that there are key principles for visual observation of heritage buildings in order to determine the present state of the building and build up the data for detailed investigation. This first step work gives possibility for assessing seismic risk of heritage building structures, prioritizing repair and restoration works, managing the budget for those works. Particularly in countries that have a few specialists and many building structures, less budgets etc. this is very important to make first step survey.

In preservation of the heritage building, condition assessment of building structure will help to identify the potential existing heritage buildings for hazard mitigation, disaster preparedness and prior knowledge of potential hazards. Depending on the aim, budget and requirements there are simple and detailed methods for assessing the present condition of heritage building. Detailed analyses are technically complex, expensive, take more time and can be applied to limited number of buildings. Since the experts on the heritage building who can carry out detailed researches are a few and the heritage building stock is huge, it is very important to use visual observation methods as a first step of the building condition assessment works. The results of visual inspections will lead to detailed methods in order to prioritize the intervention works that require team of specialists.

It is very important to determine the defect or damage state of the structural elements, to research the possible causes of defect processes, monitor the building and see is there any continuation of the defect process and to take into account building as a whole during final decisions. The intervention decision should be considered by interdisciplinary team of specialists. The main basis of intervention decision is the results of investigation of possible damages' causes and to propose an approach for preventing them otherwise the whole works could be just making up and damage process could be accelerated.

ACKNOWLEDGMENT

This research has been supported by Postgraduate Student Centre, University Tun Hussein Onn Malaysia, for making this publication possible.

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FORM-FINDING OF MEMBRANE IN THE FORM OF COSTA USING NONLINEAR ANALYSIS METHOD

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Abstract
Tensioned membrane structures are suitable to be used in large space area. Form-finding of membrane surface bordered by Costa has investigated. The possibility of adopting the form of Costa as a surface for tensioned membrane structure has studied also. The combination of the shape and internal forces for stiffness and strength is a feature of the membrane. For this purpose, form-finding needs to be carried out. Nonlinear analysis method has used in the form-finding analysis of the Costa. Form-finding has been found to converge for the case of half-Costa tensioned membrane structure models in XZ-plane, models with the larger size opening, smaller size opening and mathematically defined opening in the initially assumed shape. Convergence has obtained in form-finding of half-Costa tensioned membrane structure models in YZ-plane with larger size opening and opening lying in the horizontal plane in the initially assumed shape.

Keywords: *Costa; form-finding; membrane; boundary.*

INTRODUCTION

The growth of construction activities came from the industrial buildings and commercial infrastructures. On top of that, construction activity has been a stimulant for economic growth as it steams up development in other sectors (Ismail, 2007). Tensioned membrane structure has turned into a new leaf as another type of structure that would be more efficient as an application for big infrastructures project. Besides that, building materials are important in determining the surrounding microclimate on human thermal comfort. Thus, the designer should take tensioned membrane structures into the consideration (Harimi et al., 2007). Additionally, sustainable environment can achieve by focusing on sustainable building material. This has been given more attention in construction field (Misnon et al., 2017).

Tensioned membrane structure is highly suitable to be used for realizing surfaces of complex or new forms. Apart from other mathematical models, another source of inspiration which has adopted as a form for tensioned membrane structure is the recently discovered Costa minimal surface by Brazilian mathematician Celso Costa in 1984. Figure 1 shows a Costa surface.

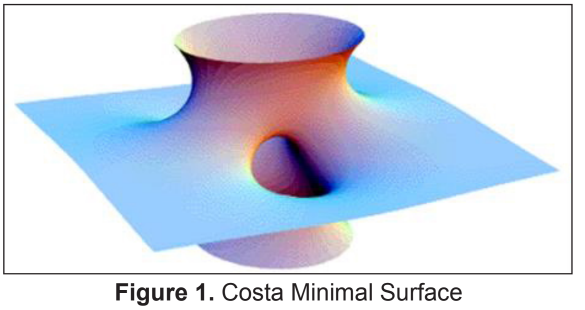


Figure 1. Costa Minimal Surface

It belongs to the class of unbounded minimal surfaces. It is also a surface of finite topology, which means that it has formed by puncturing a compact surface. Weber (1998) has mentioned that Costa's surface has a continuous deformation, all these have constructed with higher dihedral symmetry and one can increase the number of ends of these examples by one or two or even increase the topology of three ended. The portion of the Costa minimal surface has the potential to be applied to form of tensioned membrane structure, creating a unique structural form. Although a study was carried out on tensioned membrane structure in the form of Costa minimal surface, e.g. form-finding on Costa minimal surface by Pauletti and Pimenta (2009), more in-depth study on the aspect of form-finding has not been carried out. Furthermore, the possible use of the portion of Costa minimal surface as a form of tensioned membrane structure has not studied. Such use of the portion of Costa minimal surface is seen to be more suitable than the whole of Costa. Among the few works which have cited are Cipra (1999) which mentioned about a snow sculpture in the form of Costa minimal surface and Pauletti and Pimenta (2009) who carried out form-finding on tensioned membrane structure in the form of Costa minimal surface. No other works on the use of Costa minimal surface as an idea of structures have found.

Tensioned membrane structure is very versatile in producing the structure with different varieties of surface shapes. Previous studies have shown that, Abdul Hadi et al. (2016), Mohd Noor et al. (2013), Yee & Abdul Hadi (2015a), Yee & Abdul Hadi (2015b), Yee & Abdul Hadi (2016), Yee & Arabi (2015), Yee & Norman (2015), Yee & Samsudin (2014a), Yee & Samsudin (2014b), Yee (2011), Yee, Abd Malek & Abdul Hadi (2015), Yee, Abd Malek & Aziz (2017), Yee, Abdul Hadi & Choong (2015), Yee, Abdul Hadi, & Abdul Hamid (2015), Yee et al. (2016), Yee, Choong, & Kim (2011) and Yee, Kim, & Mohd Noor (2013) have carried out form-finding using nonlinear analysis method in Catenoid, Helicoid, Scherk, Enneper, Oval, Costa, Moebius Strip, Monkey Saddle, Chen-Gackstatter, Bour's, Richmond's, Modified Chen-Gackstatter and Cable Reinforced Bour's TFS models. Tensioned membrane structures with a surface in the form of Costa minimal surface have not studied by other researchers. This study was carried out as an initial study on characteristics of initially equilibrium shape of tensioned membrane structure in the form of Costa minimal surface. Understanding the possible initially equilibrium shapes to be obtained will provide alternative shapes for designers to consider. Before the shapes have considered for structural application, their behaviour under load must be properly studied. For the case of tensioned membrane structures, the first step in any structural analysis is the determination of initially equilibrium shape. Factors affecting initially equilibrium shape in the form of Costa minimal surface need to be studied.

GENERATION OF COSTA MEMBRANE SURFACE

Form-finding of half-Costa tensioned membrane structure model has carried out. Due to the reason that half-Costa tensioned membrane structure model is more suitable to be applied as roof structures then full Costa. Figures 2 and 3 show the half-Costa TFS models in XZ and YZ-plane, respectively. The surface of full Costa as shown in Figure 4 has split down an axis of symmetry in the XZ and YZ-plane which as shown in Figure 2 and 3.

The surface of tensioned membrane structure in the form of Costa as shown in Figure 4 has evaluated by using the following set of the equation as mentioned by Gray (1999):

$$x = \frac{1}{2}R\left\{-\zeta(u+iv) + \pi u + \frac{\pi^2}{4e_1} + \frac{\pi}{2e_1}\left[\zeta\left(u+iv-\frac{1}{2}\right) - \zeta\left(u+iv-\frac{1}{2}i\right)\right]\right\} \tag{1}$$

$$y = \frac{1}{2}R\left\{-i\zeta(u+iv) + \pi v + \frac{\pi^2}{4e_1} - \frac{\pi}{2e_1}\left[i\zeta\left(u+iv-\frac{1}{2}\right) - i\zeta\left(u+iv-\frac{1}{2}i\right)\right]\right\} \tag{2}$$

$$z = \frac{1}{4}\sqrt{2\pi}\ln\left|\frac{\rho(u+iv)-e_1}{\rho(u+iv)+e_1}\right| \tag{3}$$

where $\zeta(z)$ is the Weierstrass zeta function, $\rho(g_2, g : z)$ is the Weierstrass elliptic function with $(g_2,g_3) = (189.07272\ldots.0)$ and $e_1 = 6.87519$.

The mathematically defined Costa minimal surface has used for the geometry of Costa model. All x, y and z translation of nodes lying along the boundary edge of the half-Costa models have restrained. The member pretension in the warp and fill direction, denoted as σ_W and σ_F respectively, was 7000N/m. The shear stress was zero. The total number of 343 nodes and 576 elements were used to generate the half-Costa tensioned membrane structure model. The material properties for half-Costa membrane model in XZ and YZ-plane membrane model used were based on Cheong (2005) and shown in Table 1.

Table 1. Material Properties for half-Costa membrane model in XZ and YZ-plane

Tensile modulus in the warp direction, $E_W t$	429200N/m
Tensile modulus in the fill direction, $E_F t$	337910N/m
Poisson's ratio corresponds to warp direction, v_W	0.84
Poisson's ratio corresponds to fill direction, v_F	0.57
Shear modulus, Gt	64700N/m
Modulus of elasticity of cable, E_c	1.4×10^{11} N/m^2

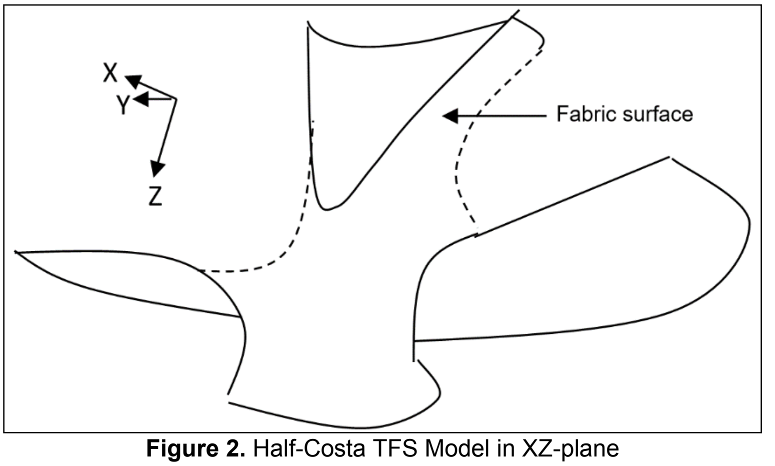


Figure 2. Half-Costa TFS Model in XZ-plane

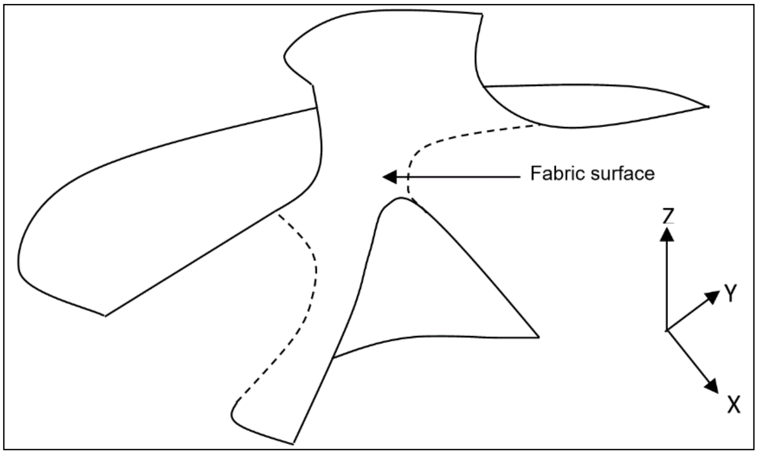


Figure 3. Half-Costa Tensioned Membrane Structure Model in YZ-plane

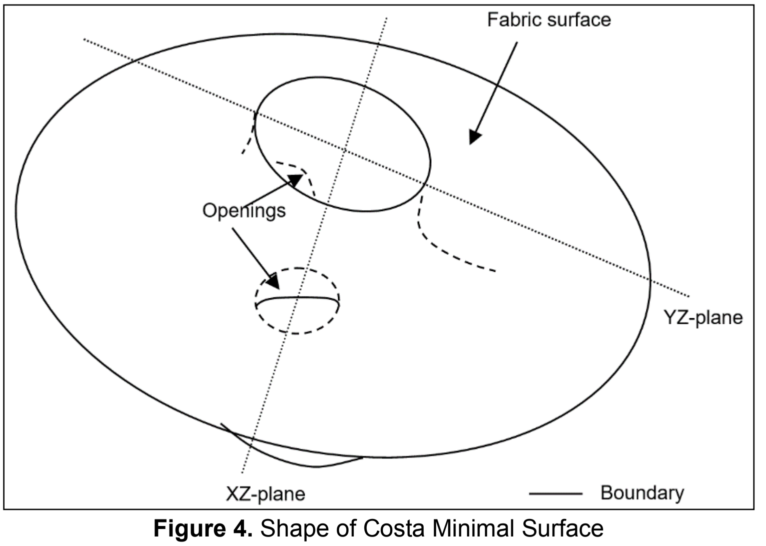


Figure 4. Shape of Costa Minimal Surface

COMPUTATIONAL STRATEGIES FOR FORM-FINDING ANALYSIS USING NONLINEAR ANALYSIS METHOD

Nonlinear FE analysis procedures for stress analysis of tensioned membrane structures proposed by Yee (2011) has used for form-finding in this study. The principle of nonlinear analysis method has based on Equation [4]. The large displacement finite element formulation used for analysis of structural behaviour under external loads. Since the method has used for both the converged shape problem and load analysis, the approach using nonlinear analysis is quite common. The basic equation used has expressed as follows:

$$({}_0^t\boldsymbol{K}_L + {}_0^t\boldsymbol{K}_G)\boldsymbol{u} = {}^{t+\Delta t}\boldsymbol{F} - {}_0^t\boldsymbol{f} \tag{4}$$

Where ${}_0^t\boldsymbol{K}_L$ is linear strain incremental stiffness matrix, ${}_0^t\boldsymbol{K}_G$ is nonlinear strain incremental stiffness matrix, ${}_0^t\boldsymbol{f}$ is vector internal forces, ${}^{t+\Delta t}\boldsymbol{F}$ is load vector and $\boldsymbol{u}$ is a vector of increment in displacement.

As given in Figure 5, a first shape for the start of form-finding procedure adopted in this study was called an initially assumed shape which was needed. The proposed computational strategies involve two phases of analysis in one cycle. The first phase (denoted as Phase I) was an analysis which starts with an initially assumed shape to obtain an updated shape for initially assumed shape. This was then followed by the second phase of analysis (denoted as Phase II) aimed at checking the convergence of updated shape obtained at the end of Phase I by means of iterative calculation. The resultant shape at the end of the iterative step was considered in the state of initially equilibrium under the prescribed warp and fill stresses and boundary condition if the difference between the obtained and the prescribed tensioned fabric stresses relative to the prescribed stress was negligibly small. Such checking of difference in the obtained and prescribed stresses has presented in the form of total stress deviation in the warp and fill direction versus analysis step. The criteria adopted for checking of convergence of form-finding was least square error (LSE) of total warp and fill stress deviation.

As a first shape for the start of form-finding procedure proposed by Yee (2011), initially assumed shape was needed. For the generation of such initially assumed shape, knowledge of the requirement of anti-clastic nature of membrane surface has used. The incorporation of anti-clastic feature into the model will help to produce a better initially assumed shape. The difficulty to control final shape has overcome by starting the form-finding form on properly selected initially assumed shape. Due to the reason Costa minimal surface has no boundary, various assumptions have made about the boundary for generation of half-Costa TFS model. Initially equilibrium shapes of membrane structures were solved under a condition of equal tension in the warp and fill directions corresponds to minimal surfaces.

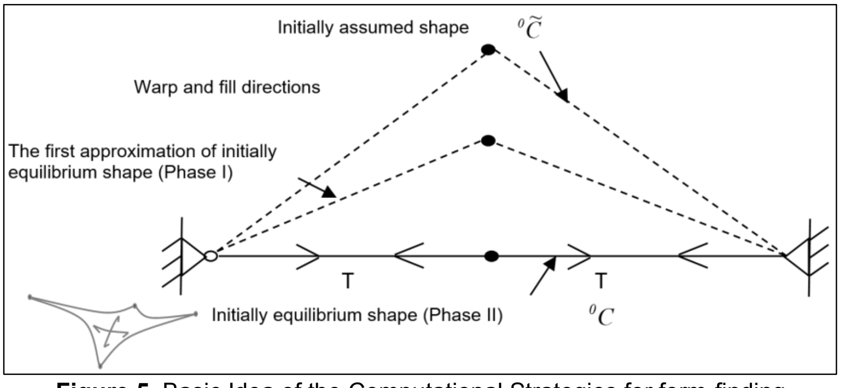


Figure 5. Basic Idea of the Computational Strategies for form-finding

ADINA (2003) has chosen as a tool for mesh generation in this study. The orthotropic nature of fabrics materials as given in Figure 6. The checking of the orientation of mesh in terms of element node sequence is essential because the orientation of element nodes directly affects the direction of material axes. Figure 7 shows the direction of element nodes for an element. The first edge connecting i and j nodes corresponds to the warp direction which has used in the FE formulation. The direction that was perpendicular to the warp direction (edge i-j) corresponds to the fill direction. The edge i-j which has oriented to the warp direction was defined first. Then the third element node k was numbered in such a way that the sequence i-j-k was in a counterclockwise direction. The proper sequence of node i-j-k for all elements was important for the generation of a good mesh representing the shape of membrane model.

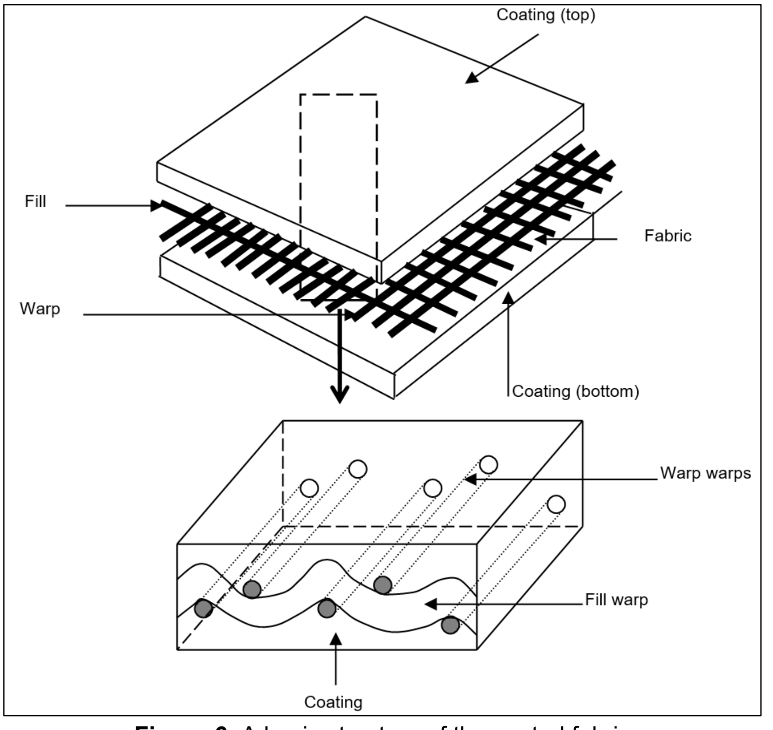


Figure 6. A basic structure of the coated fabric

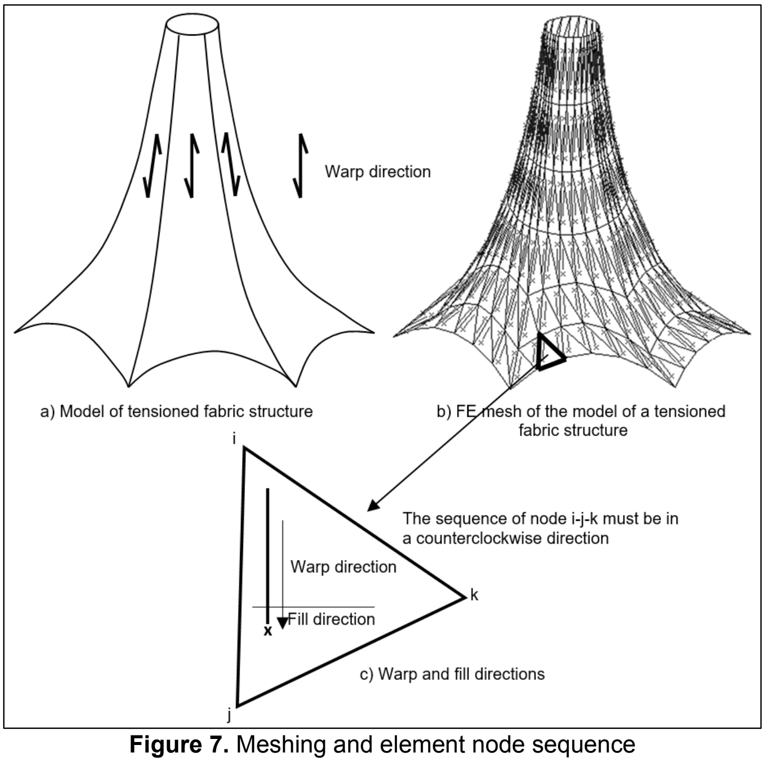


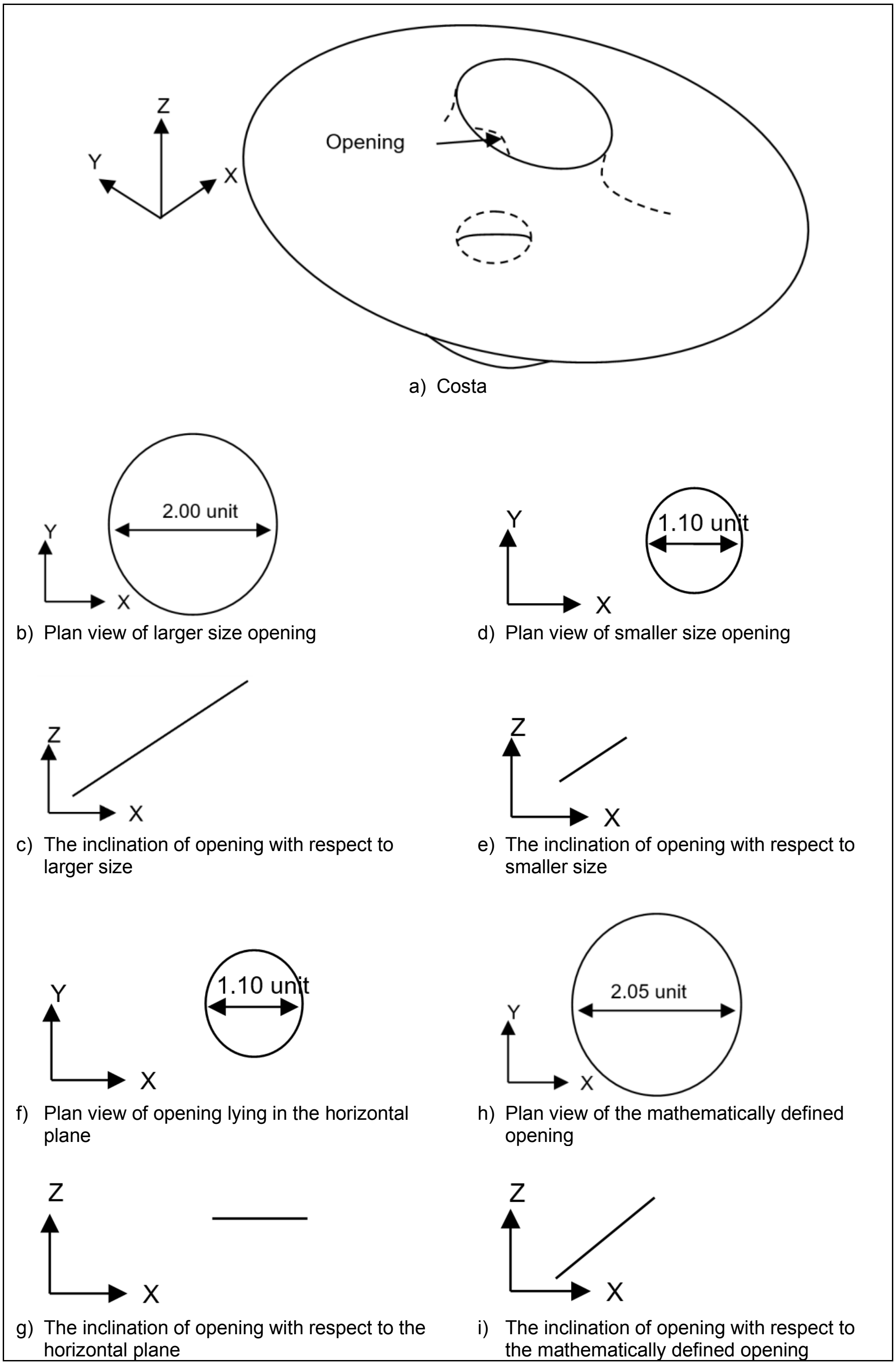
Figure 7. Meshing and element node sequence

NUMERICAL ANALYSIS

Half-Costa Tensioned Membrane Structure Model in XZ and YZ-plane

Four of the initially assumed shape of half-Costa tensioned membrane structure models in XZ-plane were generated based on different orientations of openings as shown in Figure 8. Similar of the case of half-Costa tensioned membrane structure model in XZ-plane, four of the initially assumed shape of half-Costa tensioned membrane structure models in YZ-plane were generated based on four cases of orientations of openings as shown in Figure 9.

Figure 10 shows the initially assumed shape of half-Costa tensioned membrane structure model with larger size opening. Figure 11 shows different views and converged shape of the half-Costa tensioned membrane structure model. The ratio of warp to fill stresses, $\sigma W/\sigma F$ obtained for the half-Costa tensioned membrane structure model in Figure 12 is approximately equal to 1.000679. Figure 12 shows the variation of total stress deviation for warp and fill stresses versus stress analysis stage. Figure 13 shows the variation of root mean square deviation (RMSD) versus the stress analysis stage.



a) Costa

b) Plan view of larger size opening

c) The inclination of opening with respect to larger size

d) Plan view of smaller size opening

e) The inclination of opening with respect to smaller size

f) Plan view of opening lying in the horizontal plane

g) The inclination of opening with respect to the horizontal plane

h) Plan view of the mathematically defined opening

i) The inclination of opening with respect to the mathematically defined opening

Figure 8. Initially Assumed Shape of the Opening in Half-Costa Tensioned Membrane Structure Model in XZ-plane

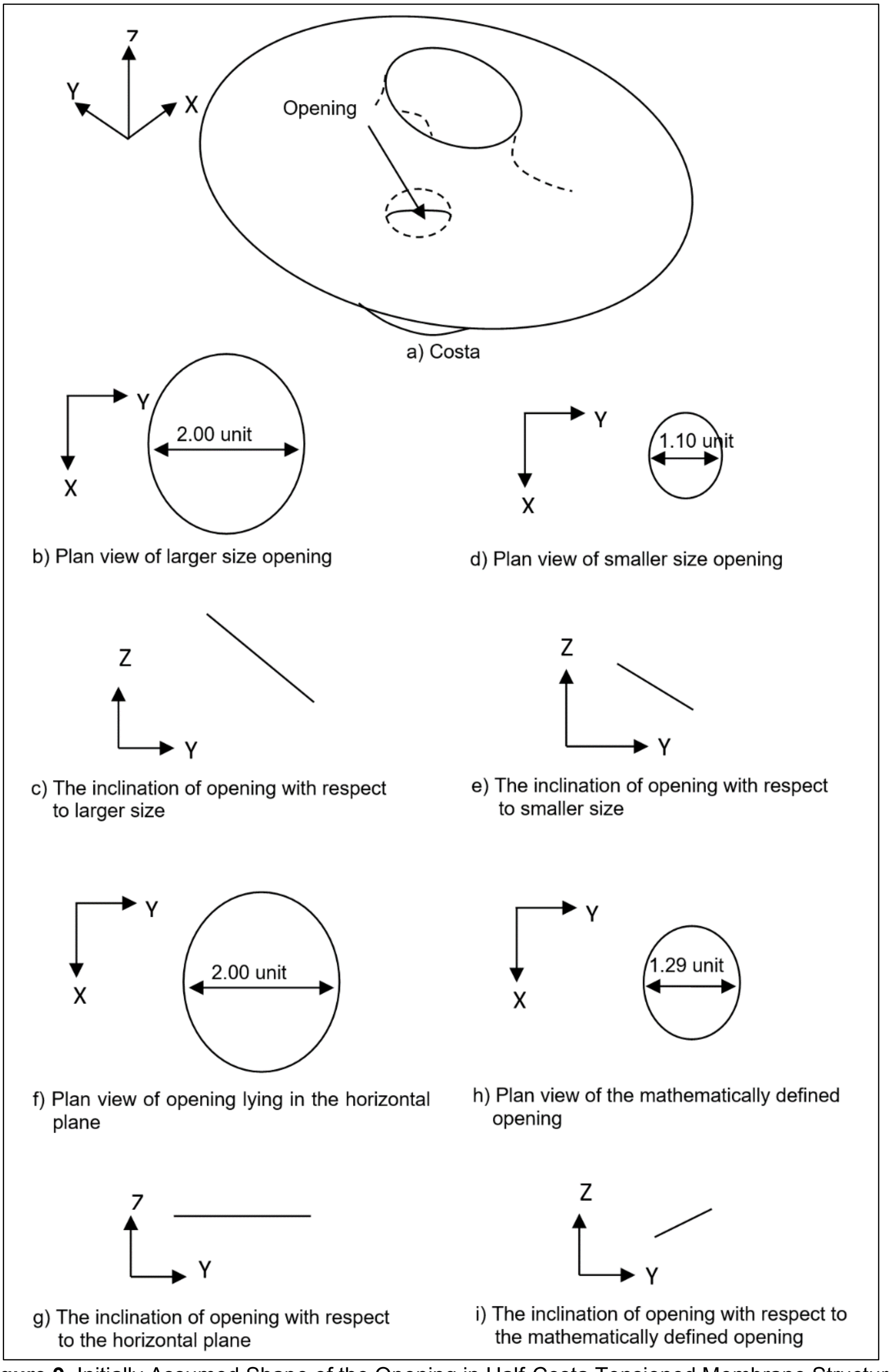


Figure 9. Initially Assumed Shape of the Opening in Half-Costa Tensioned Membrane Structure Model in YZ-plane

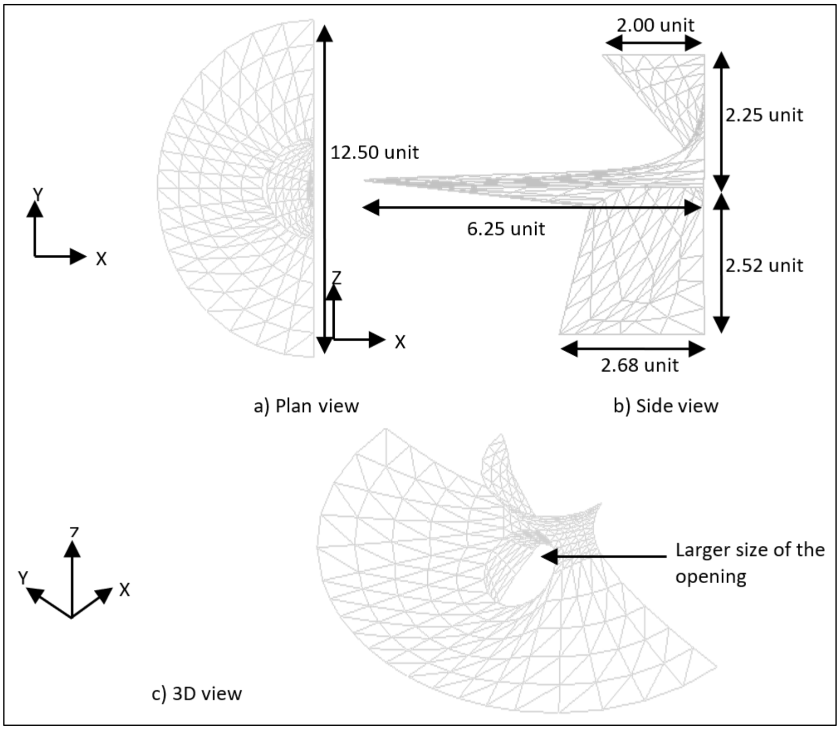


Figure 10. Initially Assumed Shape of Half-Costa Tensioned Membrane Structure model in XZ-Plane (with larger size opening)

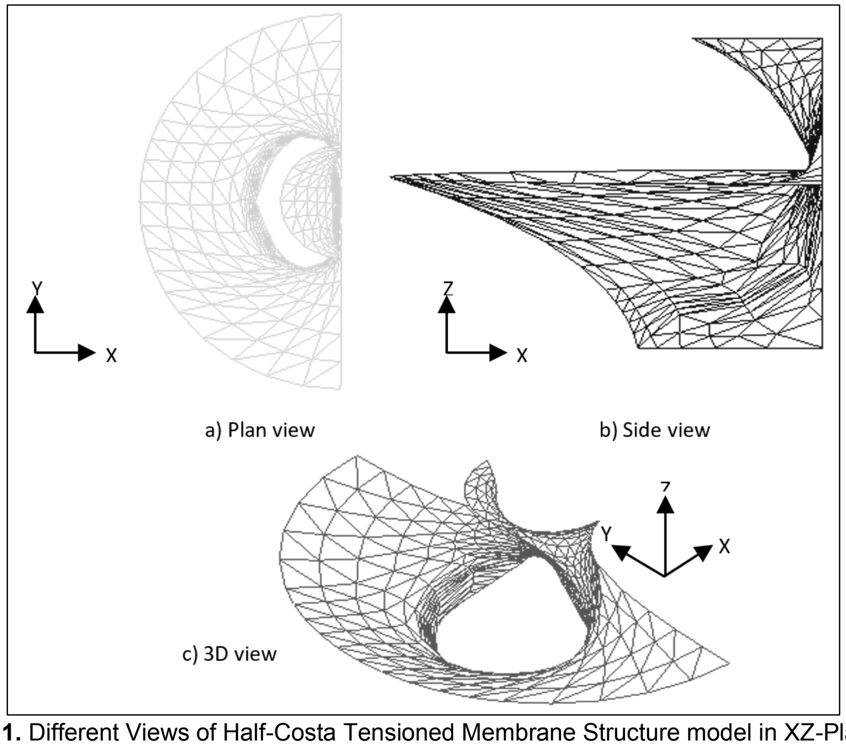


Figure 11. Different Views of Half-Costa Tensioned Membrane Structure model in XZ-Plane after Form-Finding (with larger opening)

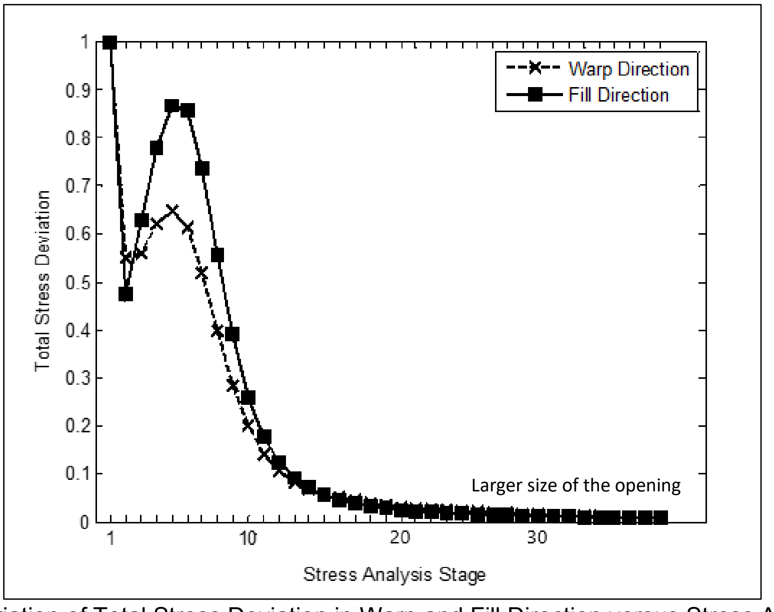


Figure 12. Variation of Total Stress Deviation in Warp and Fill Direction versus Stress Analysis Stage of Half-Costa Tensioned Membrane Structure model in XZ-Plane (with larger opening)

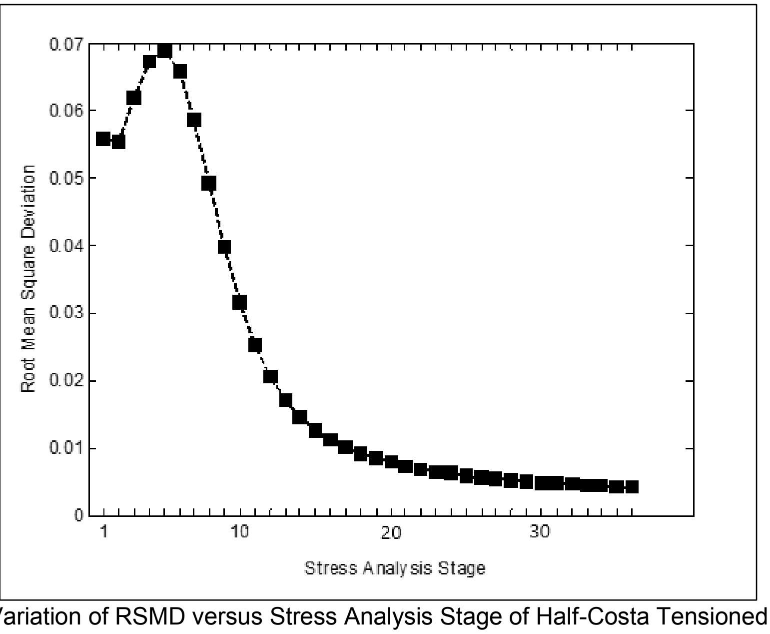


Figure 13. Variation of RSMD versus Stress Analysis Stage of Half-Costa Tensioned Membrane Structure Model in XZ-Plane (with larger size opening)

RESULTS AND DISCUSSION

Effect of opening orientation specified in the initially assumed shape for form-finding of half-Costa tensioned membrane structure models has studied. For the case of half-Costa tensioned membrane structure models in XZ-plane, models with larger size opening, with smaller size opening and the mathematically defined opening were found to converge with a non-monotonous decrease in the least square error (LSE) of total warp and fill stress deviation as shown in Figure 12. The trend of non-monotonous variation of LSE versus stress analysis stage as shown in Figure 12 is probably due to the large changes in curvature of the surfaces surrounding the opening of the Costa surface as stated by Yee (2011). The ratio of warp to fill stresses, σ_W/σ_F obtained for the half-Costa tensioned membrane structure model were in the range of 1.000275 to 1.102217. A similar trend of non-monotonous convergence has observed in the curve of variation of RMSD versus stress analysis stage (see Figure 13). Convergence has obtained in form-finding of half-Costa tensioned membrane structure model in YZ-plane modelled with larger size opening and opening lying in the horizontal plane. The divergence of form-finding has observed in half-Costa tensioned membrane structure in XZ-plane models with the opening lying in the horizontal plane and in half-Costa in YZ-plane model with smaller size opening and with the mathematically defined opening.

CONCLUSION

Form-finding on tensioned membrane structure models in the form of half-Costa in XZ-plane and YZ-plane has shown better results than tensioned membrane structure models in the form of full Costa. For the case of half-Costa tensioned membrane structure models in XZ-plane, models with larger size opening, smaller size opening and mathematically defined opening in the initially assumed shape were found to converge. Convergence has obtained in form-finding of half-Costa tensioned membrane structure model in YZ-plane modelled with larger size opening and opening lying in the horizontal plane in the initially assumed shape. The divergence of form-finding has observed in half-Costa tensioned membrane structure in XZ-plane models with the opening lying in the horizontal plane and in half-Costa in YZ-plane model with smaller size opening and with the mathematically defined opening in initially assumed shape. The results of half-Costa models show that the convergence of form-finding was sensitive to initially assumed shape especially with respect to the orientation of opening in Costa surface.

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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

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[1]*Department of Quantity Surveying, Faculty of Architecture, Planning and Surveying, Universiti Teknologi MARA, Sarawak, Malaysia*
[2] *Institute of Ocean and Earth Sciences (IOES), University of Malaya, Malaysia*

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

Keywords:(Arial Bold, 9pt) *Finite Element Analysis; Modal Analysis; Mode Shape; Natural Frequency; Plate Structure (Time New Roman, 9pt)*

HEADING 1 (Arial Bold + Upper Case, 11pt)

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.

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.

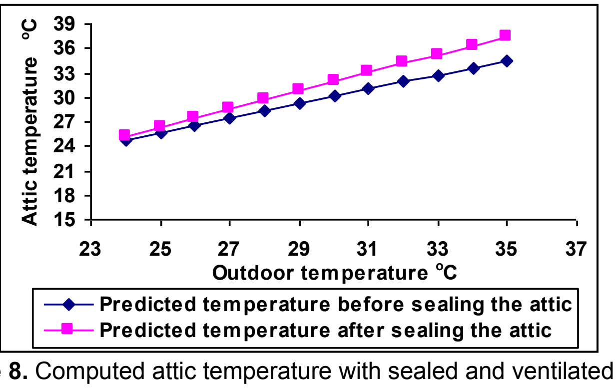

Figure 8. Computed attic temperature with sealed and ventilated attic

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)

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

Citation:

Passage Type	First reference in text	Next reference in text	Bracket format, first reference in text	Bracket format, next reference marker in text
One author	Walker (2007)	(Walker, 2007)	(Walker, 2007)	(Walker, 2007)
Two authors	Walker and Allen (2004)	Walker and Allen (2004)	(Walker & Allen, 2004)	(Walker & Allen, 2004)
Three authors	Bradley, Ramirez, and Soo (1999)	Bradley et al. (1999)	(Bradley, Ramirez, & Soo, 1999)	(Bradley et al., 1999)
Four authors	Bradley, Ramirez, Soo, and Walsh (2006)	Bradley et al. (2006)	(Bradley, Ramirez, Soo, & Walsh, 2006)	(Bradley et al., 2006)
Five authors	Walker, Allen, Bradley, Ramirez, and Soo (2008)	Walker et al. (2008)	(Walker, Allen, Bradley, Ramirez, & Soo, 2008)	(Walker et al., 2008)
Six or more authors	Wasserstein et al (2005)	Wasserstein et al. (2005)	(Wasserstein et al., 2005)	(Wasserstein et al., 2005)
Organisation (easily identified by the initials) as the author	Sultan Idris Education University (UPSI, 2013)	UPSI (2013)	(Sultan Idris Education University [UPSI], 2013)	(UPSI, 2013)
Organisation (No abbreviation) as the author	Pittsburgh University (2005)	Pittsburgh University (2005)	(Pittsburgh University, 2005)	(Pittsburgh University, 2005)

(Source: UPSI, 2019)

Reference: Times New Roman, 11pt. Left indent 0.64 cm, first line left indent – 0.64 cm.

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MCRJ Volume 31 | No.2 | 2020

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ISSN 1985-3807
9 771985 380005

eISSN 2590-4140
9 772590 414000