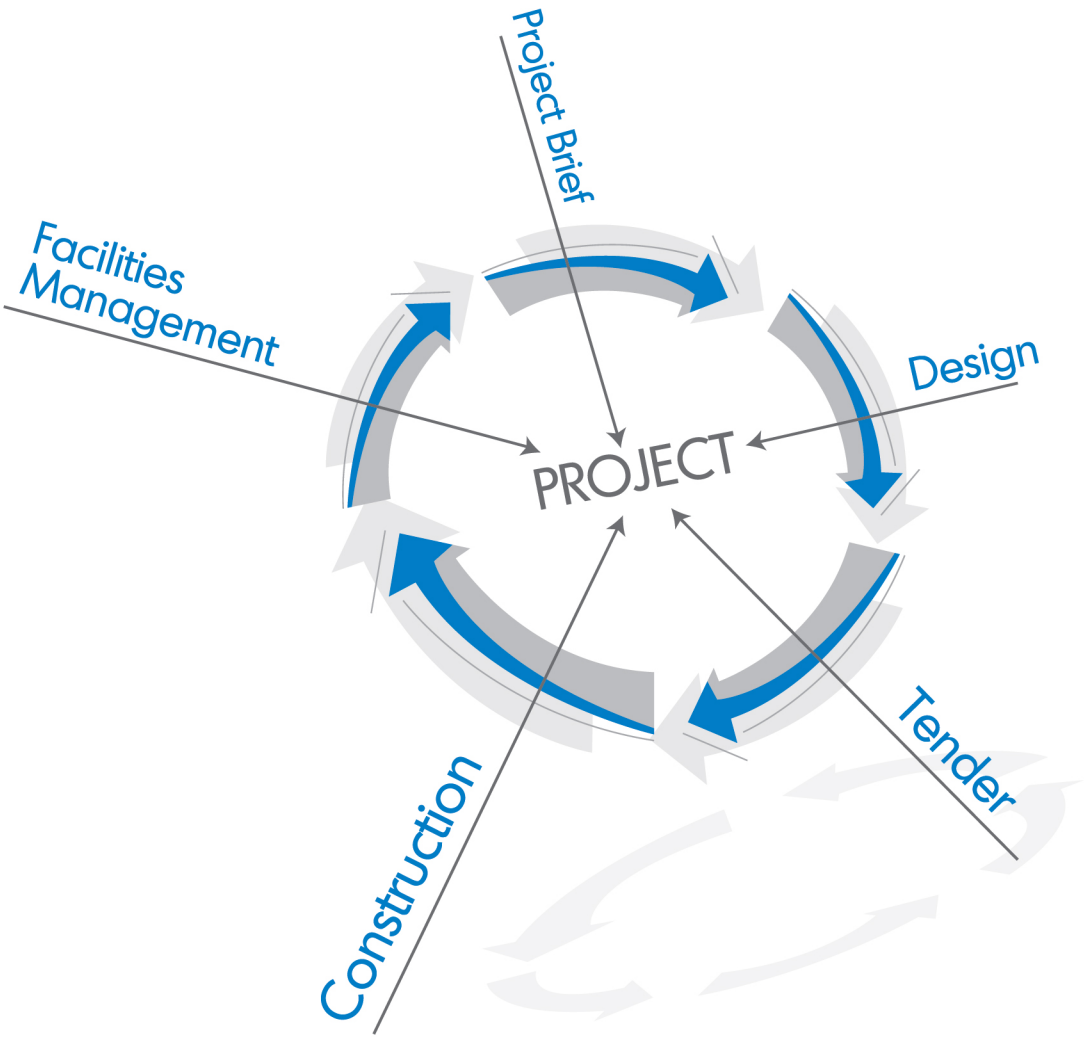
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Editorial

Welcome from the Editors

Welcome to the thirty-third (33rd) issue of Malaysian Construction Research Journal (MCRJ). In this issue, we are pleased to include eight papers that cover a wide range of research areas in construction industry. The editorial team would like to express our sincere gratitude to all contributing authors and reviewers for their contributions, continuous support and comments.

In this issue:

Hasmawi Khalid et al., suggested the new process of drying and treatment process of Oil Palm Stem (OPS) veneer. The methods used in this research is through experimental test-up for the physical properties of the plywood for moisture contents, density, delamination and wetting properties. Three samples used in this research denoted as PTA, PTB and PTC. The finding shows that the pre-dying of the OPS veneer and medium molecular weight phenol formaldehyde (MMwPF) treatment process has resulted in lower MC for PTA.

Norshariza Mohamad Bhkari et al., presented the flexural strength properties and the integrity of glue lines of glulam manufactured from Malaysian hardwood timber species in various strength grades. Five methods used in this research which are flexural test, glue line integrity test, block shear, delamination and moisture content and density. It can be concluded that the flexural strength for glulam beam from various strength grades depends on their density and moisture content. The higher strength grade with higher density provides higher flexural strength.

Yusrin Wahab et al., established the new approach on landslide vulnerability assessment and landslide risk index for critical infrastructures in Malaysia. Qualitative methods using vulnerability matrices, vulnerability curves, vulnerability indicators are implemented in this research. It is suggested that the semi-quantitative indicator-based method (IBM) is suitable for landslide vulnerability assessment. IBM and vulnerability matrices approach requires less amount and detailed information.

Ummi Junaidda Hashim et al., identified whether the specific characteristics of construction company will influence the Key Audit Matters (KAMs) reporting for Malaysian companies. 46 companies are selected in this research and 14 KAMs items is identified. The revenue recognition is among the items that has been identified as the important issues to be raised as the KAMs in construction sector with a total of 49 items and the final of 3 items of KAMs extract has been justified.

Oveys Ghodousian et al., investigated about behaviour and the compelling factors on the concrete. The methodology used in this research is Fuzzy series theory to analysing of vague and imprecise concepts by using a 28-day strength of concrete. Test implementation test are using slump loss, compressive strength and the relationship between the type and amount of pozzolan with compressive strength. The findings found that generalized nearest neighbourhood clustering method has the most accurate prediction of 28-day compressive strength.

Sharifah Nurfarhana Syed Sahimi et al., assessed the relationship of good management practices towards the successful implementation of Green Supply Chain Management (GSCM) in IBS construction project. Mixed methodology was adopted in this study through survey questionnaire and interview session. Ten management practices towards the successful implementation of GSCM in IBS construction is highlighted.it was found that all management practices are significant to the successful implementation of GSCM in IBS construction.

May Raksmey and Siti Aishah Sahul Hamid investigated the saltwater intrusion phenomena in coastal aquifers. Experiment's method was used in this research using laboratory rectangular model tank filled with fresh water, well sorted sand and salt water into three separate chambers. The experiment was carried out to observing the patterns of saltwater-freshwater interface inside the tank. The finding shows that the higher order of magnitude in direct surface recharge and constant freshwater head can be used to attenuate or eliminate the rising tide of salinity from further encroachment into our vital groundwater resources near coastal areas.

Siti Jahara Matlan et al., analysed of rainfall effect on slope failure in Sabah. Soil Water Index (SWI) method was used to 10 landslide events that occurred in Ranau, Sabah area between years 2007 to 2015 using rainfall threshold. The findings shows that the landslide events did not occurred during the highest daily rainfall thus concluded that the prolonged and continuous rainfall could play a role in building up the mechanism of landslide.

Editorial Committee

EFFECT OF VENEER TREATMENT WITH PHENOL FORMALDEHYDE ON PHYSICAL AND WETTING PROPERTIES OF OIL PALM STEM (OPS) PLYWOOD

Hasmawi Khalid[1], Zakiah Ahmad[2], Paridah Md. Tahir[3] and Jamaludin Kasim[4]
[1]Civil Engineering Department, Polieknik Sultan Salahuddin Abdul Aziz Shah, Selangor, Malaysia.
[2]Faculty of Civil Engineering, Universiti Teknologi MARA, Selangor, Malaysia.
[3]Institute of Tropical Forestry and Timber Product (INTROP), Universiti Putra Malaysia, Selangor, Malaysia.
[4]Faculty of Applied Sciences, Universiti Teknologi MARA, Pahang, Malaysia.

Abstract
Plywood is wood products frequently utilized by building as in concrete formwork systems. During concreting, the plywood formwork needs to be tough in order to withstand the pressure subjected by the wet concrete as well as the impact. To reduce the dependency on solid timber as plywood, oil palm stem offers the alternative materials. There are four main weaknesses of the oil palm stem (OPS) as a material for plywood manufacturing; high variations in density and moisture content (MC), high water absorption and high surface roughness. Therefore, this study modified the current process of drying and treatment process of OPS veneer. The veneer was pre-dried by using roller presser for removing a large amount of water and later treated with medium molecular weight phenol formaldehyde (MMwPF) in order to bond together parenchyma tissue, cell wall and lumen for much uniform density and better surface bonding. Then these veneers were used for manufactured plywood using 100% OPS and denoted as a PTA. In order to determine the performance of this plywood, a series of physical tests was conducted, namely moisture content, density, delamination and contact angle. These properties were compared with the properties of another two types of commercial OPS plywood denoted as PTB and PTC which were manufactured using OPS veneer with tropical hardwood veneer for face and back veneer and control plywood denoted as PTD which was manufactured using 100% tropical hardwood veneer. The results show that the pre-drying of the OPS veneer and the MMwPF treatment process has resulted in lower MC for PTA. All plywood passes the delamination requirement based on JAS 1751:2008. PTB exhibited highest density and contact angle value for PF resins. Whereas PTD has highest contact angle value for distilled water.

Keywords: *Delamination; oil palm stem plywood; physical properties; wetting properties.*

INTRODUCTION

Plywood has been used by millennia in the construction industry and one of the major contributors to the Malaysian economy. The plywood industry from tropical wood species constituted 21% of the total timber exported products in 2015 and this is expected to increase annually (MTIB, 2015). Although the demand of plywood is increased, it also suffers due to the shrinkage of wood sources and consequently increment of the logs price (Abdul Rahim and Mohd Shahwahid, 2009). The logs production volume decreased to 4.4 million m^3 from 6.5 million m^3 in year of 2005. This was due to the heavy logging activity which damaged the potential tree for future source. In order to support wood-based panel industry, the use of wood and non-wood residue as a source is an ideal way to overcome this issue.

Oil palm *(Elaeisguineensis Jacq)* is a non-wood material and are the most abundant crops planted which can be obtained at any plantation around Malaysia. Based on statistical data of the Malaysian Palm Oil Board, there are 5.39 million ha of land are being planted with this agriculture in 2014 (Baharuddin et al., 2009; MPOB, 2014). The oil palm stems (OPS), oil palm fronds (OPF) and oil palm empty fruit bunches (EFB) is the residues that are left behind

after cutting process as the tree reach 25 to 30 years' age (Anis et al., 2011; Azizan et al., 2016). Among oil palm residues, OPS is the biggest residue with a total of 13.6 million can be obtained from 100 000 hour of replanting each year. The abundant of OPS after falling not only increase the liability of the industry due to the difficulty to be dispose, but also lowering the quality of seedlings due to the attack of the biodeteriorating agent. The uses of oil palm residue as raw material in wood- based panel reduce on the costs of production and increase on economic growth of the country. According to Paridah and Anis (2007), the oil palm stem veneer could be used as an alternative raw material for LVL and plywood manufacture. However, the drawback of OPS veneer is its high moisture content and segregation in density.

Veneer drying to reduce moisture content after peeling is an important process in plywood manufacturing (Walker, 2016). The temperature used to dry the veneer significantly affected on the mechanical and physical properties of plywood. The process took approximately 70% and 60% of production, energy consumed and mill energy requirement Ozsahin and Aydin (2014). Since moisture content of OPS could be as high as 500% (Bakar et al., 2008), high temperature is required in drying the veneer; this however increases the cost of plywood production.

Quite a number of researches have been conducted in improving the drying process of plywood veneers (Loh et al., 2010; Choo et al., 2010) in order to enhance the properties of plywood. The moisture in the veneer can be removed by roller presser method. To further enhance the properties of the plywood, the veneer was impregnated with phenolic resin. Impregnated the veneer with low molecular weight phenolic resin has resulted in the properties of the OPS plywood which is still not as good as medium molecular weight phenolic resin.

Therefore, in this study, the roller pressing method has been used for pre-dying prior to oven dry and at the same time the produced OPS veneer was impregnated with medium molecular weight phenolic resin with the aim to improve the physical and mechanical properties of plywood. The properties of this new process of OPS plywood were compared to commercial plywood.

EXPERIMENTAL TEST-UP

Materials

Plywood

Four sources of plywood were used in this study. PTA is the plywood prepared by the author using 100% oil palm plywood with the veneers treated with phenol formaldehyde. PTB and PTC were commercial oil palm plywood from two commercial factories. This plywood used tropical veneer for its face and back layer and PTD was tropical plywood.

Manufacturing of 100% Oil Palm Plywood (PTA)

Three fresh pieces of outer OPS veneers with thickness between 4.5-6.0mm were cut to the dimensions of 4x3 feet. The veneer was then feed into roller pressing machine with the gap of the roller; R_1:R_2:R_3=3.5:2.8:2.8 mm and speed of the roller was 8.0 rpm to squeeze out

the water. Then, the veneer passing through a continuous roller dryer with temperature at 150°C and speed 480 rpm for 1 hour until the final moisture content (MC) of the veneers were 0%MC. After that, Medium molecular weight phenol formaldehyde (MMwPF) with ratio 80:20 for resin and water respectively, and solid content 49.1% was applied two times on the veneer through glue spreader in gluing process. Then these veneers undergo platen press machine for pre-curing process for 9 minutes with the temperature of the plate ranging between 90-140°C. The targeted MC on this stage was between 6-20%MC. These veneers were combined together and subjected to heat and pressure in the hot press until the glue was cured to form plywood panels at a temperature 115°C and pressure 120 kg/cm^3 for 15 minutes. After curing the boards are ready to be used. Figure 1 shows the different types of plywood used in this study.

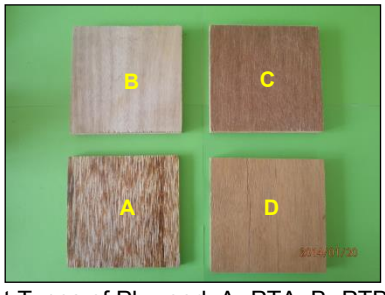


Figure 1. Different Types of Plywood; A- PTA, B- PTB, C- PTC, D- PTD

Methods

The physical properties of the plywood were determined for moisture contents, density, delamination and wetting properties.

Determination of Moisture Content

The moisture contents were determined in accordance with BS EN 322 (1993). The oven dry weight of plywood specimen was determined after drying in an oven at 103± 2° C to constant weight. The samples were reweighted after cooling in desiccators. The moisture content (H) was calculated using the following equation.

$$H = \frac{m_H - m_o}{m_o} x100 \quad (1)$$

Where,

m_H = weight of the test specimen before drying (g),
m_o = weight of the test specimen after cured and cooling(g).

Determination of Density

The density (ρ) of the specimens was determined by measuring the weight and volume of the plywood with nominal dimension of 50 mm x 50 mm. The samples were weighed with an accuracy of 0.01 g. The sample dimensions were measured using a sliding calliper, in accordance with BS EN 323 (1993). The ρ was then calculated using the following equation.

$$\rho = \frac{m}{b_1 \, x \, b_2 x \, t} x \, 10^6 \quad (2)$$

Where,

m = mass of the test pieces (g),
b_1= length (mm),
b_2 = width (mm),
t = thickness (mm).

Determination of bonding properties of delamination test

Delamination test (D) was carried out in accordance with standard method specified in Japanese Agricultural Standard JAS 1751 (2008). Plywood with nominal dimension of 75 mm (length) x 75 mm (width) were prepared. The specimens were immersed in boiling water for 4 hours and dried in an oven at 60±3°C for 20 hrs. These schedules were then repeated for 4 h and 3 h using similar drying temperature. The failed glue line over the total perimeter of glue line was calculated using the following equation.

$$D = \frac{PglF}{PglxN} x100 \quad (3)$$

Where,

$PglF$ = perimeter of failed glue line,
Pgl = perimeter of glue line,
N = number of glue line.

Determination of Contact Angle

Four specimens each, from all types of plywood (PTA, PTB, PTC and PTD) were prepared to determine the contact angle of resin droplet for usability studies. The specimens (100 mm x 20 mm) were conditioned at 20 ± 2°C and relative humidity (RH) 65 ± 3% for a week prior to testing. The tests were carried out by mean of Contact Angle Analyzer; model L&W 4-1. Using an injection tube, 5 ml of distilled water was dropped onto the surface of the plywood specimens. Contact angles were measured in time series such as 10, 20, 30,40,50,60,70,80,90 and 100 seconds for both upper and lower surface of plywood specimens. The heights and diameter of the droplets were measured from the glass plate with measuring tangent (α) and scale calculated. Three measurements of the contact angle were done on each of plywood specimens. The contact angle was calculated using the following equation.

$$\text{Contact angle} = \text{Tangent } \alpha/2 = 2\text{h}/\text{a} \quad (4)$$

Where

α = static contact angle
a = diameter of contact angle
h = height of droplet

EXPERIMENTAL RESULTS

The physical properties of plywood from four different sources were tested and evaluated. Table 1 shows the summary of the M.C, density and delamination of all plywood investigated.

Table 1. Physical Properties of All Types of Plywood

Types	M.C (%)	Density (Kg/m^3)	Delamination (%)
PTA	8.32	685.69	99.76
	12.76*	5.80*	0.47*
PTB	13.07	792.38	99.19
	4.35*	5.26*	1.32*
PTC	11.85	551.33	99.17
	3.29*	1.79*	0.96*
PTD	10.79	524.36	96.44
	2.18*	13.59*	4.66*

Where: *CoV;

Moisture Content

The amount of water in wood or a wood product is expressed as the moisture content (MC). Table 1 shows the results of the MC of the plywood. Generally, the results showed that the PTA plywood had MC of 8.32%, while commercial-plywood (PTB, PTC and PTD) had an MC range from 10.79% to 13.07%, respectively. PTA plywood which is glued with MMwPF resin had lowest moisture content value. The MC for PTA < MC to PTD < MC to PTC < MC for PTB.

The low MC for PTA may be contributed by the new process developed in this study as explained in Section 1. It is important to control the MC of wood products because the physical and mechanical properties, resistance to biological deterioration and dimensional stability of any wood-based products are affected by the amount of water present (Bowyer et al., 2007). The pre-drying of the OPS veneers as well the impregnation with phenolic resin prior to hot press to form the plywood has improved the resistance to moisture uptake. During pre-drying, the veneer has gone through pre-press to remove the moisture and by this process the veneers have become denser. The pressure of resin inside the veneer has further resisted the penetration of moisture. For normal production of plywood, the resin is only present in between layers of veneers.

For any wood products with MC below 12%, the MC is considered in equilibrium which will not affected by the changes in the moisture of the environment. This will make the products have better stability. This is due to the wood which a hygroscopic material, which absorbs moisture from its surrounding if it is dry and at the same time it will release moisture if it is in a dry surrounding and achieving moisture equilibrium with the vapor pressure around it (Dinwoodie, 2000). Therefore, for satisfactory use of wood as a raw material in construction, it has to be dried below or equal to 12% MC during the manufacturing process.

Density

The densities of all plywood are shown in Table 1. From that figure, density of PTB exhibited highest value; 792.38 kg/m^3. This may be due to the differences in the density of the raw materials that ultimately affects the plywood panel density. There are many factors that can affect the value of density. Density of light tropical hardwood is (400-720 kg/m^3) and density of OPS (200-700 kg/m^3). PTA has 100% OPS, which impregnated with phenol formaldehyde (PF) and the amount of PF absorbed in the OPS will affect the density. PTB and PTC were made from OPS and tropical hardwood veneer.

The density of PTB was (43.7%) higher than the density of PTC. This may be due to the difference in the density of tropical hardwood veneer used in this plywood. The tropical hardwood veneer used in PTB was from Sepetir with the density of (530-785 kg/m^3). Meanwhile the density of the tropical hardwood veneer for PTC was from Medang with density of 700 kg/m^3. However, the variation in the densities of the veneers and amount of PF absorbed by that plywood were not identified.

PTD was made from 100% mixed tropical hardwood which has a high variation in density due to different types of species being used for the veneers. At any production, the species were different due to availability of the timbers during production. To see if there is a correlation between the MC and density, Pearson correlation coefficient (r) was determined and was found to be 0.106. Since r = 0.106 it can be concluded that there is no correlation between MC and density.

Delamination

Table 1 shows the delamination results of four types of plywood. PTA has the highest delamination value which is 0.57% higher than PTB, 0.59% higher than PTC and 3.44% higher than PTD. Based on JAS 1751:2008, if the delamination value is not less than 90%, the plywood passes the standard requirement but if less than 90% it is considered fail. This means that all the plywood tested in this study passed the delamination test. The performance of plywood panels depends on the bonding strength between the resin and the substrate. Any failure in the glue bond weakens the mechanical strength. Bhat et al., (2010) stated that the mechanical strength of plywood depends on the glue bond between the veneers, whereas good durability is associated with the absence of delamination in the plywood samples. There are low variations in the delamination test as can be seen in the CoV values for all plywood.

Although PTA has the highest value of delamination, but it still considered pass the requirements. Since there are not much different in the delamination values in all the plywood, the tested samples were observed for any physical changes. PTA has the highest value of delamination can be associated with the highest contact angle with the resin.

Contact Angle

The contact angles for water of all panels were analysed and the results are summarized in Table 2.

Table 2. Contact Angle of Water on Different Surfaces of Plywood

Time (s)	CONTACT ANGLE (°)			
	Plywood			
	PTA	PTB	PTC	PTD
10	58.67	45.15	33.52	76.21
20	53.13	52.40	32.19	71.32
30	50.97	54.17	30.07	68.25
40	48.44	51.40	30.30	65.48
50	46.78	51.37	29.78	64.25
60	45.13	48.21	25.83	62.27
70	42.48	50.50	24.98	61.44
80	40.76	47.62	21.35	60.22
90	39.25	50.62	23.86	59.33
100	33.96	50.06	31.81	58.54

From Table 2, it can be seen that PTD has the highest contact angle (58.54°) for distilled water at 100s, followed by PTB (50.06°), PTA (33.96°) and PTC (31.81°). The lowest the value of contact angle means the surface is considered to be easily wet. Figure 2 further illustrated comparison between contact angles for water.

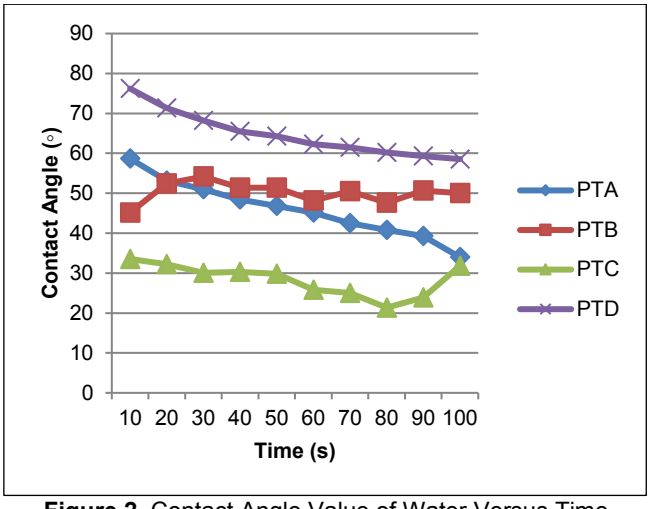


Figure 2. Contact Angle Value of Water Versus Time

Contact angles for PTD and PTA keeps steadily decreasing towards the end, while PTC constantly decreasing until 80s before contact angle keep on increasing until 100s. Meanwhile PTB shows zig-zag movement within 100s. From Figure 2, it can be seen as the time increases the contact angle for PTA is the lowest followed by PTC, PTB and PTD. This indicates that the PTA can easily penetrate by water even though the MC of the PTA is the lowest.

The contact angles for MMwPF of all panels were analysed and the results are summarized in Table 3.

Table 3. Contact Angle of MMwPF on Different Surfaces of Plywood

Time (s)	CONTACT ANGLE			
	Plywood			
	PTA	PTB	PTC	PTD
10	56.78	50.10	38.25	66.86
20	51.82	54.78	34.27	58.73
30	49.83	53.95	29.10	52.87
40	48.22	39.54	30.24	48.44
50	46.65	48.59	25.71	45.72
60	45.76	45.47	26.33	43.30
70	44.61	42.96	25.83	40.86
80	44.24	39.74	24.24	39.44
90	43.44	47.38	23.73	37.87
100	42.25	55.41	19.95	35.95

From Table 3, it can be seen that the contact angle of MMwPF on PTB (55.41°) is the highest at 100s followed by PTA (42.25°), PTD (35.95°) and PTC (19.95°). Smaller the contact angle, the better penetration of the resin into the adherent. Figure 3 further illustrated comparison between contact angle values for MMwPF.

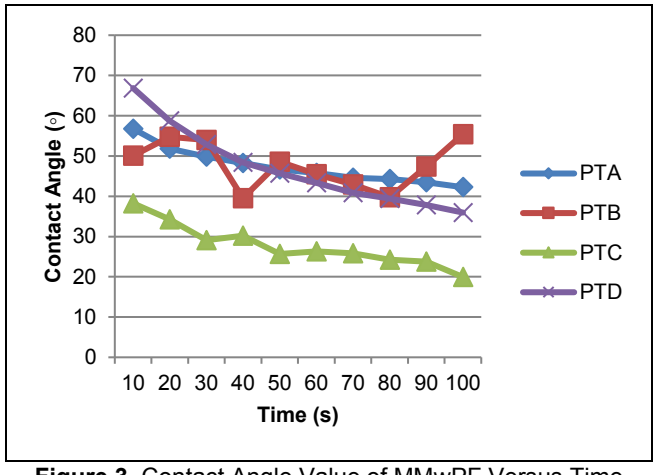


Figure 3. Contact Angle Value of MMwPF Versus Time

Contact angles for PTD, PTA and PTC steadily decreasing towards the end. Meanwhile PTB shows zig-zag movement within 100s. PTA has considerably high contact angle after 100s which shows that the resin is not easily penetrate with the veneer compared to other plywood except PTB as the OPS veneer has already impregnated with resin. As for PTB, PTC and PTD, the resin was dropped on the top veneer surface of the plywood and the veneer was not treated with resin hence the resin can easily penetrate into the veneer. The penetration depends on the type of species.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be drawn as follows:

1. The MC for PTA was found lower than the MC for the commercial plywood (PTB, PTC and PTD) and reflected to the successful of the new drying method.
2. PTB has the highest density followed by PTA, PTC and PTD.

3. All plywood passed the delamination tests which indicate good bonding.
4. PTB exhibited highest contact angle value for PF resins.
5. PTD has highest contact angle value of distilled water.
6. It is recommended that the properties of oil palm plywood should be further examined for durability against white rot fungus and termite test to further confirm the resistance of plywood to the presence of water.
7. The need to study about the water that comes out from drying process using roller presser. It seems to have a lot of starch and protein.

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FLEXURAL BEHAVIOUR OF GLUED LAMINATED TIMBER BEAM FROM VARIOUS SPECIES OF MALAYSIAN TROPICAL HARDWOOD TIMBERS

Norshariza Mohamad Bhkari[1], Wan Hazira Wan Mohamad[2] and Zakiah Ahmad[3]
[1]Faculty of Civil Engineering, Universiti Teknologi MARA, Shah Alam, Selangor, Malaysia.
[2]Faculty of Applied Science, UniversitiTeknologi MARA, Shah Alam, Selangor, Malaysia.
[3]IIESM, Engineering Complex, Universiti Teknologi MARA, Shah Alam, Selangor, Malaysia.

Abstract
This paper presents the flexural strength properties and the integrity of glue lines of glulam manufactured from Malaysian hardwood timber species in various strength grades namely strength group (SG) from SG2 to SG7. The test specimens were manufactured according to MS758:2001 under normal production process. The third-point loading test was conducted on all test glulam beams in accordance with ASTM D198:2013. The integrity of glue line through block shear and delamination test was performed in accordance with MS758:2001. The results show that MOR value was greater in the higher strength group with higher density. However, lower strength grades (SG6 and SG7) presented better ductile characteristic. The MOE value for glulam is not affected by their density for Melagangai and Jelutong. In terms of integrity of glue line, all glulam beams surpassed the requirement set for block shear but not for delamination test. Although higher strength group with higher density (SG2 and SG4) did not met the requirement for delamination, these species failed in timber surface, not in glue under flexural loading.

Keywords: *glulam; glue line integrity; modulus; or rupture; modulus of elasticity; tropical hardwood.*

INTRODUCTION

Modulus of rupture (MOR) and modulus of elasticity (MOE) are the most important flexural properties in a structural application of timber. These strengths are the primary criterion in material selection (Bowyer et al., 2007). Normally, these mechanical properties are measured in flexural members in order to determine the strength and resistance of timber material under static load. MOR and MOE are usually measured and calculated using experimental data, since no theoretical predictive equations exist for most materials (Bodig and Jane, 1982). MOR and MOE for timber materials give different properties depending on the three perpendicular directions or axes, known as orthotropic material. Generally, strength properties such as MOR and MOE of timber parallel to the grain are higher as compared to the strength properties of timber perpendicular to the grain. Therefore, most of glulam pieces are manufactured parallel to its longitudinal axis of the member.

As the glulam beam, strength of timber material can be improved significantly. Numerous researches have been executed on glulam timber focusing on strength properties and its behaviour especially in flexural for building application either in the same or mixed species for softwood and hardwood (Castro and Paganini, 2003; Gary and Annette, 2011; Abu Bakar et al., 2004; Wan Mohamad et al., 2011). Through laminating, strength in glulam can be increased by dispersing timber defects throughout the length of glulam member and the location of different strength grades of timbers within the glulam member cross-sections can be controlled. Generally, the strongest timber with high strength grade is placed in the regions of the greater stress, which located at the top and bottom cross section in the cased of a flexural member.

As in the study by Wan Mohamad et al. (2011) on tropical hardwood in lower strength group (SG4 and SG5), the flexural strength of glulam has been proven to improve into two grades higher from its original strength grade. However, for tropical hardwood in higher strength group (such as SG2 and SG3), the flexural strength is expected at least to be at par with the solid timber for the particular strength grade (Norshariza et al., 2013; Norshariza et al., 2014 and Norshariza et al., 2016).

In determining the structural strength, the integrity of glue lines in glulam timber plays an important role. The appropriate adhesive with the service environment needs to be taken into account in order to sustain life span of the structure. Exteriorly exposed structures have the most demanding bond-quality requirements which include glue line strength, durability and reliability of the structure. It is determined by bond quality testing through block shear and delamination. The factors affecting the quality of adhesive-wood are related to the heterogeneous and anatomical characteristic, permeability, density and moisture content (MC), fibre bonding sites and nature of adhesives (Voichita Bucur, 2011). In order to obtain a high - quality bond during destructive testing, the adhesive strength should exceed the strength of the timber (Schniewind, 1981).

The information on the strength classes of glulam from Malaysian tropical timber, at this moment is still considered unknown as they are not yet published in BS EN 338. The strength data in BS EN 338 are for softwood and European hardwood timber. Therefore, there is a need to determine the strength properties of glulam manufactured from Malaysian tropical timber.

The aim of this study is to determine the flexural strength properties of glulam from selected Malaysian tropical timber. Block shear and delamination testing was conducted to ensure the quality of glue lines in glulam and their relationship with the strength as these properties are dominant for the reliability of the whole structures.

MATERIAL AND METHODS

Materials

Glulam beams were manufactured from six commercially available Malaysia timber species. These timber species are representative of each different strength grades (SG2 to SG7) according to Malaysia timber standard (MS544: Part 2). Each species has two replicate test beams and were produced in accordance with MS758:2001 by a glulam factory under normal and controlled production process. Timber pieces for these beams are parallel to the longitudinal axis and jointed using finger joint. The moisture content of every lamella at the time of gluing is in the range of 8% to 15% as required in MS758:2001. All test beams were untreated and bonded using Phenol-resorcinol formaldehyde (PRF) adhesive which provided the test beams suitable for the exterior used. However, the durability which might contribute to the strength of the test beam under environmental exposure is not considered in this study. Table 1 summarizes all related strength classes, density and dimensions according to Malaysian timber species. Since all depths of the beams are more than 150mm, therefore, no depth adjustment is necessary in this study.

Table 1. Detailed of Test Beams

Timber species	SG (MS 544: Part 2, 2001)	Density (kg/m^3) (MTIB, 2010)	Dimension (mm)
Kekatong (KT)	2	880 - 1155	100 x 200 x 4000
Melagangai (ML)	3	590 - 850	100 x 200 x 4000
Resak (RS)	4	655 - 1155	100 x 200 x 4000
Bintangor (BT)	5	465 - 865	150 x 300 x 6200
Jelutong (JT)	6	415 - 495	150 x 300 x 6200
Sesendok (SK)	7	305 - 655	150 x 300 x 6200

Flexural Test

The third-point loading was set up for the flexural test in accordance with the standard of ASTM D198 (2013), *Standard Test Methods of Static Tests of Lumber in Structural Sizes*. The flexural test was used the IPC 1000kN universal testing machine which including the apparatus of roller support, reaction bearing plates and load bearing blocks. The lateral support allowed the specimen to deflect without significant resistance in order to prevent buckling from happened. The internal and external transducers are used to record the vertical displacement of the beams. The test was performed under a displacement control at a constant rate of 0.05mm/sec. The load cell and transducers were connected to the UCAM60B Kyowa data logger to obtain load and displacement laboratory data. The MOR and MOE values for the test beams are calculated using the equation as provided in ASTM D198 (2013) via these laboratory data.

$$MOR = \frac{P_{max}L}{bh^2} \quad \text{Equation 1}$$

$$MOE = \frac{23PL^3}{108bh^3\Delta} \quad \text{Equation 2}$$

Where, P_{max} is maximum load; P is load below proportional limit; L is span of beam; b is width of beam; h is height of beam; Δ is deflection of beam.

Glue Line Integrity Test

Glue lines integrity tests are required for quality control measurement in glulam manufacturing and taken into a full cross-sectional specimen. The strength and integrity of glue lines can be derived by a block shear and delamination tests according to standard MS758:2001. Samples preparation and test methods for these tests are discussed herein.

Block Shear

The block shear test is conducted in order to measure the shearing strength of the glue line parallel to the grain direction. Shear test was conducted on specimens with square areas taken from the full cross section beam with at least three glue lines tested at each of lower, middle and upper part of the beam. However, for beam composed of less than ten lamellas, all glue lines are required to be tested. Multiple samples must be taken into account in order to get reliable global estimations of member's properties for the effect of irregularities (Tannert et al., 2010). The load was applied to the glue line using the UTM-500kN machine capacity under constant displacement control. The rate is adjusted until the specimen failed

after 20 seconds. In the acceptance criteria, shear strength of each glue line shall be at least 6.0N/mm^2 while 4.0N/mm^2 is acceptable for lighter density timbers if the wood failure percentage is 100% (MS758, 2001). Besides shear strength, the wood failure percentage (WFP) is measured visually to determine the weakened part of glue line either in timber or glue. The required values for the shear strength and WFP either for individual or average specimen are given in the standard, MS758:2001.

Delamination

Delamination can be determined as debonding of glue line and it is probably the most frequently observed damaged in structure (Voichita Bucur, 2011). Test specimens are introduced with a variation of moisture content in order to build up the internal stresses and produced the tensile stresses perpendicular to the glue lines. Consequently, delamination's of the glue lines happen in the inadequate glue-timber bonding. This accelerated ageing test is performed depending on the purposed of the end-product of glulam beam. For glulam beams intended for the exterior use (service class 3), method A with two-cycles impregnating period is required with additional cycle when necessary (MS758, 2001). The test samples with 75mm thickness are cut from the full cross-section of the beam. A minimum of five test specimens is required for each timber species.

According to MS758:2001, the test specimens were placed completely submerged in water in the pressure vessel. The entire end-grain surface must be freely exposed to the water. Test specimens are imposed with vacuum and pressure in sequence about 70kPa to 85kPa and 500kPa to 600kPa, respectively within 65 minutes. The process was then repeated for the second and third cycles when required. For drying process, specimens are taken out from pressure vessel and dried in the oven at the temperature of 60^{0}C to 70^{0}C around 22 hours with required relative humidity and air velocity. Test specimens are immediately measured after drying period. The length of opening in the glue line is measured on the end-grain surface with considering a condition specified in the standard of MS758:2001. The percentage of total delamination and maximum delamination were obtained using Equation 3 and 4 as in the standard (MS758, 2001). These values were compared to the upper limit allowed by standard for tropical hardwood, i.e., 10% for total delamination percentage and 40% for maximum delamination percentage. The similar values are given in European standard (BS EN 14080, 2013).

$$\text{The total delamination percentage} = 100 \times \frac{l_{tot,delam}}{l_{tot,glue\ line}} \qquad \text{Equation 3}$$

$$\text{The maximum delamination percentage} = 100 \times \frac{l_{max,delam}}{2l_{glue\ line}} \qquad \text{Equation 4}$$

Where, $l_{tot,delam}$ is delamination length of all glue lines in the test specimen; $l_{tot,glue\ line}$ is an entire length of glue lines on the two end-grained surfaces or each test specimen; $l_{max,delam}$ is a maximum delamination length of one glue line in the test specimen; and $l_{glue\ line}$ is a length of one glue line.

Moisture Content and Density

Test specimens for moisture content and density were cut from the full cross-section of glulam beam with a thickness of 25mm after the flexural test. The moisture content and density were determined by the oven-dry method accordance to MS837:2006. For density of specimens which not tested at the reference moisture content of 12%, the measured density of specimens are required to be adjusted according to Equation 5 (BS EN384, 2016).

$$\rho = \rho(u)\left(1 - 0.005\left(u - u_{ref}\right)\right) \quad \text{Equation 5}$$

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

RESULTS AND DISCUSSIONS

The strength properties of MOR, MOE, the integrity of glue lines and the fracture characteristics were analysed and discussed in this section. Table 2 shows the summary of the flexural strength and stiffness values as well as density and moisture content of all glulam from different species. Detail discussions on those values are given in the next sections.

Moisture Content and Density

Density has a great on timber properties and structural performance. Thus, the reference density is calculated at the 12% of moisture content, where comparison is made under equilibrium moisture content (EMC) for various strength grades. The values are then summarized in Table 2. From the results, the density was obtained in oven-dry mass and the density at the 12% moisture content has not much difference with a range from 0.01% to 2.73%. The coefficients of variance (COV) values obtained are quite low, indicate as that a low dispersion of mean values for density at reference moisture content.

Table 2. Average Flexural Strength Properties of Glulam Beam from Various Strength Groups (SG)

Test Beam	P_{max} (kN)	δ_{max} (mm)	MOR (N/mm^2)	MOR_{12} (N/mm^2)	MOE (N/mm^2)	MOE_{12} (N/mm^2)	ρ (kg/m^3)	ρ_{12} (kg/m^3)	MC (%)
Kekatong (SG2)	84.18	42.86	75.76 (9.66)[a]	101.79 (14.45)[b]	24527.93 (487.68)[a]	24529.05 (1.99)[b]	1001.66 (6.68)[a]	1019.56 (0.63)[b]	8.43 (0.08)[a]
Melagangai (SG3)	44.90	43.25	40.41 (2.23)[a]	62.28 (2.47)[b]	12859.12 (428.60)[a]	12860.05 (3.33)[b]	798.39 (28.26)[a]	820.79 (3.36)[b]	6.38 (0.37)[a]
Resak (SG4)	89.34	57.80	80.40 (1.74)[a]	109.13 (4.14)[b]	20860.01 (619.96)[a]	20860.98 (2.97)[b]	908.08 (20.68)[a]	924.57 (2.58)[b]	8.38 (0.62)[a]
Bintangor (SG5)	55.74	43.49	33.44 (2.32)[a]	32.70 (7.01)[b]	16509.55 (71.34)[a]	16509.44 (0.43)[b]	711.00 (0.62)[a]	708.77 (0.06)[b]	12.47 (0.06)[a]
Jelutong (SG6)	68.93	72.49	41.36 (0.82)[a]	41.99 (2.11)[b]	13324.36 (2046.22)[a]	13324.41 (15.36)[b]	492.00 (0.80)[a]	492.04 (0.17)[b]	11.73 (0.02)[a]
Sesendok (SG7)	58.15	79.06	34.89 (1.02)[a]	34.89 (2.65)[b]	9869.16 (355.93)[a]	9869.16 (3.61)[b]	489.00 (5.83)[a]	488.14 (1.16)[b]	12.04 (0.06)[a]

Note: [a] is standard deviation and [b] is the COV.

Load-Deflection Behaviour

Figure 1 shows a representative of relationship between load and midspan deflection of glulam beam manufactured from six species until global failure. All six beams of different species of timber show linear load-deflection relationship until it fractures and failed. In the glulam beam, lamination serves as crack arrester in the tension mode. From observation, for all test beams, crack initiated in timber near the tension zone as shown in Figure 2. The crack slowly propagated until it reached weaker zone, which was at the finger joint area. Then, the cracks continued towards the compression zone. Figure 2 presents the failure mode under flexural for one of glulam beam, namely Melagangai.

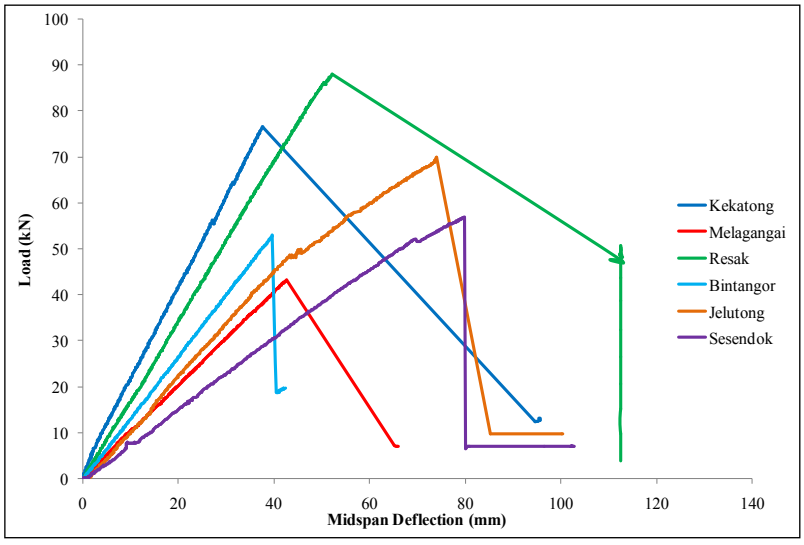


Figure 1. Load versus Deflection of Glulam Beam from Various Strength Groups of Timber



Figure 2. Melagangai beam showing, A: failure started near the tension zone under the point load and the cracks proceeded to the weaker zone which was at finger joint area; B: cracks propagated toward the compression zone and reached at another finger joint at different lamellas; C: failure surfaced at timber not in glue lines.

MOR and MOE

Table 2 gives the summary of the basic properties on strength (MOR) and stiffness (MOE) for all glulam beam based on moisture content at the oven-dry mass and adjusted MOR and MOE at the reference moisture content of 12%. Equation 6 and 7 is a relationship of flexural strength and elasticity modulus for moisture content at 12% (Forest Products Laboratory, 2010).

$$MOR_{12} = MOR + \frac{MOR-16.6}{0.276-MC}(MC - 12) \quad \text{Equation 6}$$

$$MOE_{12} = MOE\,\frac{1857-0.0237MC}{1857-0.0237\times12} \quad \text{Equation 7}$$

The value of the average COV for characteristic flexural strength is within 2.11 to 14.45 depends on timber species and this is considered low as the standard mentioned that the acceptance COV for timber is 20% (BS EN384, 2016). The value of MOR value after adjusted to 12% moisture content is much higher than before adjustment for Kekatong, Melagangai and Resak with increament of 26%, 35% and 26%, respectively. However, there is not much different in the MOR values before and after adjustment for Bintangor, Jelutong and Sesendok. The moisture content for Kekatong, Melagangai and Resak are less than 9% while the moisture content for Bintangor, Jelutong and Sesendok are around 12%. Therefore, the MOR values for Bintangor, Jelutong and Sesendok are the true value.

The MOR values do not follow the order of the strength grade of solid timber, where the order of strength grade of solid is Kekatong > Melagangai > Resak > Bintangor > Jelutong > Sesendok. However, for glulam, the strength in order is Resak > Kekatong > Melagangai > Jelutong > Sesendok > Bintangor. The bending strength of glulam from Sesendok, Bintangor and Jelutong are in the lower range as to other glulam which can be attributed to lower density and high moisture contents, around 12%. Whereas the MOR value of glulam from Kekatong, Melagangai dan Resak are higher where the moisture content is around 8% and has the higher densities. In the case of MOE, the order of the MOE for glulam is Kekatong > Resak > Bintangor > Jelutong > Malagangai > Sesendok. After the MOE has adjusted to 12% moisture content, the value is not much different with the true value.

The Integrity of Glue Lines

The block shear test results for all glulam beams in terms of shear strength and wood failure percentage are tabulated in Table 3.

Table 3. The Shear Strength and Wood Failure Percentage (WFP) of Glulam Beams

Timber Species	Shear Strength		WFP (%)
	Mean (N/mm^2)	COV (%)	
Kekatong (SG2)	12.25(1.90)	15.50	74
Melagangai (SG3)	10.67(2.57)	24.09	92
Resak (SG4)	8.57(1.58)	18.41	70
Bintangor (SG5)	14.90(1.60)	10.90	79
Jelutong (SG6)	8.01(0.51)	6.30	89
Sesendok (SG7)	9.23(1.60)	17.30	76

Note: Value in the bracket/parentheses is standard deviation

From Table 3, it can be seen that the shear strength of Bintangor > Kekatong > Melagangai > Sesendok > Resak > Jelutong. This order of shear strength is not in the same order of MOR and MOE. This shows that the MOR of the glulam may be affected by many factors such as species types, density, moisture content and bonding strength as well as finger joint strength. At this moment, the main factor influencing the MOR and MOE is not evaluated.

The shear strength and wood failure percentage are further explored. Figure 3 shows the relationship between the shear strength and wood failure percentage. The black line represents the requirement to be fulfilled for this glue line integrity where the shear strength at each glue line should be at least 6% with WFP of 74%. Table 3 shows average shear strength for each species range between 14.90 N/mm^2 to 8.01 N/mm^2 and the WFP ranging from 70% to 92%. Based on the results, all glulam beams surpassed the requirement of the MS 758:2001. Cohesive failure was found as the dominant failure surface for all test specimens. The failure occurred in the timber with less appearance of PRF indicate that there is a good adhesion and wetting between the adhesive and timber.

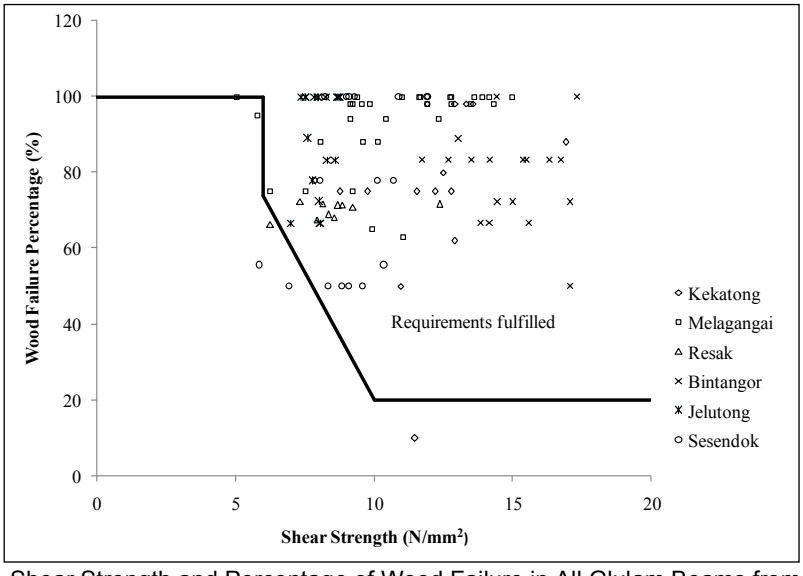


Figure 3. Shear Strength and Percentage of Wood Failure in All Glulam Beams from Different Species.

It is noted by Bourreau et al. (2013), the beam which failed in shearing test also failed in delamination test. However, the beam that failed in delamination test can pass the shearing test. This condition is similar for this study. According to Aicher and Ohnesorge (2011), requirements made in the EN386:2002 for block shear are set too low when using hardwood. The requirement for delamination in this standard is similar with the requirement for delamination established in Malaysian standard, MS 758:2001 which required 10% for total delamination percentage and 40% for maximum delamination percentage.

Figure 4 shows the graph of total delamination per test piece versus maximum delamination per glue line. The black line was drawn to indicate the boundary for the limit set for both parameters that is 10% for total delamination percentage and 40% for maximum delamination percentage. The distribution of the data for all glulam is shown in Figure 4 where

most of higher density timbers (Kekatong and Resak) did not comply with the requirement set by the MS 758:2001. The denser timber makes the glue difficult to be absorbed by the timber. It is evident that by the value of WFP as in Table 3 where Kekatong and Resak have lower percentage of WFP compared to other timber species.

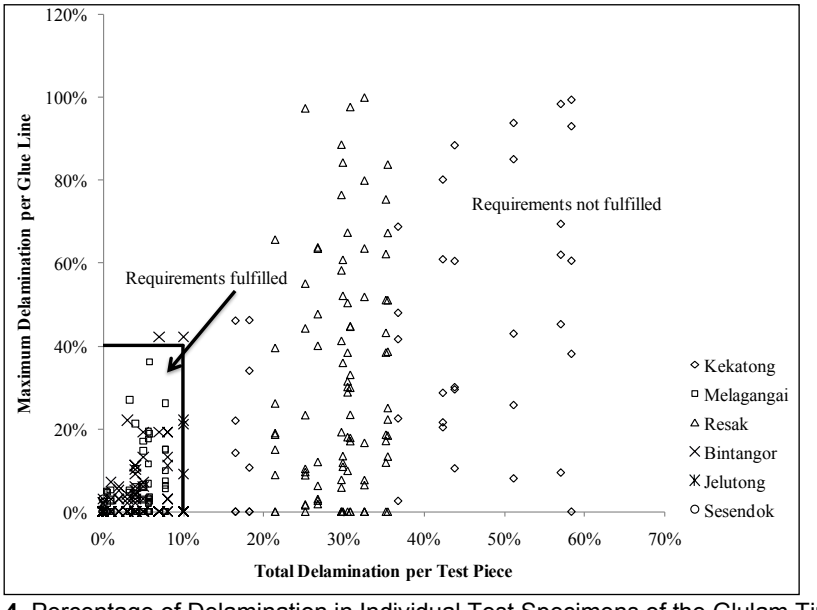


Figure 4. Percentage of Delamination in Individual Test Specimens of the Glulam Timber.

CONCLUSIONS

Based on the above analysis and discussion, it can be concluded that:

1. The flexural strength (MOR) for glulam beams from various strength grades depends on their density and moisture content. The higher strength grade (SG2 to SG4) with higher density provides higher flexural strength.
2. However, the stiffness (MOE) of glulam does not affected by their density for Melagangai and Jelutong. The MOE value of adjusted moisture content is not much different with the true value.
3. The lower strength grade (SG6 and SG7) glulam beam shows more ductile characteristics with greater value in deflection.
4. In terms of glue line integrity, all timber beams from various strength groups surpassed the requirement for shearing test. However, for delamination test, beam with higher strength group and density such as Kekatong and Resak did not comply with the requirement. Since application of glulam beam using Malaysian tropical hardwood timber is still new, more strength and integrity glue line data on glulam beam from various hardwood species is essential.
5. Most of failure mode occurred in timber rather in glue. Although glulam beam in higher density do not meet the bonding requirement under delamination test, from observation, the beam failed in timber surface but not in glue.
6. Manufacturing of glulam from hardwood should be controlled in order to get better bonding, thus the beam strength especially in clamped with appropriate pressure and in cured processes.

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A NEW APPROACH ON LANDSLIDE VULNERABILITY ASSESSMENT AND LANDSLIDE RISK INDEX FOR CRITICAL INFRASTRUCTURES IN MALAYSIA

Yusrin Wahab[1,2], Zuhairi Hamid[1], Ferdaus Ahmad[3], Rohaizi Jusoh[1], Khairolden Ghani[1], Aznah Anuar[2] and Wafiy Ramli[4]

[1]Sustainable Development Department, Construction Research Institute of Malaysia, Kuala Lumpur, Malaysia.

[2]Malaysia Japan International Institute of Technology, Universiti Teknologi Malaysia, Kuala Lumpur, Malaysia.

[3]Mineral and Geoscience Department, Ministry of Water, Land and Natural Resources, Putrajaya, Malaysia.

[4]School of Civil Engineering, Universiti Teknologi Malaysia, Johor Bahru, Malaysia.

Abstract

This paper intended to assess and develop parameters-indicators of landslide vulnerability assessment for critical infrastructures. The quality of landslide risk assessment is very dependent on the input data especially the landslide vulnerability mapping and assessment. The records can be directly merged into the database that permits more accurate vulnerability curve construction for each element-at-risk. The recommended choice of indicators and their weight values are based on a combination of qualitative (expert judgment on previous records) and quantitative approach (specific numerical modelling on the impact of landslides). The experts' judgement recommendation should be prioritized if previous landslide damage record is insufficient. This study proposed vulnerability of a building-residential for translational landslide type which constitutes of four clusters, namely as Susceptibility of CI, Surrounding Environment, Landslide Intensity and People with respective indicators (C, E, I, P), sub-indicators and weightage. The distribution of weightage value for cluster should be in sequence, from 0.1 (low influence to increase vulnerability) to 1.0 (high influence to increase vulnerability). The summation weight value for indicators C, E, I and P must be equal to 1.0. The proposed methodology has been tested in Lembah Bertam, Cameron Highlands and the results shows among all the buildings in this area (358), 117 numbers were categorized as Medium risk while the rests (241 numbers) were identified as High risk.

Keywords: *landslide; hazard; vulnerability; risk; critical infrastructure; element-at-risk.*

INTRODUCTION

Malaysia experienced numerous geohazards throughout the years. Most of the geohazards are associated with the failure of the natural hill slopes. The increasing geohazards in the mountainous and hilly terrain of Malaysia are often associated with soil mass wasting. The failed soil masses are transformed into liquefied debris or mudflow of tremendous velocity and momentum, capable of sweeping away everything found along its path. Understanding the natural geomorphic and geological processes on tropical mountainous terrain are the key to understand the nature and extent of the associated landslides.

The Construction Industry Development Board (CIDB) consortium has taken a significant effort to support and directly address the specific agenda related to SDG 11: Sustainable Cities and Communities and SDG 13: Climate Action by publishing this landslide disaster risk reduction guidelines to holistically adapt to climate changes. Landslide risk assessment prescribed in National Disaster Risk Reduction mentioned that in practice, effective landslide risk mitigation should be implemented at local (individual slope) or regional level. On the local scale, the design of a risk mitigation measure, for example an

early warning system, can be based on a number of reasonable scenarios and may involve the following steps: (UNISDR, 2004):

- Define scenarios that triggering the landslide(s) and evaluate their probability of occurrence.
- Estimate the volume and extent of the landslide and compute the run-out distance for each scenario.
- Estimate the losses of all elements at risk for each scenario.

Landslide disaster risk is a function of landslide hazard, exposure and vulnerability expressed as the probability of the loss of life, injury, and destroyed or damaged physical structures in a given time (De Bono and Mora, 2014). The risk map can be generated by combining three contributing factors i.e., the probability of occurrence of a landslide of a given magnitude (Hazard); the value of element-at-risk (Element-at-risk); and the expected degree of loss resulting from the specified landslide magnitude (Vulnerability) (Vega and Hidalgo, 2016). Vulnerability identifies the relationship between the landslide potential damages to a specific element-at-risk for instance critical infrastructure. It can also be defined as the degree of loss to a given element at risk or set of elements at risk resulting from the occurrence of landslide phenomenon of a given magnitude and expressed on a scale from 0 (no damage to element-at -risk) to 1 (total damage to element-at-risk) (Corominas et al., 2014). Consequently, the determination of landslide vulnerability is the most critical steps towards landslide risk analysis.

REVIEW ON LANDSLIDE VULNERABILITY AND RISK ASSESSMENTS ON CRITICAL INFRASTRUCTURES

Thorough review made on issues regarding landslide vulnerability assessment and model used to define risk index on critical infrastructures for urban, urban highlands, sub-urban and rural areas in Malaysia.

Issues on Scale and Transferability of Vulnerability and Risk Assessment Approach and Results

One of the important issues in vulnerability and risk assessment is on the scales of such analysis (Uzielli et al., 2008). The scales account for time or temporal and space scale of the assessments. The scale of analysis will determine type of approach for vulnerability and risk assessment as well as the amount of data required. For example, the most detailed vulnerability assessment at local level i.e., of individuals or household usually required huge amount of not readily available data. Normally for decision making purposes, vulnerability and risk assessment at regional or national level assessment is adequate. Another related issue is on the downscaling and up-scaling assessment results into different scales that might involve different levels of generalization and assumption and significantly affect the results. The quality and quantity of data will significantly affect the outcome of the vulnerability and risk assessments. Furthermore, vulnerability and risk assessments are site-specific that accounts characteristics of the hazard and element-at-risk (Fuchs et al., 2012; Lo et al., 2012). Different area might require different parameters and approach for susceptibility and hazard mapping depending on the natural and anthropogenic causal factors of a specific hazard.

Issues on the Field Data Collection to Improve Existing Vulnerability Model

Papathoma-Köhle et al. (2015) reported difficulty of recording information regarding the intensity of the process on each building during field data collection to update existing vulnerability model. In their study the intensity of debris flow was not only determined by the deposition height and other factors for example velocity, the viscosity and the direction of the flow that approaches the building. The recording process of these factors is very challenging for the affected buildings (Jakob et al., 2011).

Issues on Input Data for Vulnerability and Risk Assessment

Detailed vulnerability assessment inevitably requires huge amount of data compiled during previous occurrences of landslides. Remondo et al. (2008) has setup an inventory of direct losses due to landslide disaster during nearly 50 years. The inventory was developed based on field surveys and consultations with both local inhabitants and public and private. The inventory was used to develop vulnerability curve, in which the vulnerability was defined as the ratio between loss of the element due to a landslide of a given type and magnitude and value of the element. Careful data collection of present and future landslide damages are also required to update the vulnerability model developed from the previous data (Papathoma-Köhle et al., 2017; Papathoma-Köhle et al., 2015). Advances in remote sensing technologies allow remotely sensed data to be obtained at fine resolution. This has permitted elaboration of more precise hazard models up-to-dated information on infrastructure and socio-economic activity (Bendimerad, 2001; Remondo et al., 2008).

Issues on Uncertainty in Landslide Hazard and Risk Assessment

There are also issues on the landslide vulnerability and risk assessment uncertainties. The quantification of such uncertainties provides important information to stakeholder to support cost-benefit analyses of mitigation measures (Eidsvig et al., 2014). For example, a scenario with low potential losses should not be overlooked if the uncertainties associated with the estimations are large. The risk assessment consists of three fundamental aspects including hazard, elements at risk and their vulnerability. In order to perform a probabilistic risk assessment, the uncertainty in all these three components needs to be assessed (Eidsvig et al., 2014). Uncertainty in vulnerability can be stem from input data, model or procedure and output depending on the stage of vulnerability assessment. Input to a vulnerability model could be qualitative (described with words), semi-quantitative (ranked on a relative scale, also denoted categorical) or quantitative (described as a dimensionless number between 0 and 1).

REVIEW ON METHODS OF VULNERABILITY AND RISK ASSESSMENT

Review on Method of landslide Vulnerability Assessment

One of the most critical steps towards risk analysis is the determination of landslides vulnerability (Uzielli et al., 2008). Vulnerability identifies the element at risk as well as the evaluation of their relationships with the hazard. The relationships relate the landslide potential damages over a specific element at risk. Vulnerability can be defined as the degree of loss to a given element at risk or set of elements at risk resulting from the occurrence of a natural phenomenon of a given magnitude and expressed on a scale from 0 (no damage) to 1

(total damage). Ideally vulnerability assessment accounts various factors including physical, economic, environmental, institutional and human factors. However, from the technical science perspective, vulnerability focuses primarily on the physical aspect which emphasizes on the hazard and its impact (Uzielli et al., 2008).

In the technical perspective, number of methods have been made available for quantitative landslide risk estimation. Previous studies have shown that there is no general or universal approach in vulnerability assessment (Fuchs et al., 2011). However, Papathoma-Köhle et al. (2015) has defined three dominant approaches to express the vulnerability of element-at risk including vulnerability matrices, vulnerability indicators (Birkmann et al., 2013) and vulnerability curves (Totschnig et al., 2011). In another hand, physical vulnerability is generally defined as a scale ranging from 0 (no loss/damage) to 1 (total loss/damage), representing the degree of loss/potential damage of the element at risk.

Uzielli et al. (2008) stated that landslide vulnerability assessment is more difficult compared to other types of hazard like flooding and earthquake due to the complexity and the wide range of landslide process, the lack systematic approaches to express landslide intensity, the quantitative heterogeneity of vulnerability of different elements at risk for qualitatively similar landslide mechanisms due to their intrinsic characteristics, the variability in spatial and temporal vulnerability and lack of historical damage databases due to events which causes extensive damage are recorded and data about the type and extent of damage is often missing.

In qualitative method of vulnerability assessment suitable vulnerability values are given to a specific element-at-risk based on the landslide type (Cardinali et al., 2002; Kappes et al., 2012). The values were assigned by experts based on their experience and historical records of damages based on a specific landslide type. These methods are flexible, for example indicator-based method is easy to use and understand by decision makers. However, this method relies on the expert judgments and there is no direct (quantified) relation between hazard intensities and degree of damage (Uzielli et al., 2008).

The semi-quantitative approach reduces level of generalization in the qualitative method (Dai et al., 2002). The methods are flexible reduce subjectivity, compared with the qualitative method. Based on this method, damage matrices, for example, are composed by classified intensities and stepwise damage levels. Previous study by Frédéric et al. (1996) damage matrices were suggested based on damaging factors and the resistance of the elements at risk to the impact of landslides. The applicability of this method requires statistical analysis of detailed records on landslides and their consequences (Dai et al., 2002). However, this certainly required detailed information on the impact of a specific landslide hazard towards specific element-at-risk.

Due to complexity and highly detailed information required by the quantitative most of time this method was applied at local scale or individual infrastructures (Fuchs et al., 2007; Kaynia et al., 2008; Li et al., 2010; Uzielli et al., 2008). The method usually employed by engineers that involved in the technical decision making where more explicit objective output is required. The results can be directly used in a quantitative risk assessment with detailed analysis on the uncertainty analysis of the vulnerability assessment. The procedures for

physical vulnerability assessment can be made based on the expert judgement (heuristics), damage records (empirical), or statistical analysis (probabilistic).

Vulnerability Matrices

Vulnerability matrices are qualitative method which either being developed alone or most of the time based on the real events and more detailed vulnerability assessment approaches. The input for vulnerability matrices is often either based on empirical data or expert judgment. Intact structural element-at-risk will be assigned with 0 and 1 for totally damage building. All other conditions will be given between these values (Papathoma-Köhle et al., 2017).

Guillard-Gonçalves et al. (2016) used a qualitative assessment of the physical vulnerability of buildings to landslides. The physical vulnerability assessment was based on an inquiry of a pool of European landslide experts and a sub-pool of landslide experts who know the study area. The variability of the answers was assessed using standard deviation of each vulnerability value. In their study structural building types was divided into 4 groups namely;

i. wood or metal (SBT1);
ii. adobe / rammed earth / loose stone walls (SBT2);
iii. brick / stone masonry walls (SBT3);
iv. masonry walls confined with reinforced concrete (SBT4).

These building structural types were crossed with the landslide intensity (i.e., depth or slip surface and height of accumulated materials) as shown in Figure 1.

		Landslide body: depth of slip surface									
		1 m		3 m		5 m		10 m		20 m	
		Avg. vuln.	SD	Avg. vuln.	SD	Avg. vuln.	SD	Avg. vuln.	SD	Avg. vuln.	SD
Pool of European experts (52)	SBT1	0.60	0.24	0.73	0.21	0.84	0.18	0.90	0.19	0.90	0.20
	SBT2	0.57	0.23	0.72	0.20	0.85	0.17	0.92	0.14	0.91	0.17
	SBT3	0.46	0.22	0.60	0.22	0.76	0.18	0.88	0.18	0.91	0.18
	SBT4	0.35	0.20	0.48	0.18	0.66	0.19	0.80	0.18	0.86	0.19
Sub-pool of study area experts (14)	SBT1	0.64	0.19	0.84	0.14	0.96	0.09	1.00	0.00	1.00	0.00
	SBT2	0.59	0.15	0.77	0.15	0.96	0.09	1.00	0.00	1.00	0.00
	SBT3	0.43	0.15	0.66	0.15	0.86	0.12	0.99	0.05	1.00	0.00
	SBT4	0.30	0.10	0.50	0.13	0.71	0.15	0.91	0.13	0.99	0.05

		Landslide foot: height of accumulated material							
		0.5 m		1 m		3 m		5 m	
		Avg. vuln.	SD	Avg. vuln.	SD	Avg. vuln.	SD	Avg. vuln.	SD
Pool of European experts (52)	SBT1	0.45	0.22	0.61	0.20	0.85	0.17	0.94	0.12
	SBT2	0.38	0.23	0.53	0.21	0.78	0.18	0.93	0.12
	SBT3	0.30	0.18	0.40	0.22	0.66	0.17	0.83	0.17

Figure 1. Vulnerability Matrices for Building Structural Types Crossed with The Landslide Intensity (i.e., Depth or Slip Surface and Height of Accumulated Materials) (Guillard-Goncalves et al., 2016)

Zêzere et al. (2008) defined the physical vulnerability of buildings and road not only on structural properties of exposed elements, but also on the type of process, and its magnitude (see Figure 2). The vulnerability values were defined based on the past records of landslides occurrences in the study area. Landslides affect the physical element-at-risk at various impact mechanisms for example burial, collision impact, earth pressures, differential shearing in tension, compression or torque, plastic deformation (flow), by object displacement and by

removal or deformation of valued ground, such as productive soil and foundation substrate. The degree to which these mechanisms are manifest is generally reflected by the type of landslide (Glade, 2004). However, many landslides exhibit complex behaviour, and a variety of different impact mechanisms may be represented in the one landslide type.

Elements at risk value and vulnerability considering exposure to different landslide types within the Fanhões-Trancão test site

Vulnerable elements		Value €		Vulnerability		
		m^2	Pixel	Shallow translational slides	Translational slides	Rotational slides
Buildings	Poor traditional masonry buildings	600	15,000	0.5	1	1
	Poor adobe stone or taipa buildings	600	15,000	0.5	1	1
	Poor other resistant elements (wood, metallic) buildings	600	15,000	0.4	1	1
	Usual traditional masonry buildings	1197	29,925	0.5	1	1
	Usual reinforced concrete buildings	1197	29,925	0.3	1	1
	Luxurious reinforced concrete buildings	2186	54,650	0.3	1	1
	Heritage traditional masonry	2217	55,425	0.5	1	1
Roads	Motorway, bridges, viaducts	933	23,325	0.6	1	1
	National road	933	23,325	0.6	1	1
	County road	8	200	0.6	1	1
	Rural road	2.5	62.5	0.6	1	1

Figure 2. Vulnerability Matrices for Building and Road Types Crossed with The Landslide Types Namely, Shallow Translational Slides and Rotational Slides (Zerere et al., 2008)

Frédéric et al. (1996) proposed a vulnerability matric that relates different types of element-a-risk with different intensities of landslides. Figure 3 shows a sample of the structural damage matric for different categories of buildings, which damages were explained in Figure 4.

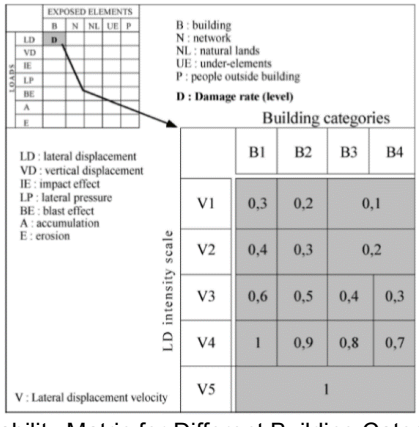


Figure 3. Sample of Vulnerability Matric for Different Building Categories (Frédéric et al., 1996)

I	Light damage (on furniture only). Stability not affected.	0,01-0,1
II	Cracks on walls, but stability not affected. Repairs not urgent.	0,2-0,3
III	Important deformations, cracks widely open. Cracks on structures. Stability affected. Doors and windows unusable. Necessity of evacuation.	0,4-0,6
IV	Breaks on structures. Separation of parts. Partial collapse of floor and breaks on walls. Necessity of evacuation. Rehabilitation compromised.	0,7-0,8
V	Total collapse. Area must be evacuated. No possibility of rehabilitation.	0,9-1

Figure 4. Sample of Vulnerability Matric for Different Building Categories (Frédéric et al., 1996)

Vulnerability Curves

Vulnerability function or also known as vulnerability curves are commonly used to express physical vulnerability in a quantitative way (Papathoma-Köhle et al., 2017). Vulnerability curves can be defined as "a continuous curve associating the intensity of the hazard (X-axis) to the damage response of a building (Y-axis)" (Tarbotton et al., 2015). The function defines the degree of loss generally between 0 and 1 and it requires significant amount of empirical data to be reliable. In this case, the information on magnitude of loss and intensity of landslide hazard is required for different categories of element-at-risk. The intensity of the hazard required detailed information of hazard characteristics that could potentially cause damages to the element-at-risk for example for debris flow, intensity can be represented by debris height, velocity or viscosity. The damage records for element-at-risk can be obtained directly either from the actual damage cost or compensation cost or indirectly from photographic documentation or earth observation data that may be translated later into monetary costs (Papathoma-Köhle et al., 2012).

Remondo et al. (2008) developed a landslide vulnerability from a detailed analysis of past damage over 50 years from mass movements. The inventory of landside damages was based on field surveys and consultations with both local inhabitants and public and private. The vulnerability values were determined from a detailed analysis of past damage from mass movements, which has been recorded over nearly 50 years. The inventory is based on field surveys and consultations with both local inhabitants and public and private. However, the data is still not complete and extrapolations and theoretical assumptions were needed for the assessment. The vulnerability of infrastructure was defined as the ratio between Loss of the element due to a landslide of a given type and magnitude and value of the element. In general, the vulnerability values could theoretically be greater than 1, since repair could cost more than construction of a new structure. However, the maximum value considered in this analysis is 1 (total loss). In this study, the values of past losses were corrected for inflation, an assumption which can also be used for predicted future losses. They have only considered shallow slide for the risk estimation. The risk was estimated using Equation 1.

$$R = H \times E \times V, \tag{1}$$

Where R is the risk, E is the value for each element-at-risk and V is the vulnerability. They have pointed that a complete view of the risk for a given element-at-risk should combine multiple scenarios of hazards and vulnerability values for the different types of mass movements and their magnitude.

Vulnerability Indicators

Based on the study of social vulnerability by Birkmann (2006) vulnerability indicators can be defined as "variables which are operational representations of a characteristic or quality of the system able to provide information regarding the susceptibility, coping capacity and resilience of a system to an impact of an albeit ill-defined event linked to a hazard of a natural origin". Vulnerability indicator for landslide hazard includes the selection of relevant indicators, the identification of variables, their weighting and, finally, their aggregation in a vulnerability index (Papathoma-Köhle et al., 2017).

Isaza-Restrepo et al. (2016) estimated physical vulnerability of building as in Equation 2 This is an example of an equation:

$$VE = V_E \times A_E \times D_S, \tag{2}$$

where VE is the structural vulnerability (given in likely total economic losses), VE is the structural vulnerability index, AE is the area of the structure or of the built area and D$ is the commercial cost of the structure per square meter. The structural vulnerability was defined based on the type of structure, state of maintenance and age (Equation 3).

$$VE = BSF \times W_{BSF}, \tag{3}$$

where BSF is the building susceptibility factor and WBSF is the weight of each factor.

Uzielli et al. (2008) proposed a vulnerability assessment framework quantitative estimation of physical vulnerability of element-at-risk for a specific landslide hazard. In their study, vulnerability was defined as the product of landslide intensity and susceptibility of element-at-risk (Equation 4). Both intensity and susceptibility (Equation 5) are expressed in dimensionless terms with values between 0 and 1.

$$V = I \times S, \tag{4}$$

$$S = 1 - \prod_{i=1}^{n_s}(1 - \varepsilon_i) \tag{5}$$

Where ε_i is the i-th of ns susceptibility factors (each defined in the range [0, 1]) contributing to the category susceptibility. The susceptibility of the element-at-risk refers to the lack of inherent capacity of the elements in the spatial extension under investigation to preserve their physical integrity and functionality in the course of the physical interaction with a generic sliding mass. They have stressed that susceptibility as defined in the study is independent of the characteristics of the acting agent like the landslide. The susceptibility value accounts its physical resilience to landslide occurrence, which mainly depends on the technological and its state of maintenance. The susceptibility classes for structural typology (ξSTY) were divided into 6 classes as listed in Table 1.

Table 1. Proposed Values of Susceptibility Factor for Structural Typology

Structural Typology	Resistance	Susceptibility
Lightest, simple structures	None	1.00
Light structures	Very low	0.90
Rock masonry, concrete, and timber	Low	0.70
Brick masonry, concrete structures	Medium	0.50
Reinforced concrete structures	High	0.30
Reinforced structures	Very high	0.10

The state of maintenance (ξSMN) expresses the reduced capacity of structures in comparison with the "very good" category in which maximum capacity is expected. Table 2 shows the proposed values for the state of maintenance.

Table 2. Proposed Values of Susceptibility Factor for Structural Typology

State of maintenance	ξ_{SMN}
Very poor	0.50
Poor	0.40
Medium	0.25
Good	0.10
Very good	0.00

The landslide intensity parameter can be defined quantitatively using various approaches. Landslide intensity has been addressed and defined quantitatively using a variety of parameters i.e., maximum velocity, total displacement, differential displacement (relative to points adjacent to the point under consideration), depth of the moving mass, depth of deposits after the movement ceases, depth of erosion, unit discharge, kinetic energy per unit area, maximum thrust, impact pressure, maximum normal or shear strain at or below ground surface (Hungr, 1997). Uzielli et al. (2008) in their study used a composite landslide intensity parameter that accounts kinetic and kinematic characteristics of the interaction between the sliding mass and the reference area (Equation 6).

$$I = k_S + I_K + r_M + I_M \tag{6}$$

$$k_S = A_i / A_t \tag{7}$$

k_S is the spatial impact ratio, A_i is the area pertaining to the element-at-risk that is affected by the landslide, A_t is the total area pertaining to the element-at-risk, r_K is the kinetic relevance factor of the category, r_M is the kinematic relevance factor of the category, I_K is the kinetic intensity parameter of the element-at-risk and I_M is the kinematic intensity parameter of the element-at-risk.

Li et al. (2010) proposed a new quantitative model for vulnerability of structures and persons based on landslide intensity and resistance of exposed elements. In their study, vulnerability (V) was defined as a function of the hazard intensity (I), the exposed elements at risk and the resistance ability (R) of the elements to withstand the hazard.

$$V = f(I, R) = 2 (I^2 / R^2) \text{ for } I / R \leq 0.5, \tag{8}$$

$$V = f(I, R) = 1.0 - 2 ((R-I^2)) / R^2 \text{ for } I / R \leq 1.0 \tag{9}$$

$$V = f(I, R) = 1.0 \text{ for } I / R > 1. \tag{10}$$

In their study, the landslide intensity is defined as a function of dynamic and geometric intensity factors.

$$I = I_{dyn} + I_{dpt,} \text{ structure outside landslide area} \tag{11}$$

$$I = I_{dyn} + I_{dfm,} \text{ structure inside landslide area} \tag{12}$$

where I_{dyn} is the dynamic intensity factor that accounts stationary and non-stationary element-at-risk, I_{dpt} is the debris-depth factor, which was used to evaluate the element-at-risk outside

rapid landslide area, and I_{dfm} is the deformation factor, which is mainly used to evaluate the structures within the sliding mass of a slow landslide. The resistance reflects the physical resilience of vulnerable element-at-risk to withstand specific landslide hazard intensity. The resistance of structure was defined as follows:

$$R_{str} = (\varepsilon_{sfd} \times \varepsilon_{sty} \times \varepsilon_{smn} \times \varepsilon_{sht})^{1/4}, \quad (13)$$

Papathoma-Köhle (2016) highlighted that it is important to account information on intensity of the process in the indicator-based method (IBM) and indicators that reflect the physical resilience of the buildings (Figure 5). In this study, the relative vulnerability index (RVI) is calculated using Equation 14. The same vulnerability index was also used by Kappes et al. (2012) in their works for vulnerability analysis.

$$RVI = \sum_1^m w_m \times I_m \times S_n \quad (14)$$

where w represents the m different weights, I the m indicators and s the n scores of the indicators.

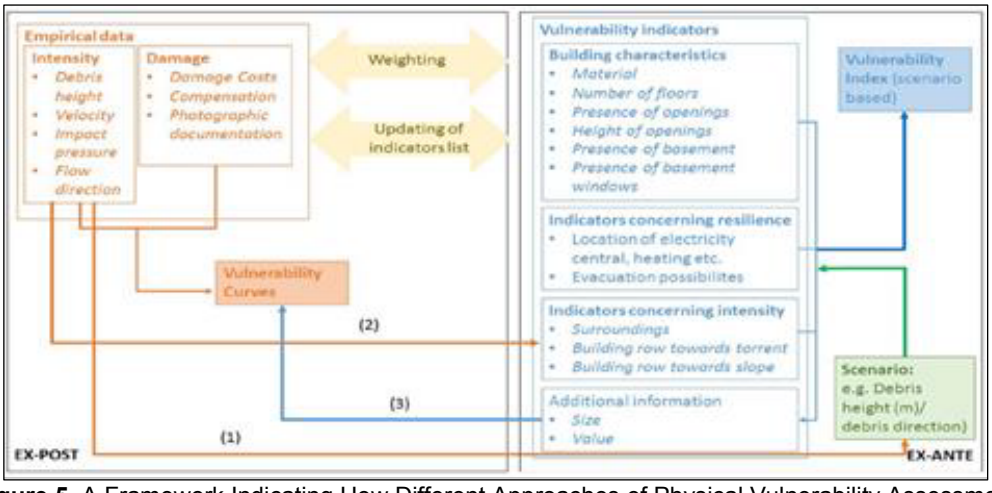


Figure 5. A Framework Indicating How Different Approaches of Physical Vulnerability Assessment May Interact With Each Other (Papathoma-Köhle, 2016)

The methods mentioned above are focusing on the physical vulnerability assessment of the critical infrastructure. However, for other types of vulnerability assessment e.g., social welfare, economics and ecology aspect require different methods and indicators. For example, vulnerability to natural hazards from the perspective of social science has been defined by several researchers e.g., Blaikie et al. (1994) provided the following definition: "…the characteristics of a person or group in terms of their capacity to anticipate, cope with, resist and recover from the impacts of natural or man-made hazards". The importance of indicators for social vulnerability was also stressed as one of the key activities proposed in the Hyogo Framework. The framework suggested that the development of "systems of indicators of disaster risk and vulnerability at national and sub-national scales that will enable decision makers to assess the impact of disasters on social, economic and environmental conditions and disseminate the results to decisionmakers, the public and populations at risk" (UN, 2007).

Cutter et al. (2003) highlighted factors that influenced social vulnerability which includes lack of access to resources (including information, knowledge, and technology), limited access to political power and representation, social capital, belief and customs, building stock and age, frail and physically limited individuals, type and density of infrastructure and lifelines.

Review on Method of Landslide Risk Assessment

Final map of landslide risk shall technically present the subdivision of the terrain into zones that are characterised by different probabilities of losses that might occur due to landslides of a given type within a given period of time. Two common methods are available, either in the form of qualitative or quantitative manner. Qualitative risk analysis refers to an analysis that uses word form, descriptive or numerical scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur, whereas quantitative risk analysis is based on numerical values of the probability, vulnerability and consequences, resulting in a numerical value of risk (IUGS, 1997; UN-ISDR, 2004; Fell et al., 2008a). Suitable approach for landslide risk analysis will be determined based on requirement by the authorities for example scale of mapping and essential information that should be considered by the risk index.

Loss can be grouped into direct and indirect losses which both can be further separated into human, physical, economic and cultural environment (Winter et al., 2016). The loss assessments required different level of information depending on the level of information obtained for element-at-risk.

Previous studies have shown that quantitative risk (R) can be defined conceptually based on Equation 1 (Vega and Hidalgo, 2016). Hence the risk map can be generated by combining three contributing factors i.e., 1) the probability of occurrence of a landslide of a given magnitude (Hazard); 2) the valued attributes at risk (Value); and 3) the expected degree of loss resulting from the specified landslide magnitude (Vulnerability).

The qualitative risk assessment is based on the classes of relative risk for example the risk matrix that can be generated by combining the relative probability of landslide occurrence and relative classes of loss (Dai et al., 2002).

Table 3. Examples of Matrix Selection for Landslide Mitigation Categories

Degree of Hazard	Degree of Vulnerability					Categories
	Very Low	Low	Moderate	High	Very High	
Very Low	I	I	II	III	III	
Low	I	II	II	III	III	
Moderate	II	II	III	III	IV	
High	III	III	III	IV	IV	
Very High	III	III	IV	IV	V	

(Source: Rodeano, 2015; 2018)

Table 3 shows an example of how the risk index is determined using qualitative approach of risk assessment. Once determined, the selection of mitigation method is conducted by referring to Table 4. Examples of options for mitigation of risks for a slope or group of slopes would include in reducing the frequency of landslide and reduce the probability of landslide reaching element at risk.

Table 4. Examples of Landslide Mitigation Method

Categories	Mitigation Method	Structural Cost	Non-Structural Cost
I	Accept and modify the risk	High	Moderate
II	Modify and reduce the risk	Very High	High
III	Reduce the risk, risk monitor and postpone the results	Very High	Very High
IV	Risk monitor, postpone the results and ignore the risk	High	Moderate
V	Postpone the results, ignore the risk and risk transfer	Moderate	Moderate

(Source: Rodeano, 2015; 2018)

Figure 6 highlighted the information or input data for landslide risk assessment. Notice that the top part of the figure specifies this input data which are environmental factors, landslide inventory (or occurrences), triggering factors and elements at risk (Hazarika, 2016). Each input data corresponds to the identical component of Equation (3) where the environmental and triggering factors, plus landslide inventory subsequently will produce hazard maps or H, the element at risk is used for vulnerability assessment or V and also the value of E. On top of listing corresponding data for landslide risk assessment, the bottom of Figure 7 also shows the specific type of quantitative and qualitative risk assessment which are risk curve, event-tree, risk matrix and indicator approach.

Suitable approach for landslide risk method will be determined based on requirement by the authorities for example scale of mapping and essential information that should be considered by the risk index. Practically, the remaining time scale of the project will also greatly influence the selection of landslide risk methodology. In addition, the risk index is proposed for each element-at-risk and detailed explanation on the losses is explained clearly. As for quantitative approach, the availability of complete set of data ranks high as a criteria selection.

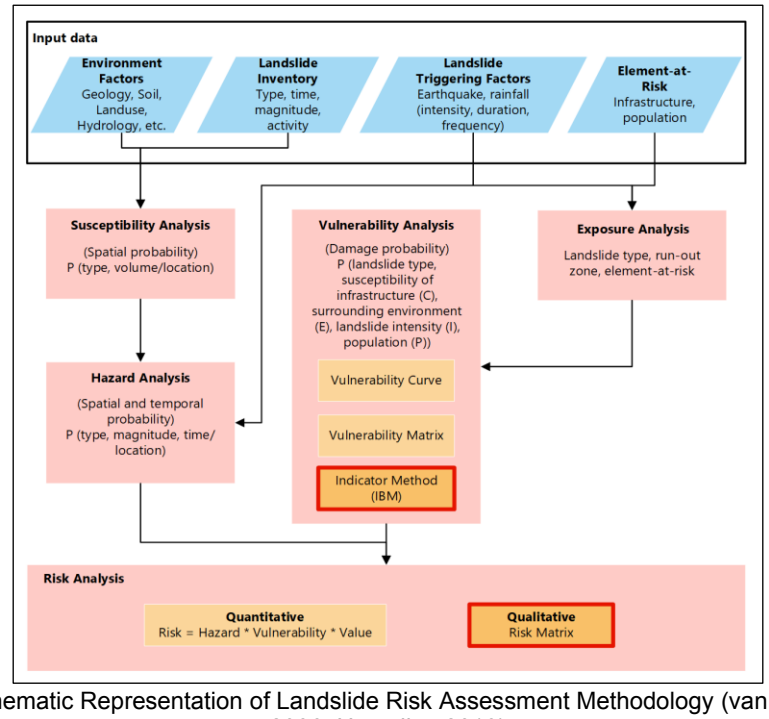


Figure 6. Schematic Representation of Landslide Risk Assessment Methodology (van Westen et al., 2006; Hazarika, 2016)

SUGGESTED METHODS OF LANDSLIDE ASSESSMENT FOR CRITICAL INFRASTRUCTURE

Vulnerability Assessment

Based on previous study, the proposed landslide vulnerability assessment is based on the semi-quantitative approach indicator-based vulnerability assessment (IBM) introduced by Papathoma-Köhle (2016) and and Uzielli et al. (2008). The previous method however did not clearly consider the intensity of landslide hazard in the IBM method. Therefore, the hazard intensity will be clearly integrated as part of the indicators in the IBM as it was strongly suggested by Papathoma-Köhle (2016). Uzielli et al. (2008) on the other hand considered the susceptibility of people inside the building but did not consider surrounding environment or mitigation measure in the vulnerability estimation. In the proposed method the vulnerability indicators are grouped into 4 i.e., susceptibility of critical infrastructure (C), effect of surrounding environment or mitigation measures (E), susceptibility of people inside the residential building (P) and intensity of landslide hazard (I) (Equation 15).

$$V = f(C, E, P, I), \tag{15}$$

Suitable parameters and indicators for landslide vulnerability will be selected based on intensive literature review, expert knowledge and past records of landslide occurrences and related damages in Malaysia. Suitable indicators will be selected to represent landslide intensity for different type of landslide and susceptibility of the critical infrastructures that is basically relied on the characteristics of the structures. Suitable indicators will be critically defined, listed and prioritized from literature reviews, damage records and series of workshop with the experts. The selected indicators will be used in the next stage in proposing suitable vulnerability assessment method for Malaysia. Figure 4 show the complete process for landslide vulnerability indicators selection.

Parameters for landslide risk assessment will be mainly defined by the method of risk assessment and the critical infrastructure. The quantitative risk assessment method requires detailed information on the spatial and temporal probability of landslide occurrence (i.e., landslide hazard), probability of losses for a particular critical infrastructure (i.e., landslide vulnerability) and monetary value for each critical infrastructure. Meanwhile the qualitative approach of landslide risk assessment generally requires combination of different degrees of landslide hazard and vulnerability in defining level of risk for each critical infrastructure. Selection of parameters and indicators for landslide risk assessment will be also based on literature reviews, damage records and series of workshop with the experts (Ibrahim and Esa, 2018). Suitable indicators will be thoroughly defined, listed and prioritized for the next stage of proposing suitable landside risk assessment method for Malaysia. Therefore, the vulnerability index for CI (V) is defined as in Equation 16.

$$V = \sum_{i=1}^{m} \quad \sum_{j=1}^{n} w_i \times S_j \tag{16}$$

where w_i is the i-th weight of m different indicator groups (in this case m = 4 for residential and m = 3 for other CI, i.e., S, E and I) and S_j is j-th score for n indicators under each group.

The weight for each group ranges from 0.1 (low influence to increase vulnerability) to 1.0 (high influence to increase vulnerability). The total weight value must be equal to 1.0. The score value for each indicator under specific group also ranges from 0.1 (low influence to increase vulnerability) to 1.0 (high influence to increase vulnerability). Figure 7 shows the overall concept of the proposed vulnerability estimation method.

The building susceptibility indicator group indicates the susceptibility of the critical infrastructure towards specific landslide hazard intensity that accounts the physical characteristics. The surrounding environment or mitigation measures group will take into account the role of existing mitigation measures and surrounding elements in reducing or increasing the impact of landslide on the CI. The intensity of landslide hazard is the final indicator group will indicate the intensity of landslide hazard on a specific CI. The final vulnerability value is a weighted and aggregated value explained by Equation 17. In this method, the weighting is not static and varies depending on the needs of the users. The method allows the user to set their own priorities and change the weighting accordingly. Kappes et al. (2012) suggest that the main advantage of the model is the flexibility. In addition, they also consider that the method is not hazard-intensity specific to be an advantage, in which the method can still be used in absence of the intensity or the process characteristics.

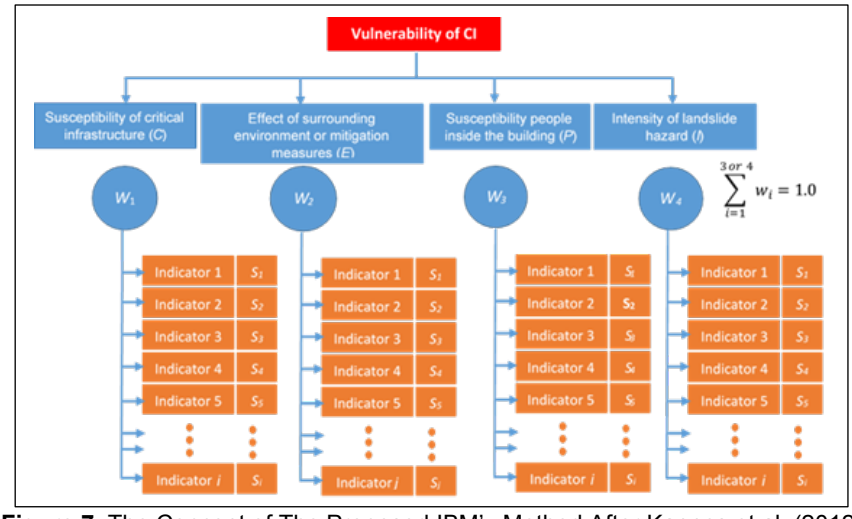


Figure 7. The Concept of The Proposed IBM's Method After Kappes et al. (2012)

Risk Assessment

Unlike the suggested semi-quantitative method for landslide vulnerability assessment, it is proposed that a qualitative method for landslide risk assessment via risk assessment matrix is used in this project (Anbalagan and Singh, 1996). Primarily, this suggestion is made given that the majority of literatures and practice for landslide risk assessment is based on a qualitative method, therefore the method is proven to be effective and convenient (Glade, 2004). For this to happen, Equation (1) is adjusted to only accommodate two elements of qualitative risk assessment matrix which are hazard and vulnerability, please refer to Equation (17) below.

$$R = H \times V, \tag{17}$$

In Equation (18), R = risk, H = hazard and V = vulnerability. To execute this, the challenge is to define the scale of the risk index, however, much earlier, the qualitative measurement of hazard and vulnerability needs to be established. That said, either to straightaway use the already existing qualitative hazard and vulnerability measurement or to modify the quantitative measurement to qualitative. It is proposed that the measurement used is adopted from the Australian Geomechanic Society's Qualitative Measures of Likelihood of Landsliding (Hazard Measurement) and Qualitative Measures of Consequences to Property (Vulnerability Measurement) as shown in Table 6 and Table 7 (Fell, 2000).

Table 6. Qualitative Measures of Likelihood of Landsliding (Hazard Measurement)

Level	Descriptor	Description
A	Almost certain	The event is expected to occur
B	Likely	The event will probably occur under adverse condition
C	Possible	The event could occur under adverse condition
D	Unlikely	The event could occur under very adverse condition
E	Rare	The event is conceivable but only under exceptional circumstances
F	Not credible	The event is inconceivable or fanciful

(Source: Fell, 2000)

Table 7. Qualitative Measures of Consequences to Property (Vulnerability Measurement)

Level	Descriptor	Description
1	Catastrophic	Structure completely destroyed or large-scale damage requiring major engineering works for stabilisation
2	Major	Extensive damage to most of structure, or extending beyond site boundaries requiring significant stabilisation works
3	Medium	Moderate damage to some of structure, or significant part of site requiring large stabilisation works
4	Minor	Limited damage to part of structure, or part of site requiring some reinstatement/stabilisation works
5	Insignificant	Little damage

(Source: Fell, 2000)

The combination of Table 6 and 7 translates into Table 8 which is international standard risk assessment matrix (Ko Ko et al., 1999). Combining likelihood with consequence results in a risk assessment matrix divided into 5 classes of risk index from very low risk (VL) to very high risk (VH). Although the risk index seems straightforward, the measurement of hazard and vulnerability is ideally done by experts to avoid spurious outcomes and for it to be value-adding (Fell, 2000).

Table 8. International Standard Risk Assessment Matrix (Ko Ko et al., 1999)

Likelihood (hazard)	Consequences to property (Vulnerability)				
	Catastrophic	Major	Medium	Minor	Insignificant
Almost certain	Very High Risk	Very High Risk	High Risk	High Risk	Moderate Risk
Likely	Very High Risk	High Risk	High Risk	Moderate Risk	Low Risk – Moderate Risk
Possible	High Risk	High Risk	Moderate Risk	Low Risk – Moderate Risk	Very Low – Low Risk
Unlikely	Moderate Risk – High Risk	Moderate Risk	Low Risk – Moderate Risk	Very Low Risk – Low Risk	Very Low Risk
Rare	Moderate Risk – Low Risk	Low Risk – Moderate Risk	Very Low Risk – Low Risk	Very Low Risk	Very Low Risk
Not credible	Very Low Risk	Very Low Risk	Very Low Risk	Very Low Risk	Very Low Risk

(Source: Ko Ko et al., 1999)

CASE STUDY IN CAMERON HIGHLANDS

The proposed method has been tested by case study in Lembah Bertam, Cameron Highlands, Pahang. The overall landslide risk analysis framework with the landslide vulnerability assessment method and processes are as shown in Figure 8. The generation of CI maps using digital image processing approach is based on the supervised image classification process. The classified remote sensing data is in raster format and should be converted into vector format to be used in the next data processing stage. The landslide exposure analysis involves the process of identifying the exposed CI within the landslide and run-out zones. The CI map is overlaid with the landslide inventory map and each CI is marked based on its location either within the landslide and runout zones or outside both zones. While landslide vulnerability cluster maps (C, E, I and P) is generated depending on the type of CI obtained from CI maps and landslide types obtained from landslide inventory map.

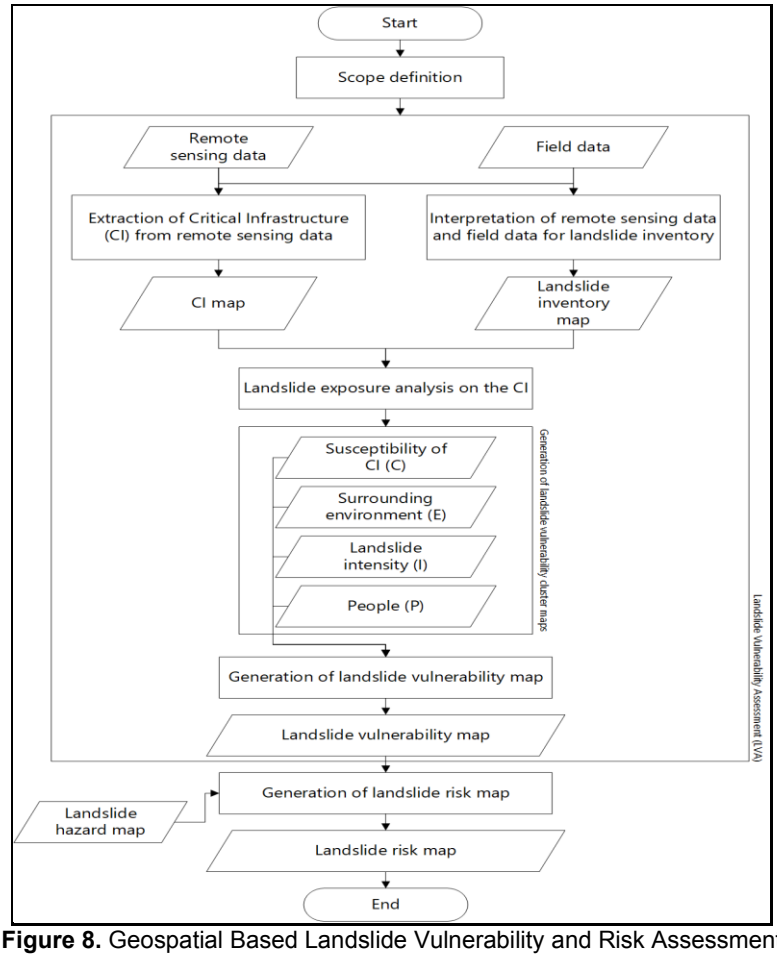


Figure 8. Geospatial Based Landslide Vulnerability and Risk Assessment

Suitable weight value for indicators and sub-indicators for each landslide cluster is determined and stored in each polygon of CI in the CI map. The landslide vulnerability map for each CI is generated by combining all cluster maps, C, E, I and P using Equation 15. The resulting landslide vulnerability index is classified into its specific vulnerability class for each CI as shown in Table 8. The landslide risk map for each CI is generated by combining the

hazard map and vulnerability map by using Equation (17). The resulting landslide risk index is classified into 5 classes from very low (VL) to very high (VH) risk using risk assessment matrix as in Table. The resulted combination of all cluster maps is shown in Figure 9, the landslide vulnerability map for the location.

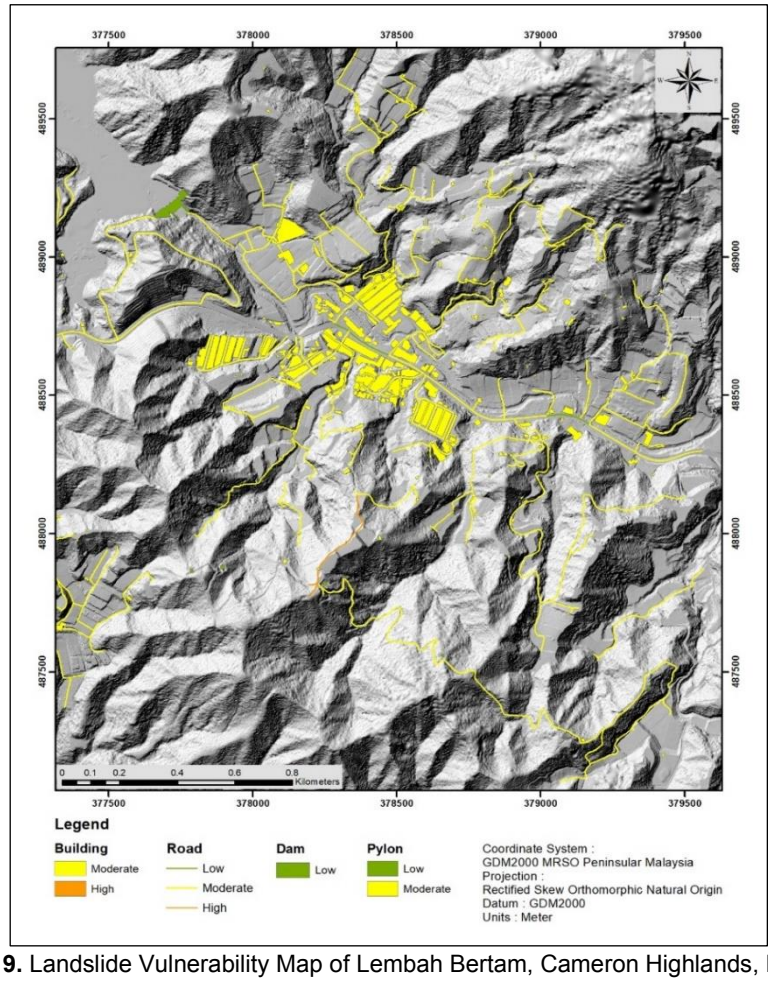


Figure 9. Landslide Vulnerability Map of Lembah Bertam, Cameron Highlands, Pahang.

The landslide risk map is produced by the combination of landslide hazard map and landslide vulnerability map. A geospatial raster processing method of combining both maps called 'raster calculator' has the specific ability to merge two or more raster layers to come out with a single output raster layer. The final landslide risk map of the same area for the respective CI is shown in Figure 10. Similarly, as the vulnerability map, the landslide risk map has only five classifications from very low until very high.

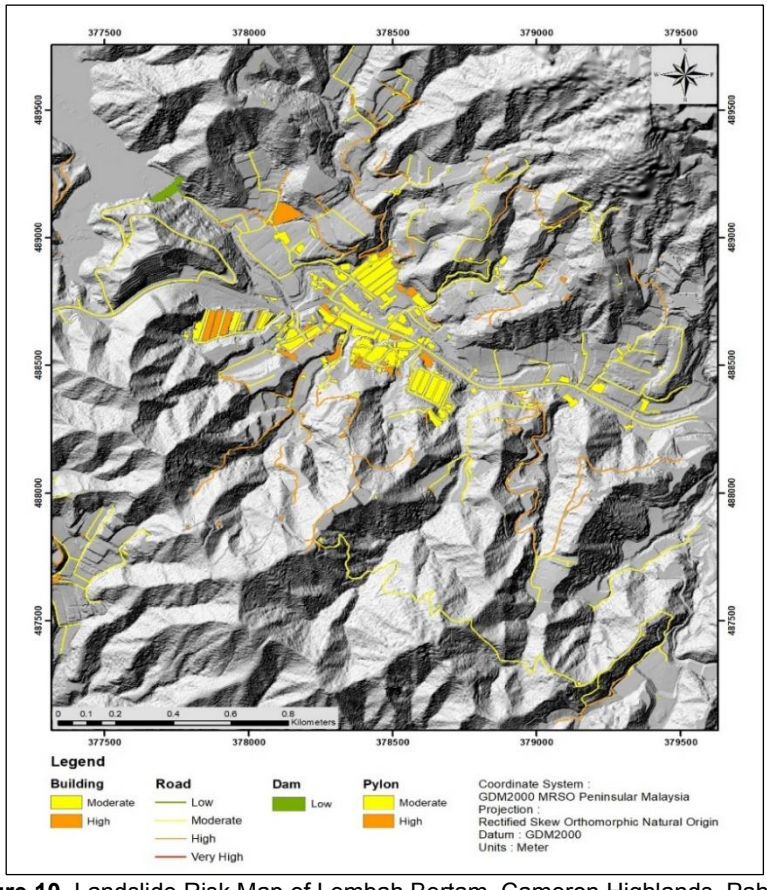


Figure 10. Landslide Risk Map of Lembah Bertam, Cameron Highlands, Pahang.

From the recommended procedure, Figure 11 shows an example of the overall process of landslide vulnerability assessment of a particular CI (building) presumably subjected to translational or rotational landslide hazard in deriving to the vulnerability index of the CI.

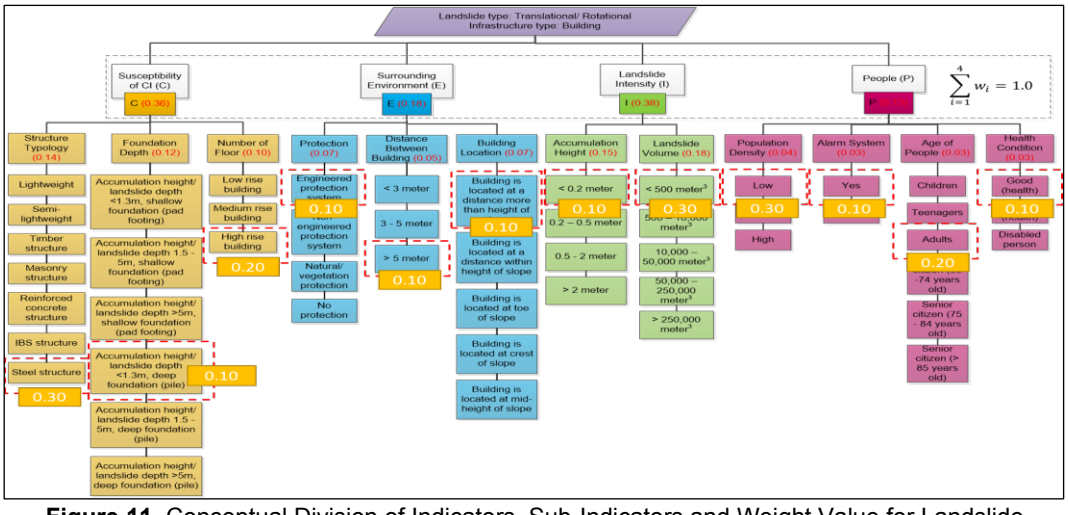


Figure 11. Conceptual Division of Indicators, Sub-Indicators and Weight Value for Landslide Vulnerability Scenario

An example of Scenario 1 (CI- Building, Landslide type – translational/rotational) is shown in Figure 12. The summation of all scores in yellow box is equivalent to the vulnerability index for Scenario 1 (Figure 12). The index is then referred to the vulnerability classes in Table 8 and description of possible degree of severity of damage to the critical infrastructure is determined.

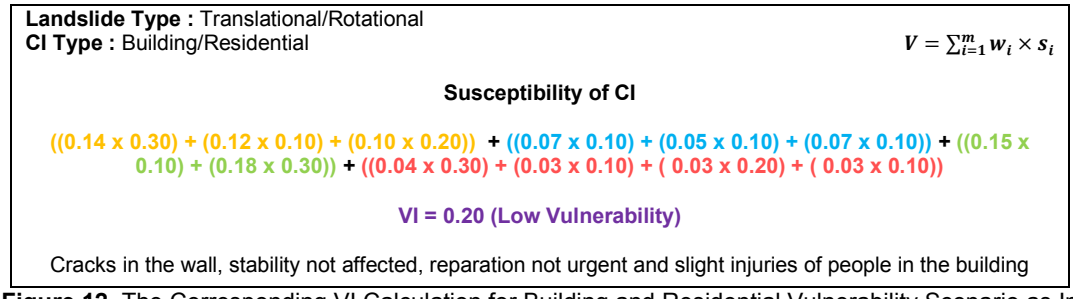


Figure 12. The Corresponding VI Calculation for Building and Residential Vulnerability Scenario as In Figure 11.

Table 9 summarises the total number of 358 buildings within the study area. The number of building is based on hazard classification and vulnerability classes is as shown. Ultimately the risk classifications of buildings were determined from the respective maps.

Table 9. Statistics of Building from Hazard, Vulnerability and Risk Maps at Lembah Bertam

Building at Lembah Bertam	Number of Building
Total Building	358
Hazard	
Building at Hazard (Low)	13
Building at Hazard (Medium)	104
Building at Hazard (High)	150
Building at Hazard (Very High)	91
Vulnerability	
Building at Vulnerability (Moderate)	358
Risk	
Building at Risk (Medium)	117
Building at Risk (High)	241

DISCUSSION

In general, the issues of each method can be grouped into the following aspects which are data requirement and availability; subjectivity and uncertainty of the approaches; applicability of the methods and transferability of the methods. Generally, the quantitative approach of landslide vulnerability assessment through the application of vulnerability curves requires huge amount of record compilation for landslide damages. On the other hand, the application semi-quantitative and qualitative approaches using IBM and vulnerability matrices requires less amount and detailed information. However, the subjectivity and uncertainty of these methods are higher compared to the quantitative approach. In terms of the applicability of the method, the semi-quantitative method is more popular for decision maker with its versatile approach in selection of indicators and estimation of the final vulnerability value for a specific element-at-risk. The methods allow participations of non-expert users for data collection and it can be relatively easy to be used by the owner of the element-at-risk to estimate the

vulnerability index for their properties. Furthermore, the use of Geographical Information System (GIS) in the vulnerability assessment allows spatial analysis for "what if" scenario on the vulnerability data for example analysis that aims at reducing vulnerability and risk by applying specific mitigation measures. However previous studies have also strongly suggested the inclusion of the landslide hazard intensity in the vulnerability assessment. In addition, detailed method of vulnerability index classification (e.g., low, medium and high vulnerability level) and its corresponding description of damages should be defined carefully for each element-at-risk.

One of the important components in constructing the vulnerability index using indicator-based method is the landslide intensity. High landslide intensity values indicate vulnerable CI. As there is no specific procedure and standard approach to computing landslide intensity in Malaysia, computation is based on three parameters (accumulation high of the landslides, landslides thickness and landslide volume). These parameters were estimated from expert judgement as no proper record of landslide inventory is available. The proper method to quantitatively measure the landslide inventory can improve the accuracy and reduce the error of estimating landslide intensity values. The landslide vulnerability and risk assessments require the hazard map to be produced for a different type of landslides. This allows separate landslide risk estimation process for a different type of landslides and critical infrastructures. The landslide hazard map should be developed based on the landslide susceptibility map that takes into account important local landslide causal factors and triggering factors.

CONCLUSION

Inevitably in Malaysia the compilation and recording important information of landslide damages are still at early stage. Therefore, development of empirical model between landslide hazard intensity with the damage in a quantitative manner are always hampered with the quality and the quantity of the required data. Thus, based on the literature reviews and current situation in Malaysia, the semi-quantitative IBM method is suitable for landslide vulnerability assessment. In this case, determination of indicators and its corresponding weight values will be highly based on the expert judgments. The assignment of weight will be also assisted with the existing validated quantitative landslide impact models in Malaysia.

In conclusion, the study has successfully achieved the objective to assess and develop the parameters-indicators of landslide vulnerability assessment of critical infrastructures (CI) and assigning level for each parameter is addressed. The landslide vulnerability indicators, sub-indicators and its corresponding weights were tested in Ringlet and Lembah Bertam, Cameron Highland with supports from various remotely sensed data, field data and other ancillary geospatial data.

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EARLY PRACTICES OF KAMs REPORTING IN MALAYSIA: EVIDENCE FROM CONSTRUCTION SECTOR

Ummi Junaidda Hashim[1], Norsiah Ahmad[1], Wan Amalina Wan Abdullah[1] and Zalailah Salleh[2]
[1]School of Accountancy, University Sultan Zainal Abidin, Malaysia.
[2]Accounting Program, University Malaysia Terengganu, Malaysia.

Abstract
The construction sector has been the subject of the study due to the fact that construction sector, despite being a high-risk sector, is among important sectors that contribute to the development of the economy of a country. This paper aims to identify whether the specific characteristics of construction company will influence the Key Audit Matters (KAMs) reporting for Malaysian companies. The current study examines 46 of the construction companies that listed at the main market of Bursa Malaysia. The maximum and minimum numbers of KAMs disclosed are six and one respectively for this sector. The main issue addressed as KAMs in construction sector is revenue. The revenue recognition in the construction sector is affected by the unexpected occurrences and this, in turn, will affect the estimated cost and profit. The findings discover that the practice on the KAMs disclosed has been critically assessed by the auditors, but there is exception on some of the practice. In the construction sector, the items listed as KAMs basically relates to areas that required a significant amount of judgement from auditors (matters that involve high uncertainty estimation). However, for some of the KAMs disclosure, 24% of the sample do not highlight the reason of why the items listed as KAMs. This circumstance would provide questions whether auditors had dismissed the guidelines of the disclosure of KAMs or auditors did not understand the KAMs' practices comprehensively. Therefore, further monitoring is needed to ensure the successful implementation of KAMs.

Keywords: *Key Audit Matters (KAMs); new audit report; construction sector.*

INTRODUCTION

The new requirement on auditors' report; International Standard Auditing 701 (ISA701), Communicating the Key Audit Matters has been released by the International Auditing and Assurance Standards Board (IAASB) during the year 2015. The ISA 701 is mandatory to all listed companies starting from 15 December 2016 onwards. The KAMs is reported the most significant matters for the current period based on the auditor judgement. There is only a few of literature in this area of study and none mainly focus on the construction sector.

Prior to the issuance of this new standard, few regulators including of International Auditing and Assurance Standards Board (IAASB), Financial Reporting Council (FRC), Public Company Accounting Oversight Board (PCAOB) and others had worked on the similar project to revise and change the auditors' report including of KAMs. This particular KAMs is required by as the existing audit practices offer less informative disclosure to investors (Carcello, 2012: Mock et al., 2013). In spite of that, the findings of the engagement shared by the auditor are limited to an opinion paragraph only. This provides limitation of information to the investors and do not assist them in their decision-making process towards the companies. Accordingly, it is also observed that the transparency and integrity level is low in auditing practice. Therefore, the additional disclosure in the independent auditor's report is important to enhance the communicative value of the report (FocusM, 2016).

Accordingly, the construction sector has been identified as the sample for this study due to the fact that this sector is among the important sectors that contribute in developing the economy of a nation (Fayed & Ehab, 2017) . This sector has the power to generate wealth, improve the quality of living and hence, impose the booster effect to the other sector. It can be characterized as a unique sector whereby it involves extraordinary specialist, professions, variety of suppliers and applies construction technology to enhance profit and to be competitive with global market (Ramli et al., 2018). However, in Malaysia, this sector has been lagging behind as compared to other sectors.

The current study will examine the extent of KAMs practice in the independent auditor's report and also to identify whether the critical factors would be included as the primary issues of the KAMs disclosure. The new study of KAMs will add the literature in auditing particularly that focusing in construction sector which is as a vital part of the market. In spite of that, the finding of the current study is expected that would slightly impose implications to the regulators towards enhancing the reporting of KAMs in the independent auditor's report.

BACKGROUND OF MALAYSIAN CONSTRUCTION SECTOR

The industrial process and the development of a country is directly in relationship with the construction activities (Ofori, 2015). The construction industry is often considered as an engine of economic growth moreover in developing economies. This is because the sector assists other sectors growth. Ramli et al. (2018) demonstrate that this sector provides employment opportunity to the society on various levels of people including of expert and non-expert of them. Hence, it has a great impact on the socio-economic development of a country.

The development of the construction sector grows differently for each country. For the Malaysian market, the construction sector has been accepted as an important sector as early of its independence year, during 1957. Year by year, the construction sector in Malaysia has been modernised and advanced to be at par with developed markets such as U.S, Australia and United Kingdom. The construction process employs highly sophisticated techniques to construct complex infrastructure and the skyscraper projects. These techniques allow speedy execution of many projects such as commercial and industrial buildings, highways, expressways, bridges and tunnels, housing schemes, schools and hospitals, monorail and mass rapid transit rail system, and power plants.

There are a number of major projects that had been completed in Malaysia after thirty years of the independence. The most prominent of these are Petronas Twin Tower as the tallest world tower at the time it completed and Kuala Lumpur International Airport (KLIA) that has been awarded as the best world airport. Both iconic constructions occurred during the year 1992 to 1998.

The recent building which planned to be completed by 2019 is the Exchange 106 or Tun Razak Exchange. The Exchange 106 will be the new tallest building in Malaysia and South East Asia with 492.3 meters versus the previous highest Malaysian building 451.9 meters. It also included among the top fifteen tallest building in the world. The other interesting facts about Exchange 106 are wide parking spaces up to 2,000 units of car, using green technology, super-fast process as each floor completed within two to three days and involved 2,000

workers from different nationality. However, it is found that a delay of completion to the year 2019 instead of the actual plan should be completed by the year 2018. That's as the common issues of construction project as discussed in the following section.

Khan et al. (2014) had discussed that during the year 2010 there are 1.02 million people engage in this sector. In the year 2017, the Annual Economic Statistics Construction Report (2018) shows that 1.33 million people had engaged in this sector. This provides a cumulative incremental of 30%. This shows that the workforce in construction sector has increased over the years and the due demand are from the market including of civil engineering, residential buildings and non-residential buildings.

LITERATURE REVIEW

Construction sector is one of the sectors that always suffer on financial issues either in local or global market. For example, in the Malaysian market, big construction companies always manage to obtain government "injection" in ensuring its continuity in operation due to the importance of the construction sector to the economy. The injection is in the form of new projects by government. The government itself is one of the biggest clients in the industry. Literature also stresses that the construction sector has power to influence and it is important to the overall economy condition of a country (Fayed & Ehab, 2017; Hamzah et al., 2011; Malik et al., 2018; Tzeyin, 2013). Therefore, further attention to the sector is important to ensure its continuity to accelerate the countries' growth.

Issues in the Construction sector

The conceptual framework for financial reporting provides that the primary objective of the financial reporting is to provide decision useful information to all existing or potential stakeholders (IASB, 2018). Particularly, the stakeholders will focus on the profit earned for the year when deciding whether or not to invest in the company. The profit is produced from revenue earned during the year and deducted over the expenses incurred for the year. Few issues have been identified related to this sector including revenue recognition, construction delay and financial problems which are interconnected to one another. The first issue on revenue recognition has discussed follows.

Revenue is one of the most important part to measure financial performance of companies (Wagenhofer, 2014). The survey conducted by Graham et al. (2005) reveal that financial executives officers have ranked revenue as more important than cash flow to the stakeholders. Therefore, the determination and reporting of revenue should be correct and reflects the actual condition of the company. In a construction company, determining revenue is an important part since it involves estimation and judgement of the management on the percentage of completion and cost incurred.

Determination of revenue in the construction company is one of the toughest parts to the management. It is required to make estimation in percentage based on work completed. The estimation could be either over or underestimate. The under estimation of cost would impact to the over declaration of profit for the current year. If the over declaration of revenue occurs, it would be un-avoided problem that need to be catered by the management. In a construction project, the efficiency of the project management would impact the four items including cost,

time, quality and profit (Al-rubaiei et al., 2018). These four aspects are interrelated to one another in influencing the successfulness and profitability of the company. However, not all of the companies manage to apply and monitor the four mentioned aspects in their construction process.

Nevertheless, construction sector has always being criticized due to various factors either during or after the project had been completed. The second issue is on the delayed or abandoned project. This situation occurs due to various factors and it indicates the seriousness of the problem that would impact the financial health of the company. The delay of the project is sometimes due to the variation orders, rework and destruction (Oyewobi et al., 2016). Riazi et al. (2018) identify that the labour productivity issues in Malaysian market had caused lack of efficiency on the project that need for a revolution in construction process. These are among the factors that avoid of the zero-delay achievement.

The delay is a crucial issue in construction sector as it would increase the cost and overhead of a company. Hamzah et al. (2011) provide another view that the financial problems are among the determinant factors of the project delay. The third issue is on financial problem which is an interrelated issue as the payment pending has rooted to the cash flow matters, and subsequently the cash flow problems would be a constraint on getting a loan for continuing with other projects. Market factors such as high interest rate and inflation also deteriorate the abovementioned circumstance, thus would influence the revenue recognition process.

The Reporting of KAMs

Independent auditor's report is an important part of disclosure in an annual report of a company. However, the report has been presenting common information over the years and hence, less informative. The users themselves have demanded an informative independent auditor's report rather than providing an opinion paragraph only. In an effort to satisfy the users that demand on more informative audit report (Mock et al., 2013), the reform on auditor's report has been done after the years of discussion by the IAASB and other regulators. The process of reforming independent auditor's report includes introducing a new standard, ISA 701 Communicating Key Audit Matters (ISA 701) in the Independent Auditor's Report. The reporting of KAMs in the independent auditor's report is mandatory for all listed companies starting from 15 December 2016. The survey conducted by Carcello (2012) reveals that the professional users of financial statements request on having more disclosure by auditors which highlights the significant matters of the financial statements and audit risks.

ISA 701 was released by the IAASB in January 2015 and Malaysia Institute of Accountant (MIA) has approved the standard in April 2015. The standard has defined KAMs as any matters, based on auditor's judgment, are that most significant matters in the audit financial statements for the current period and the matters are among matters that have been communicated with those charges with governance of the company.

In addressing the KAMs, there are three items that needs to be considered by auditors as; (1) auditors need to assess the area that higher risk of material misstatement, (2) assessment need to carry out on the areas that required the significant judgement of auditor for the matters that involve high uncertainty estimation, (3) auditor need to consider the effect on the audit of significant events or transactions that occurred during the period.

The identifying on the significance matters for the current period imposed a risk-based that need further recognising process including of assessing on the matters that has higher risk of material misstatement, designing and performing audit procedures responsive to those risks and obtain sufficient and appropriate audit evidence. The KAMs also would come from the matters that impose challenges to auditor in collecting the audit evidence.

The standard also provides that the significant matters would arise from the complex matters that need auditor judgement and this would reflect the audit strategy or further necessary involvement of senior personnel to resolve those issues. The communicating of KAMs require to have a separate paragraph in the independent auditor's report and the lengthy of the paragraph would depend on the auditor's judgement in effort to assist users to understand on the matters that most significance for the current period.

In Paragraph 13 of the standard, it highlights that the communicating of KAMs must address the following;

1) the reason of why the item has been choose as KAMs,
2) the description on how the auditor address the issue,
3) the reference on the related disclosure as part of the evidence of KAMs items.

When the auditor addresses the above items, it would assist users particularly the investors to understand the issue of KAMs in their investing companies. Hence, the KAMs would be useful to them as it enhances the information in the independent auditors' report. The reporting of KAMs is supporting the communication theory whereby communication is achieved when it communicates adequate information. Therefore, KAMs will enhances the content of independent auditor's report and enable users do their best decision on their investment (Hashim et al., 2018).

RESEARCH DESIGN

The samples of the study are 48 construction companies listed on Bursa Malaysia upon the implementation of KAMs in year one. The final sample totalled to 46 as shown in Table 1 due to the unavailability of the annual report of the companies. The list of the sample as attached in the appendix. All of the final sample of the companies are satisfied and has been excluded from the filter characteristics of paragraph five, ISA 701 whereby when a company received the disclaimer opinion, the auditor is prohibited from issuing the KAMs (IAASB, 2015).

Table 1. Sample Size Selection

Total construction companies	48
Less unavailable of annual report/delisted/PN4	(2)
Less filter characteristic	0
Final sample of the study	46

In achieving the objective of this study, a comprehensive review on the KAMs disclosed in the independent auditors' report has been done by the content analysis. Content analysis is a powerful method in research as it has noted the meaning seriously and it has been used widely in all area such as public opinion research, market research, political issue and on other emerging issues (Krippendorff, 2004).

This study examines the number of KAMs disclosed by the companies and classifies them in according to their items/categories after referring to External Reporting Board and Financial Markets Authority (2017); Mazars (2016) and Securities Commission Malaysia (2018) with some adjustments according to the data of KAMs collected. The keywords applied including the reference, reason and the procedure to mitigate the KAMs matters that has been identified. These keywords are important as they would assist users to understand of why auditors judge the matters as KAMs.

RESULTS AND DISCUSSION

The following discussions provide the KAMs practices for construction sector for the year one.

Simple descriptive analysis

The descriptive analysis as Table 2 shows that average/mean on of the KAMs disclosure in construction sector is 2.87 and this is no difference with (Bédard et al., 2014) whereby the mean of the KAMs is 2.5. however, dissimilar to (Pinto & Morais, 2019) provides the mean of KAMs for 3.84. With regards to the maximum number of KAMs disclosed, the other study of KAMs in European countries, Pinto & Morais (2019) reported the maximum number of KAMs is nine which is relatively difference on KAMs reported in the current study with six items only. In contrast to the minimum number of KAMs, the current study finds that one KAMs has been reported while (Pinto & Morais, 2019) has reported zero KAMs for the minimum.

The results of this study are partly similar to (Bédard et al., 2014) for France market and it relatively difference with (Pinto & Morais, 2019) for the European market. However, this could be difference if it covers the study of KAMs in other sector of companies.

Table 2. Descriptive Statistics for KAMs (N=46)

Year one of KAMS	
Minimum	1
Maximum	6
Mean	2.87
Median	2.5
Mode	2

The frequency on number of the items disclosed is as shown in Table 3. Mostly, 41% of the samples has reported with two items of KAMs in the independent auditor's report and then follows with three items of KAMs on 23% of the samples. Another frequent on the disclosure is 13% of the samples with five items disclosed and about 9% of the sample with frequency of one item of KAMs. The 6.5% of the samples has disclosed with five items and six items of KAMs respectively.

Table 3. Frequency on Numbers of Item KAMs Disclosed

	Frequency of KAMs disclosure						**Total**
	1 item	**2 items**	**3 items**	**4 items**	**5 items**	**6 items**	
N (no of companies)	4	19	11	5	3	3	46
Percentage (%)	8.69	41.3	23.93	13.04	6.52	6.52	100

The disclosure of KAMs for the construction sector in Malaysia is made up of 14 items as presented in Figure 1. The including items are impairment on assets (other than goodwill), tax, revenue recognition, fixed assets (mostly on carrying value/fair value/valuation/disposal), other income, impairment of goodwill, provisions, liability assessments, cost/charges, acquisition, going concern, receivables, inventory and others.

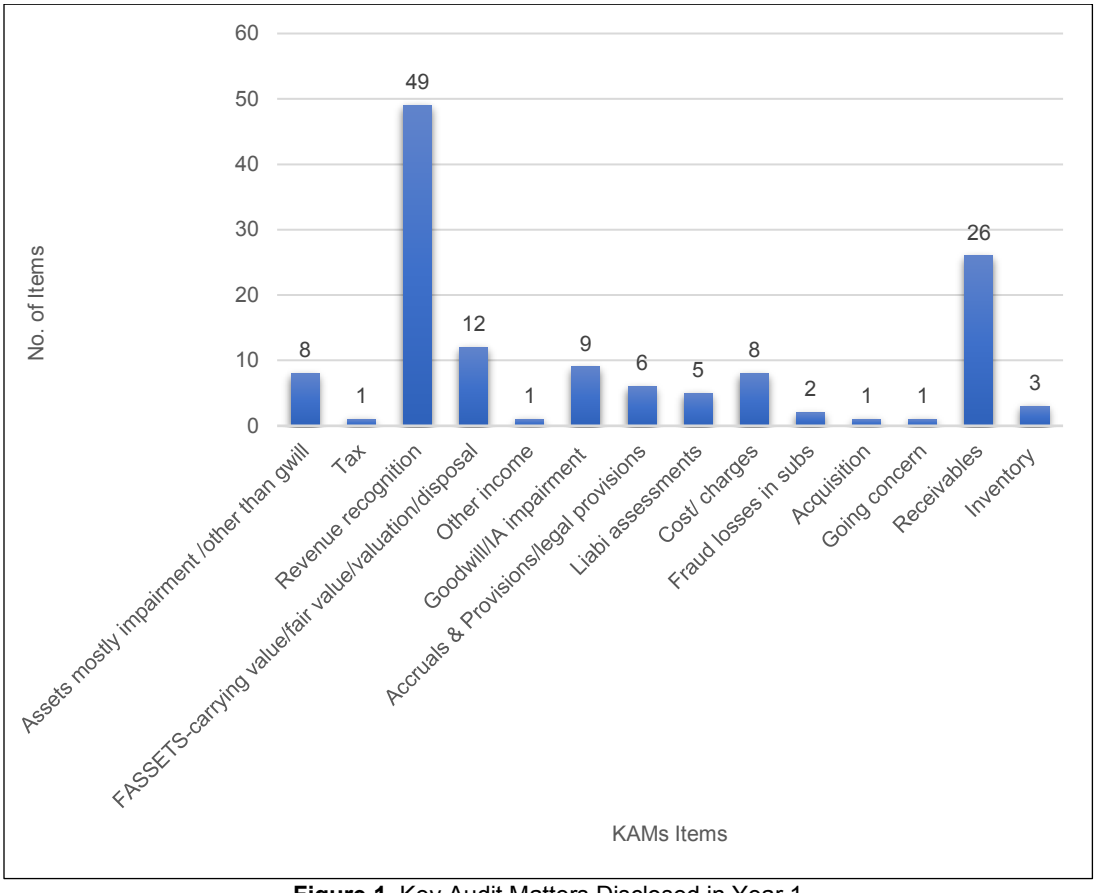


Figure 1. Key Audit Matters Disclosed in Year 1

As highlighted in the standard, the reporting of KAMs is involving the items that mostly significant of the current period. The revenue recognition is among the items that has been identified as the important issues to be raised as the KAMs in construction sector with a total of 49 items. It is identified that there are few reasons of why the revenue recognition are has listed as KAMs in the disclosure. Firstly, the revenue for the construction company is derived from long term construction contracts which are spanned over more than one accounting period. Second, the construction project involves a huge amount which is obviously material to the company. Third, it required the judgement and estimation process that inherently complex in recognition of the revenue. Fourth, the significant judgement is exercised by management in determining the percentage of completion of the contract and estimates on the forecast costs of the contracts. Fifth, it would result in variance when the cost incurred difference with the estimations and this reflect the profit or loss in the current financial year. With that, revenue recognition is agreed as the material matters to be highlighted as KAMs by majority of the auditors in construction companies.

The other items that frequently disclosed as KAMs for the construction companies is receivables with total of 26. Mostly, the issues raised are on the recoverability of the account receivables. Identically, the matters of recoverability of the receivables are involved on management's judgement to indicate either the receivables amount would be impaired or not. It also includes on the monitoring and analysing on the individual receivable's accounts based on specific known facts or circumstances of the customer to assess either the customer has ability to settle their due amount or not. The analysis of the receivable's accounts are including of trade receivables accounts, other receivables accounts and amount due from contract customer.

In summary, the details for the specific items that disclosed as KAMs is presented in Table 4 for Year 1 in construction sector.

Table 4. General and Specific Items of The KAMs Disclosed

Items/categories of KAMs:	**Specific items of KAMs:**
Revenue recognition	Revenue Recognition on Construction Contract
Receivables	Recoverability of Other Receivables on arbitration award
	Settlement of amount due from a contract customer
	Recoverable on amount due from associates & arbitration award
	Valuation of trade receivables
	Impairment loss of receivables
	Recoverability of amount due from related parties
Fixed assets	FV of Investment properties
	Valuation of investment properties
	Carrying amount of investment
	Carrying Amount of Concession Right
	The Carrying Value of Investment in Associate
	Valuation of investment in an associate
	Fair Value of The Property, Plant and Equipment
Assets mostly impairment/exclude goodwill	Impairment assessment of barges and dredgers
	Impairment Assessment of Concession Assets
	Impairment of interest in a joint venture
	Impairment assessment of the Group's and Company's investment in joint venture
	Impairment assessment of service concession assets
	Impairment assessment of property, plant and equipment
	Impairment assessments of cost of investment in subsidiaries
	Impairment assessment infrastructure development expenditure
Cost/charges	Expenses recognition for construction business
	Foreseeable loss on contract
Goodwill impairment/IA impairment	Impairment of goodwill on consolidation
	Annual impairment test of goodwill
	Impairment assessment land rights
Provisions	Assessment of liquidated and ascertained damages ('LAD') on supply and construction contracts
	Contingent Liabilities – Major Litigations and Claims Reported
	Crystallization of contingent liability
	Outflow of resources on litigations and arbitrations
Liability	Amount due from customers

Items/categories of KAMs:	Specific items of KAMs:
Inventory	Valuation of inventory
	Existence of inventory
	Inventory valuation on unsold properties
	The FV of the Inventories
Going concern	Material Uncertainty on Going Concern
Acquisition	Purchase price on acquisition
Tax	Deferred tax assets
Income	Other income
Fraud losses	Matters on group consolidation with foreign subsidiaries

Table 5. The Auditors' Reason of Choosing as KAMs

	Why choose as KAMs	Frequency
Reasons	Assess the area that higher risk of material misstatement	1
	Assessment the areas required the significant judgement of auditor (matters that involve high uncertainty estimation)	71
	Auditors need to consider the effect on the audit of significant events or transactions	4
	Materiality of the transactions	8
	Complexities of the transaction	12
	Highly subjective matters	3

Table 5 presents the auditors identified reason in rationalizing of KAMs disclosed in the independent auditor's report. Based on the review of the KAMs in the construction companies, the auditors identified that there are 71 of the KAMs items is due to the matters required significant judgement of the auditor and it involves high uncertainty estimation. The following part provides the extraction of the independent auditors' report with different reasons addressed by the auditors.

KAMs Extract

Example of such justification has been extracted as follow;

a) KAMs Items Reported Is Due to The Matters Required Significant Judgement of The Auditor and It Involves High Uncertainty Estimation

> Revenue on construction contract contributed to 93% of the Group's total revenue.
>
> The Group recognizes contract revenue and costs in profit or loss by using the percentage of completion method. The percentage of completion is determined by the proportion that contract costs incurred for work performed to date bear to the estimated total contract costs.
>
> *The revenue recognition on construction contract is considered to be a matter of significance as significant judgement is exercised in determining* the percentage of completion, the extent of the costs incurred, the estimated total contract revenue and costs, as well as the recoverability of the contract projects.

> The construction contract revenue and construction contract cost recognised in the profit or loss are disclosed in Notes 4 and 5 to the financial statements respectively.
>
> Annual report Ahmad Zaki Resources Berhad, 2016

> The Group's Investment properties are carried at fair value. The Executive Directors engaged independent external values to determine the fair value of the investment properties every five years.
>
> On an annual basis, the Executive Directors performed internal valuation based on actual transactions of similar type of properties and location.
>
> We have identified the valuation of investment properties as a key audit matter because valuation included *significant assumptions which are judgmental*.
>
> Annual report Ireka Corporation Berhad, 2017

> Financial impact of the in project in Abu Dhabi
>
> The Group terminated its project in Abu Dhabi, United Arab Emirates, on 1 October 2015 and entered into an arbitration proceeding with the project owner. The arbitration commenced on 11 December 2015. As at 31 December 2016, the Group recorded a total receivable balance of AED207.7 million (RM253.7 million) due from the project owner in the consolidated statement of financial position.
>
> The Directors are of the view that the Group has rightfully and validly terminated its employment under the contract with the project owner. The Directors have made an assessment and concluded that the total receivable balance is fully recoverable based on advice from the independent claim consultant and external solicitors.
>
> The Group also has ongoing litigation and arbitration proceeding with its sub-contractors and a counter claim from the project owner arising from this project. The Directors have not provided for the claims made by the project owner as there is no sufficient evidence to support these claims based on the advice from the external solicitors.
>
> *This is a key audit matter due to the uncertainty relating to the outcome of the various arbitration*
> proceedings in relation to this project.
>
> Annual report Zelan Berhad, 2016

b) KAMs Disclosed Due to The Complexities of The Transactions

The auditor identified 12 items of KAMs which are due to the complexities of the transactions. However, in justifying the complexities of the transactions, it also depends on the auditor judgement either a transaction is a simple or complex. The extract of KAMs with such justification as follows:

> Revenue recognition for construction contracts, due to the contracting nature of the business, involves significant judgments. This includes the determination of the total budgeted contract costs and the calculation of percentage of completion which affects the quantum of revenue to be recognised. In estimating the revenue to be recognised, the management considers past
> experience and certification by customers and independent third parties, where applicable.
>
> We *determined this to be a key audit matter due to the complexity* and judgmental nature of the budgeting of contract costs and the determination of revenue recognised.
>
> Annual report Gadang Holdings Berhad, 2017

c) KAMs Disclosed Due to The Level of Materiality

Level of the materiality of the transactions is also among the reason of an item to be identified as KAMs. The materiality is a concept that widely discuss in both, either in financial reporting or auditing. The ISA 320 provides the guidelines on how the auditor would handle the materiality level of the company. Example of the KAMs on such issue as follow:

> As at the financial year end, the management had exercised judgement by performing recoverability assessment due to no repayment was forthcoming. The management, on ground of prudence, decided to impair the receivable; whilst exploring avenues to institute legal proceedings against Hong Kong Oriental Rich and Force Investment Co Ltd ("HKOR").
>
> For the financial year ended 31 December 2016, the management had made an impairment loss of RM19,779,610 to profit or loss of the Group.
>
> This area was considered to be one of most significance to our audit *due to the substantial sum outstanding* and the inability for the debtor to settle the debt according to the repayment
> schedule agreed upon.
>
> Annual report, Kumpulan Jetson Berhad, 2016

CONCLUSION

The practices of the KAMs in the first year show some weaknesses as identified from the current sample. Few of them do not comply with the requirement on paragraph 13 of the standard (*discussed in previous section)*. Out of twelve of the samples, they had not provided the reason of why the items has been chosen as the KAMs for the current period. There are also four companies that has reported the KAMs without providing the references on the issues highlighted. The users possibly would not be able to understand the KAMs reported since it has not been addressed properly by the auditor when it not fully compliance with the guidance in ISA 701. However, the weakness on the KAMs practices is occurred during the

initial year of regulation. The disclosure practice is assured will be better and more informative for the consecutive years of practices. Table 6 provides figures on the number of companies complying with the guidance on paragraph 13.

Table 6. Compliance on Guideline Paragraph 13

Guideline paragraph 13	No. of companies complied	Percentage
How the matters addressed	46	100%
Reference	42	91%
Reasons of the matters determined as KAMs	34	74%

The KAMs disclosure is the independent auditor's report for construction sector is in agree with the main issue of revenue that concern in the construction sector. Surprisingly, none of the company in the construction sector highlights on the issue of the construction delay and financial constraint as their KAMs. There are few factors that prohibit such matters to be communicated to their stakeholders. It is possibly that the matters would be sensitive, as sharing unfavourable information would be harmful to their audited company. Therefore, the information disclosed to the users is still limited and possibly the extent purpose of KAMs is not successfully achieved in this particularly sector. Hence, the communication theory in communication of audit is still constraint for the recent year. The limitation of the study is generalization cannot be made since this study offers an empirical data for the year one of KAMs only.

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Appendix

Sample of construction sector in Malaysian market

1.	Ahmad Zaki Resources Berhad
2.	Benalec Holdings Bhd
3.	Bina Puri Holdings Bhd
4.	Brem Holding Berhad
5.	Crest Builder Holdings Bhd
6.	DKLS Industries Bhd
7.	Econpile Holdings Bhd
8.	Ekovest Bhd
9.	Eversendai Corporation Bhd
10.	Fajarbaru Builder Group Bhd
11.	Gabungan AQRS Bhd
12.	Gadang Holdings Bhd
13.	Gamuda Bhd
14.	George Kent (Malaysia) Bhd
15.	Ho Hup Construction Company Bhd
16.	Hock Seng Lee Bhd
17.	IJM Corporation Berhad
18.	Ikhmas Jaya Group Bhd
19.	Ireka Corporation Bhd
20.	JAKS Resources Bhd
21.	Kerjaya Prospek Group Bhd
22.	Kimlun Corporation Bhd
23.	Lebtech Bhd
24.	Melati Ehsan Holdings Bhd
25.	Mercury Industries Bhd
26.	Merge Energy Bhd
27.	MGB Bhd
28.	Mitrajaya Holdings Bhd
29.	MTD ACPI Engineering Bhd
30.	Mudajaya Group Bhd
31.	Muhibbah Engineering (M) Bhd
32.	OCR Group Bhd
33.	Pesona Metro Holdings Bhd
34.	Pintaras Jaya Bhd
35.	Prinsiptek Corporation Bhd
36.	Protasco Bhd
37.	Puncak Niaga Holdings Bhd
38.	Sunway Construction Group Bhd
39.	Sycal Ventures Bhd
40.	TRC Synergy Bhd
41.	TSR Capital Bhd
42.	Vizione Holdings Bhd
43.	WCE Holdings Bhd
44.	WCT Holdings Bhd
45.	Zecon Bhd
46.	Zelan Bhd

PREDICTING 28-DAY COMPRESSIVE STRENGTH OF POZZOLANIC CONCRETE BY GENERALIZED NEAREST NEIGHBORHOOD CLUSTERING USING MODIFIED PSO ALGORITHM

Oveys Ghodousian[1], Keyhan Behdad[1], Vahid Shafaie[2], Amin Ghodousian[3] and Hossein Mehdikhani[1]
[1]Department of Engineering, Takestan Branch, Islamic Azad University, Iran.
[2]Department of Natural Disaster Management, Graduate Faculty of Environment, University of Tehran, Tehran, Iran.
[3]Department of Engineering Science, College of Engineering, University of Tehran, Tehran, Iran.

Abstract
Nowadays, concrete is globally famous as one of the most consumed construction materials. Therefore, based on demand and the expanded concrete usage, the necessity of performing investigation about behaviour, and the compelling factors on its behaviour is apparent. In order to estimate the performance requirements and strength during concrete curing, it is necessary to pay more attention to its construction. Researchers provided different mathematics techniques for estimation of concrete behaviour, so that, the Fuzzy series theory is a handy tool for modelling and analyzing of vague and imprecise concepts. Here in the present article, it is aimed to predict the 28-day strength of concrete containing Metakaolin, micro-Silica, rice husk ash, and zeolite based on 7-day compressive strength using fuzzy logic. In this study we have used 5, 7.5, 10, 12.5 and 15 percent Pozzolan (by weight of cement) and we acquired 20 mix proportions together with one control mix which is free from any Pozzolan. All tests performed under the tests related to concrete workability (slump test) and the test related in hardened concrete (compressive strength) in 7, 28, and 42-day. Finally, 28-day compressive strengths were predicted using linear regression and different methods of fuzzy logic. It is found that generalized nearest neighbourhood clustering method has the most accurate prediction for 28-day compressive strength.

Keywords: *pozzolanic concrete; compressive strength; GNNC-modified PSO algorithm; table look-up scheme.*

INTRODUCTION

About the subject complexity, the regression and the traditional methods for concrete strength modelling are not capable for exact prediction, because concrete strength behaviour is under the influence of linear conditions and it takes influence from the smallest existing components in the mixture and the interaction between these components. The specification of fuzzy logic has made this linear modelling usage very interesting and suitable for predicting different concrete strength. Researchers provided different mathematics techniques for estimation of concrete behaviour, so that, these techniques were mostly based on linear and nonlinear regression in the past (Zain and Abd, 2009; Tsivilis and Parissaki, 1995). Nowadays, artificial intelligence is the foundation of these methods. Here, fuzzy logic is one of the branches of the artificial intelligence (Hong-Guang and Ji-Zong, 2000; Bilim et al., 2009; Ramezanianpour et al., 2004; Sarıdemir, 2009; Özcan, 2009). Thus, most of the decisions making, descriptions and controlling strategies of daily human life are approximate and imprecise.

Fazel Zarandi et al. (2008) provided an effective model for the concrete compressive strength with six inputs and one output by mixing GMDH (Group Method Data Handing-

type) neural networks and fuzzy logic. By utilizing neural networks, M.M. Alshihri (2009) provided a model for predicting the mechanical specifications of the light concrete. Through the usage of neural networks, Jian Ping Jian (2011) predicted the compressive strength for a 28-day concrete. By studying predicting the specification of concrete containing added materials, Dias et al. (2001) concluded that it is beneficial to utilize strength prediction models for more investigation of parameters effective on concrete strength. Zain et al. (2009) studied predicting high strength concrete through regression, and they concluded that artificial intelligence holds better results in comparison with linear regression. Sardemir et al. (2009) performed the predicting compressive strength for the concrete containing Metakaolin through the utilization of neural network and fuzzy logic, and the concluded that neural network and fuzzy logic have been very successful for strength prediction. Through testing on time strength of concrete and self-compression repairing mortar, over different concrete beds and by using rolling methods, fuzzy logic and artificial neural networks, Naderi et al. (2012 and 2013) concluded that it is possible to achieve the results more concordant with laboratory tests between the results from linear regression, Fuzzy model and neural network for self-compacting mixtures. Bagher Zadeh et al. (2015) estimated the concrete compressive strength by utilization of the Artificial neural network and comparative-neural fuzzy deduction system, and they concluded that a neural network with high precision, is capable for the predicting the compressive strength for the data series. Also, Muthupriya et al. (2011) predicted the compressive strength and durability of high-performance concrete by Artificial Neural networks; the study presented the neural networks can anticipate the values of compressive strength for concrete containing Metakaolin, Silica fume and fly ash.

During recent years also, simultaneous with the quick human progress in the field of technical and executive issues in the field of construction and as the result of the researches implemented for construction materials and utilization of natural materials, reinforcement and optimization of the artificial construction materials, it can be observed for very useful innovations and new initiatives. Utilization of Pozzolan is one of the best approaches. The most consuming types of Pozzolans are micro silica, Metakaoline, Zeolite, and rice husk ash. Its correct usage holds a very significant influence on the enhancement of strength and durability of concrete structures. In the paper presented by Swaminathan et al. (2016), the utilization of Rice Husk Ash (RHA) in different concrete was reviewed, and its effects on the concrete specification were discussed. Babu et al. (2016), in their study, showed how the usage of RHA could modify the concrete properties such as increasing the compressive strength. Self-compacting concrete mix with 15% of Rice Husk Ash can improve the strength and workability of concrete (Raviteja et al., 2016). Anjali et al. (2015) based on using Metakaolin in Self Compacting Concrete, presented that compressive and tensile strength of SCC Specimens could be reached higher values in comparison with Conventional Vibrated Concrete (CVC). Through evaluation of specification of self-compacting concrete containing rice husk ash, Ranjbar et al. (2014) concluded that compressive strength, bending and tensile strength of specimens containing RHA in the first ages is less than the control specimen. Prasada Rao et al. (2016) in their study recommended that the combination of Metakaolin (MK) and Nano-Silica (NS) could raise the structural specification of the concrete such as compressive strength. Alsaadat Sabet et al. (2013) investigated the effect of replacing cement with zeolite, fly ash, and micro silica over concrete durability. During the implementation of slump test, they concluded that in comparison with micro silica and fly ash, zeolite decreases more the concrete flow; therefore, when using zeolite, the more super plasticizer should be utilized. They also concluded that when using these types of Pozzolans, compressive strength

will increase significantly. In 2013, Ranjbar et al. also investigated the effect of zeolite and silica mixture over the fresh and hardened self-compacting concrete. In 2014, Tarighat et al. investigated the effect of zeolite on the mechanical behaviour of concrete. In 2016, through investigation of engineering specification of self-compacting concrete containing mixed cement, Ranjbar et al. (2014) did show that utilization of Pozzolan materials such as Nano Silica, micro silica, and rice-husk ash, would result in the enhancement of strength. Furthermore, using Pozzolan in self-compacting concrete would result in decreasing of water leakage and more uniformity of self- consolidating concrete. Pozzolans also decrease the self-compacting concrete flow-ability.

SPECIFICATIONS OF MATERIALS

We used rounded sand for this research. By the performed grading, the utilized sand was not in the permitted area. Therefore, all-consuming sands sieved by sieve number 4, and then we did grading for them. The 0-4.75 mm diameter consuming sand with apparent unit weight in the saturated surface dry condition is 2700 kg/m3 and 1.5 percent for 24 water adsorptions. The two consuming grains of sand grading is by ASTM C33 standard (2008). The consuming gravel passed in 19 mm sieve ¾ inch, and the gravel remained on the sieve is No.4 (4.75mm). It is also worth mentioning that the consuming gravel grading is according to ASTM C33 standard (2008) limitation. Also, the Khazar Portland cement 1-425 type was utilized.

MIX PROPORTIONS

In order to achieve the goals of this investigation, 20 mix proportions, in addition to one control mix were used, which is presented in Table 1.

Table 1. Mix Proportions in This Study

No.	Name	C (Kg/m3)	PT	P (Kg/m3)	G (Kg/m3)	S (Kg/m3)	W (Kg/m3)	W/(C+P)
0	CTRL	400	R.H.A	0	883.25	883.25	200	0.5
1	R5	380	R.H.A	20	880.03	880.03	200	0.5
2	R7.5	370	R.H.A	30	878.41	878.41	200	0.5
3	R10	360	R.H.A	40	876.79	876.79	200	0.5
4	R12.5	350	R.H.A	50	875.18	875.18	200	0.5
5	R15	340	R.H.A	60	873.56	873.56	200	0.5
6	M5	380	MK	20	881.49	881.49	200	0.5
7	M7.5	370	MK	30	880.61	880.61	200	0.5
8	M10	360	MK	40	879.73	879.73	200	0.5
9	M12.5	350	MK	50	878.85	878.85	200	0.5
10	M15	340	MK	60	877.97	877.97	200	0.5
11	S5	380	MS	20	879.66	879.66	200	0.5
12	S7.5	370	MS	30	877.86	877.86	200	0.5
13	S10	360	MS	40	876.06	876.06	200	0.5
14	S12.5	350	MS	50	875.26	875.26	200	0.5
15	S15	340	MS	60	874.46	874.46	200	0.5
16	Z5	380	ZE	20	880.17	880.17	200	0.5
17	Z7.5	370	ZE	30	878.63	878.63	200	0.5
18	Z10	360	ZE	40	877.09	877.09	200	0.5
19	Z12.5	350	ZE	50	875.56	875.56	200	0.5
20	Z15	340	ZE	60	874.02	874.02	200	0.5

(Source: Authors) (C: Cement; PT: Pozzolan Type; P: Pozzolan; G: Gravel; S: Sand; W: Water)

TEST IMPLEMENTATION AND RESULTS ANALYSIS

Slump Loss

After the construction of the 20 mix proportions mentioned above, slump test conducted for each one. We used the slump test for the estimation of the concrete's workability (Figure 1). Figure 1 shows that the slump loss intensely depends on the type and amount of Pozzolans. Slump loss just for the mixes containing 10% rice husk ash, and 15% Metakaolin, had more slump loss in comparison with control.



Figure 1. Slump Test

It can be observed that Micro Silica and Zeolite brought about the most adverse effects (approximately 54% and 57% slump loss, respectively, in comparison with the control mix). On the other hand, it seems that the least adverse effects of mixes contained Rice Husk Ash (With slump loss of about 5% compared to the control mix). Also, the appropriate percentages of each Pozzolan according to workability are as follows: 10% for the mixes containing Rice Husk Ash, 15% Metakaolin, 10% Micro Silica, and 5% Zeolite.

Compressive Strength

Three 15 cm cubic Specimens from all mix designs were made and then immersed in the water tank. After that, the specimens were taken out of the tank at ages of 7, 28 and 42-day, to test their strength based on compressive strength determination in a saturated condition. In this study, the compressive strength test is according to ASTM C39 standard (2009).

Table 2. 7, 28 and 42-Day Compressive Strength for All Mix Proportions

NO.	Name	7-day MPa)	28-day (MPa)	42-day (MPa)
0	CTRL	32.11	41.64	45.83
1	R5	22.53	34.47	44.04
2	R7.5	24.08	35.11	47.03
3	R10	25.22	36.36	48.44
4	R12.5	22.33	34.03	45.79
5	R15	20.98	29.60	38.22
6	M5	22.07	32.00	44.31
7	M7.5	23.08	33.70	45.62

NO.	Name	7-day MPa)	28-day (MPa)	42-day (MPa)
8	M10	21.53	30.51	39.11
9	M12.5	19.82	29.93	38.34
10	M15	18.67	28.27	37.56
11	S5	23.44	36.67	44.89
12	S7.5	27.25	39.10	47.55
13	S10	31.00	42.71	49.78
14	S12.5	30.78	42.04	49.82
15	S15	30.33	41.58	49.33
16	Z5	21.40	32.24	43.56
17	Z7.5	22.07	32.51	43.99
18	Z10	23.73	32.60	45.33
19	Z12.5	18.43	30.82	41.43
20	Z15	17.91	30.02	39.78

(Source: Authors)

As the graph shows, there exists much more difference between 28-day compressive strength with 42-day in comparison with control mix among Pozzolan containing mixes. The reason relates to less hydration temperature in comparison with cement and their slower reaction. For that reason, this issue results in slowing down the strength acquiring process among these specimens. Therefore, it is observed that in Pozzolan containing concretes, the 42-day compressive strength is much more than their 28-day one.

The relationship between the type and amount of pozzolan with compressive strength

Figure 2 shows the relationship between the type and amount of Pozzolan with compressive strength.

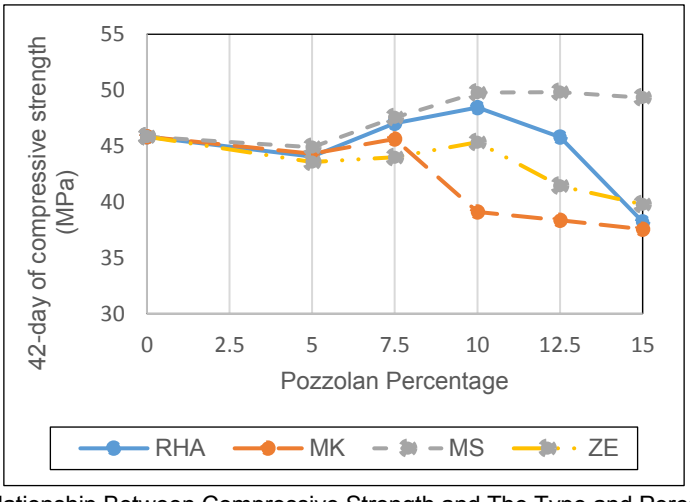


Figure 2 . The Relationship Between Compressive Strength and The Type and Percentage of Pozzolan

It is possible to determine the optimized percentage for every Pozzolan in order to achieve the highest compressive strength. As can be seen, the best percent for rice husk ash is 10, then 5 and 15 percent. When 5% Metakaolin replaces with cement, it gives the highest compressive strength, and the replacement percent enhancement leads to decreasing in compressive strength. Micro silica also with 10% replacement brings about the best result. The amount of

15% of micro silica gives the same result with less difference, and finally, 5% of micro silica produces the least compressive strength. By replacing cement with 10% zeolite, the highest compressive strength in concrete could result. The amount of 5% replacement leads to the average compressive strength and 15% replacement causes the worst results. Therefore, it could be concluded that except Metakaolin which its optimized amount is 5%, for other Pozzolans the amount of 10% is considered as the optimized one. It is also visible that the highest results relate in micro silica which brings about the highest compressive strength and this expresses one of the most important reasons for the utilization of this type of pozzolan as the most prevalent one in the concrete industry. Metakaolin also gives the minimum results.

The relationship between 7, 28 and 42 days compressive strength

Figure 3 shows the increasing amount for the compressive strength of specimens with the enhancement of curing days. Based on this chart, we see that the proportion of 7-day compressive strength to 28-day compressive strength for mix designs containing 10 & 15 percent micro silica and 5 percent of zeolite is more than other mixes. It is not observed, a significant difference between these mixes. The ratio between 7-day and 28-day compressive strength for all mixes is 0.6 to 0.8. The ratio of 42-day to 28-day compressive strength for all mixes remains 1 to 1.4. For the control mix 7-day to 28-day, the ratio is similar, but for 42-day to 28-day, the ratio is lower.

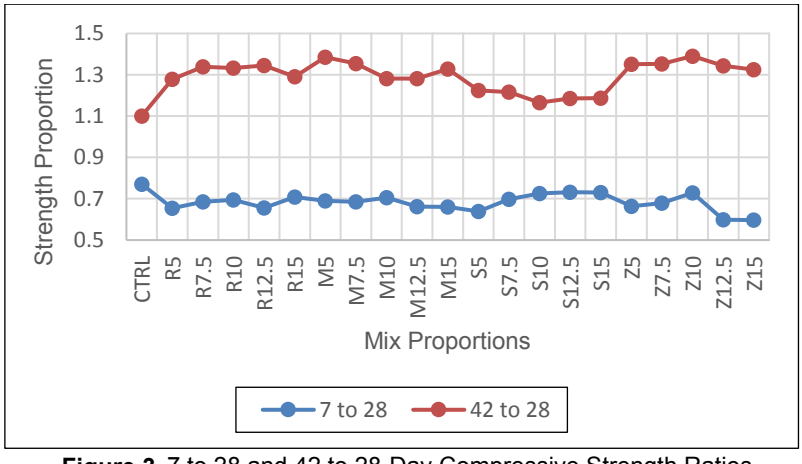


Figure 3. 7 to 28 and 42 to 28-Day Compressive Strength Ratios

The mixes containing zeolite show the most 42-day to 28-day compressive strength ratio. Therefore, more extended curing for specimens containing zeolite in compared to other Pozzolans would be necessary.

PREDICTING 28-DAY COMPRESSIVE STRENGTH

The linear regression analysis for predicting a 28-day compressive strength of specimens conducted by 7-day compressive strength, Pozzolan amount, and Pozzolan type. About the acquired R^2 coefficient, which is equal to 0.941, the acquired equation precise, is very suitable. Equation 1 acquired from linear regression analysis results.

$$CS_{28} = 14.557 - 0.0561\ PT - 0.018\ PD + 0.934\ CS_7 \quad (1)$$

Where CS_{28} is 28-day compressive strength of a 15 cm cubic specimen (MPa), PT is Pozzolan type: 1 for micro silica, 2 for rice husk ash, 3 for Metakaoline, 4 for zeolite and 5 for without Pozzolan, PD is the cement replacement by pozzolans by weight, and CS_7 is 7-day compressive strength of a 15 cm cubic specimen (MPa).

After predicting 28-day compressive strength using linear regression, we firstly generalize a simple Clustering technique called Nearest Neighbourhood Clustering (Wang, 1996) for designing the fuzzy system from input-output pairs. For the sake of simplicity, we refer to this Generalized Nearest Neighbourhood clustering as GNNC. Secondly, a modified PSO algorithm is proposed to optimize the parameters of the fuzzy system by movement in a search space, including both continuous and integer variables.

In Modified PSO, each particle modifies both the optimal fuzzy system by adjusting parameters $\sigma_1, \sigma_2, \ldots, \sigma_n$, and GNNC by adjusting the radius r. Here, each particle is then formed as follows:

$$X_p = [\sigma_{p1}, \sigma_{p2}, ..., \sigma_{pn}, r_p] \tag{2}$$

Where $\sigma_{p1}, \sigma_{p2}, \ldots, \sigma_{pn}$ and r_p are parameters and radius proposed by particle X_p for modifying the following fuzzy system. Briefly, each particle X_p ($p = 1, 2, ..., N$) provides a radius, r_p by which a GNNC is constructed. Then, by using the resulted clusters and the parameters $\sigma_{p1}, \sigma_{p2}, \ldots, \sigma_{pn}$ of X_p, the fuzzy system associated with X_p is obtained as follows:

$$f_p(x) = \frac{\sum_{l=1}^{M_p} Y^l [\prod_{i=1}^{n} \exp(-(\frac{x_i - (x_{cp}^l)_i}{\sigma_{pi}})^2)]}{\sum_{l=1}^{M_p} N^l [\prod_{i=1}^{n} \exp(-(\frac{x_i - (x_{cp}^l)_i}{\sigma_{pi}})^2)]} \tag{3}$$

Where M_p is the number of clusters and x_{cp}^l ($l = 1, 2, ..., M_p$) are cluster centres generated by the particle X_p.

Objective function: Consider p 'th particle X_p and its associated optimal fuzzy system $f_p(x)$. Also, assume that $\varepsilon > 0$ and M input-output pairs $(x^l, y^l), l = 1, 2, ..., M$ are given. The objective function minimized by the algorithm is defined as follows:

$$f(X_p) = \max\{\varepsilon, \max_{l=1}^{M}\{|f_p(x^l) - y^l|\}\} \tag{4}$$

Updating particle's positions and velocities: The first variables of $X_p = [\sigma_{p1}, \sigma_{p2}, ..., \sigma_{pn}, r_p]$ and V_p ($p = 1, 2, ..., N$) are updated by formulations (5).

However, we assume that $r_p \in \{d_0, 2d_0, ..., Rd_0\}$ where R is a pre-specified integer number and d_0 is selected as the minimum distances between the inputs of train data. The reason is based on the fact that different values for radius r_p may often generate the same clusters (notably, those values that are too close to each other). Therefore, considering a continuous rang for a variable r_p may ineffectively take a lot of time and energy. Consequently, $(n+1)$ 'th variable of X_p and V_p is updated by the following modified formulation:

$$\begin{aligned} (V_p)_{n+1}(t+1) &= \max\{(V_p)_{n+1}(t), (X_p)_{n+1}(t), (P_p)_{n+1}, (P^*)_{n+1}\} \\ (X_p)_{n+1}(t+1) &= \left[\frac{(X_p)_{n+1}(t) + (V_p)_{n+1}(t+1)}{2}\right] \end{aligned} \tag{5}$$

Where $(X_p)_{n+1}$, $(V_p)_{n+1}$, $(P_p)_{n+1}$ and $(P^*)_{n+1}$ denote the $(n+1)$ 'th component of vectors X_p, V_p, P_p and P^*, respectively. Moreover, $[z]$ denotes the highest integer less than or equal to z.

Due to updating mentioned above formulations (5), it is clear that in the modified PSO the optimization is obtained by movement in a search space that is continuous on the first n variables and discontinuous on the last variable.

The principle of our algorithm is shown in the following figure.

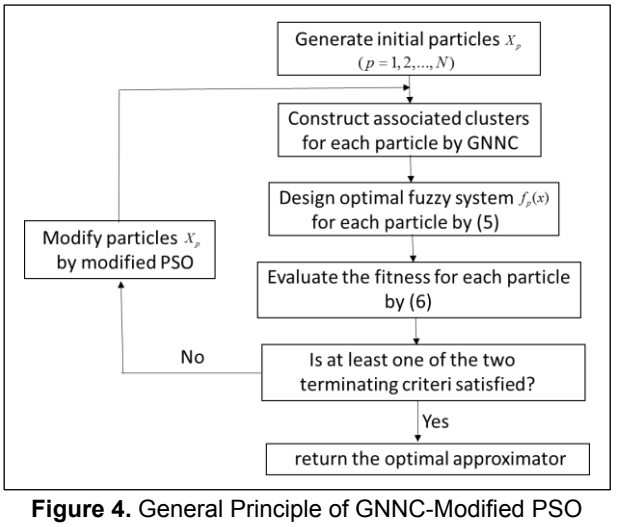


Figure 4. General Principle of GNNC-Modified PSO

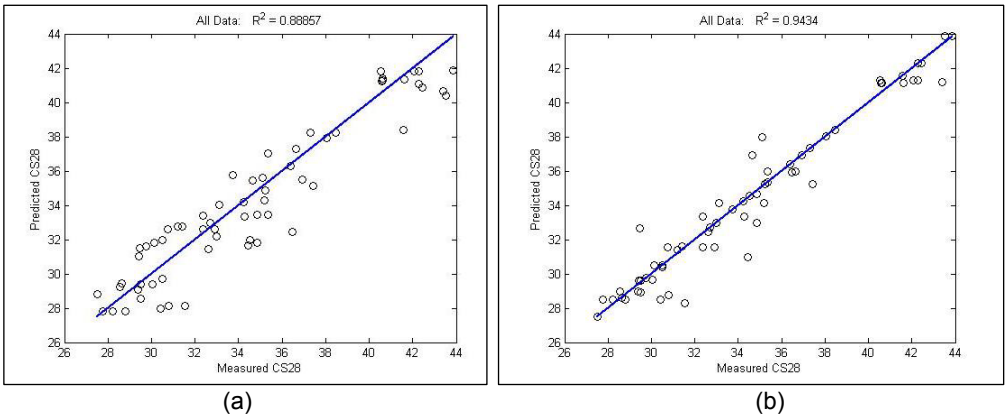
In both simple Nearest Neighbourhood Clustering Method (NNC) and GNNC-Modified PSO, an optimal fuzzy system is attained as an approximator for input-output pairs. However, there are essential differences in their strategies. Some significant differences are as follows:

1. NNC provides an approximator similar to (10) where $_1$ $_2$ ... $_n$. However, in GNNC-Modified PSO, parameters $_i$ (i $1,2,...,n$) are different, permitting the algorithm to find better approximator.
2. In NNC, clusters are only constructed according to the distribution of the input points while in GNNC-Modified PSO the distribution of both the input and output points are considered to construct the clusters.
3. In NNC the one-dimensional parameters and r are initially determined, and then the optimal fuzzy system is attained based on these values. However, in GNNC-Modified PSO the n-dimensional parameter , a one-dimensional parameter**,** and the optimal fuzzy system are simultaneously modified by lapse of iterations.
4. In NNC parameters and r are often determined by trials and errors while in GNNC-Modified PSO, these parameters are obtained by the modified PSO algorithm.

The input variables are 7-day Compressive Strength (CS_7), Pozzolan Dosages (PD) and Pozzolan Type (PT), and the output variable is 28-day Compressive Strength (CS_{28}). The results found by the proposed method are compared with the results obtained by the Table look-up scheme. Before applying the methods, we randomly choose the 70 percent of the samples as the train data and the others as the test data. For all the methods, the membership functions are considered as the Gaussian functions. In the Table look-up scheme, the numbers of the membership functions are set to 5 for all the variables. In the GNNC-modified PSO, the swarm size is set to 50, the number of iterations is 20, c_1 c_2 2 and the inertia factor w is decreasing linearly from 1 to 0.

Figure 5 illustrates the correlation between the predicted and measured 28-day Compressive Strength for the mentioned methods for all data, respectively. The figure shows the total performances of the algorithms. As this figure presents the GNNC-modified algorithm, which provides better solutions. It is due to the ability of the GNNC-modified algorithm to dynamically improve the fuzzy inference system. The optimal solution resulted from the GNNC-modified algorithm is attained as follows:

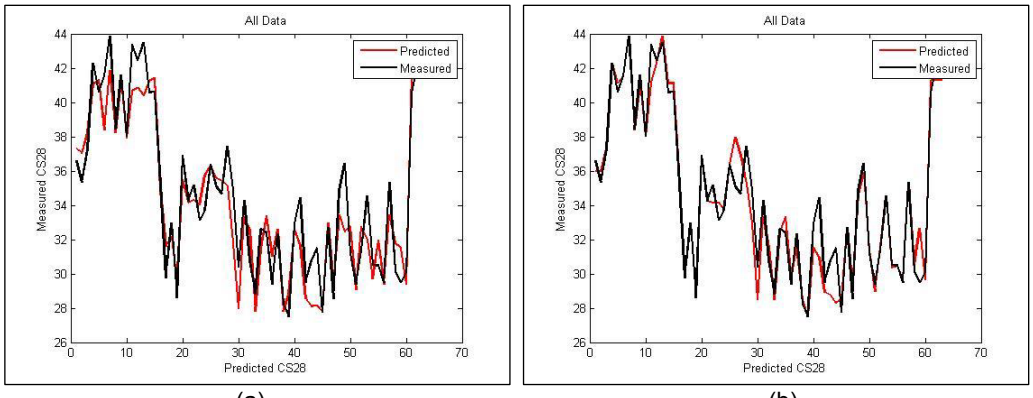
$$X_p \quad [\ ^*_1,\ ^*_2,\ ^*_3, r^*] \quad [0.640019\ ,\ 2.806706\ ,\ 0.386423\ ,\ 0.852536]$$



(a) (b)

Figure 5. Correlation Between Measured and Predicted 28-Day Compressive Strength for All the Data; (a) Results of The Table Look-Up Scheme, (b) Results of the GNNC-Modified PSO Algorithm

Moreover, the comparisons of the predicted and measured 28-day Compressive Strength (for all data) for all the methods are shown in figure 6.



(a) (b)

Figure 6. Comparison of Measured and Predicted 28-Days Compressive Strength; (a) Results of The Table Look-Up Scheme, (b) Results of the GNNCModified PSO Algorithm

CONCLUSION

1) Micro silica Pozzalans and zeolite bring about the most harmful effects for concrete slump loss, which is noticeable in a concrete mixture containing Pozzolans.
2) Except Metakaoline which its optimized amount for acquiring the highest compressive strength is 5%, other Pozzolans used in this study including micro silica, rice husk ash, and zeolite, could achieve to their optimized amount with 10% replacements by cement weight.
3) Using micro silica for achieving the highest compressive strength and utilization of the Metakaoline results in the minimum compressive strength.
4) The ratio of 7-day to 28-day compressive strength is 0.6 to 0.8, and here there is not a distinct difference between without Pozzolan specimen and Pozzolanic ones. Mixes containing 10 and 15 percent micro silica and 5 percent of Zeolite have the highest ratio.
5) The ratio of 42-day to the 28-day compressive strength of all mixes remains 1 to 1.4, and here, the mix without Pozzolan has a meaningful difference and less ratio in comparison with Pozzolanic mixes. Mixes with 10 and 15 percent of micro silica have minimum ratio after control mix and on the contrary; mixes containing zeolite have the highest one.
6) Comparing the experimental results of 28-day compressive strength with different types of prediction methods which have been used in this study, we found that generalized nearest neighbourhood clustering method has the most accurate prediction of 28-day compressive strength.

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ASSESSING PREDICTING FACTORS: GOOD MANAGEMENT PRACTICES TOWARDS THE SUCCESSFUL IMPLEMENTATION OF GREEN SUPPLY CHAIN MANAGEMENT (GSCM) IN IBS CONSTRUCTION PROJECT

Sharifah NurFarhana Syed Sahimi[1], Rohana Mahbub[2], Afzan Ahmad Zaini[1], Nadia Zaini[1] and Yon Syafni Samat[1]
[1]Faculty of Engineering, Universiti Malaysia Sarawak (UNIMAS)
[2]Faculty of Architecture, Planning and Surveying, Universiti Teknologi Mara (UiTM)

Abstract
The increasing push to use Industrialised Building System (IBS) in the current Malaysian construction industry has opened up opportunities for the implementation of Green Supply Chain Management (GSCM) in IBS to optimise the advantage of environmental sustainability and the green concept. This research aims to investigate the relationship of good management practices towards the successful implementation of GSCM in IBS construction project by adopting a quantitative method through a questionnaire survey and qualitative method to verify the responses. A total of 30 valid responses were analysed by using Regression method. The response obtained is within the IBS key players and evaluated using regression analysis. The results are validated by three (3) IBS industry experts to identify the relationship between good management practices towards successful GSCM implementation in IBS construction. Preliminary findings from the analysis found that all the management practices are significant to the successful implementation of GSCM in IBS construction. Successful implementation will lead to the investigation on how the performance of GSCM can help to digitalize the supply chain management towards the Fourth Industrial Revolution (IR 4.0) in IBS construction.

Keywords: *Industrialised Building System (IBS); Green Supply Chain Management (GSCM); management practice; successful implementation.*

INTRODUCTION

Malaysian is currently driving the implementation of a new construction method like IBS as an alternative towards sustainable and improvement of construction performance (Nawi et al., 2018; Oleiwi et al., 2017). It has stated that construction in IBS could promote sustainable construction as production related to the environment, through which waste on site can be reduced, hence protecting and conserving the surrounding environment. It is in line with the statement by Luthra et al. (2014) and Malviya (2015) who have defined green and sustainable element as integrating environmental thinking into construction. According to Luthra et al. (2014), the green supply chain management is a practice of monitoring and improving environmental performance of the supply chain in the IBS approach.

Previous studies in literature also point out that IBS technology can promote better control of resources, reduce cost and minimise construction time span (Muhammad et al., 2016). These benefits prove that the supply chain in IBS technology is a suitable platform to integrate sustainable building practices in a project through implementation of green concepts. Involvement of the Malaysian government in transforming the current construction practice into a modern construction system is supplemented by the implementation of the IBS Roadmap, the Construction Industry Master Plan (CIMP) 2007-

2015 and the Construction Industry Transformation Programme (CITP) 2016-2020. These programmes are important to improve the construction performance towards achieving vision 2020 and the Fourth Industrial Revolution (IR 4.0) whilst tremendously increasing development in the country.

SUCCESSFUL IMPLEMENTATION OF GSCM IN IBS INDUSTRY

Recently, GSCM and IBS become two of the most important performance-related issues within the construction industry. To achieve corporate sustainability within any organisation, it is essential that sustainability issues are addressed throughout the organisation's whole supply chain. The key issues of GSCM are mentioned in Figure 1.

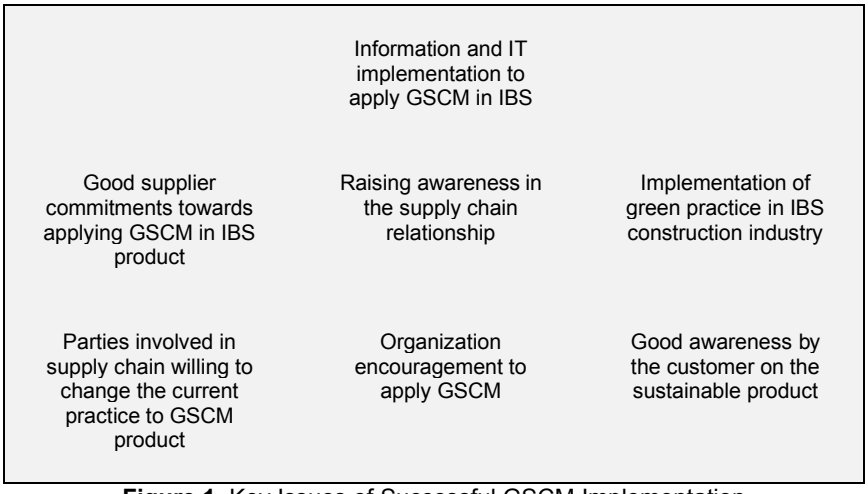


Figure 1. Key Issues of Successful GSCM Implementation

Information Technology (IT) application is a key enabler in product design which is much likely to be implemented in the construction industry. Complex construction technology can only be developed and produced by using extensive and interwoven of IT tools especially in customisation of design to fulfil the client's need especially when we react towards the Fourth Industrial Revolution (IR 4.0). This revolution is expected to change how we live, work and communicate to each other. It is also likely to change our management system especially in supply chain. Like the implementation of GSCM, IT tool like System Dynamics (SD) is important in such a way that it can help the corporate managers to improve their understanding of industrial processes (Malviya & Khan, 2015). He also added that, SD approach has been applied to GSCM as its application help in various facet such as inventory management, green supply chain design and also the information visibility in GSCM. In addition, as said by Luthra et al. (2014), the success of industrial companies depends on the interaction between the flows of information, materials, orders, money, manpower and capital equipment which can be achieved by having such a good relationship like GSCM in the system.

Apart from that, the integration of the supplier's role with various parties and other supply chain members is critically important for close cooperation within the supply chain tiers. This requires very careful definition of tasks, management of interfaces between contractors and suppliers as well as a good communication channel. It has been suggested

that by implementing an integrated approach to design and construction, the fragmentation gap could be minimised (Nawi et al., 2016). One of the approaches is through partnership between suppliers and sub- contractors from the earliest project stages, which is vital to ensure efficient and timely delivery of components and services especially in IBS construction. The other approaches are initiating good working collaboration between the design team, appoint manufacturer and project coordinator that can identify and deal with problems early and push improvement in productivity and quality.

As we know, the construction industry is one of the most significant industries that contribute towards socio-economic growth especially in developing countries like Malaysia. The industry is fragmented, unique and complex which results in problems like time overrun, cost overrun, and waste (Razak & Awang, 2014) and is regarded as one of the largest polluters to our environment. Traditional practices of construction process and management have been found to be insufficient to control unprecedented challenges including the issue of carbon emission. These challenges demonstrate the need for practitioners to improve construction technology by changing the current practice to a new and fresh approach to improve performance in the industry as a whole (Nawi et al., 2016). Thus, this is relevance to the implementation of GSCM in IBS construction with a good management practices applied.

Hussin et al. (2013) highlighted that awareness and knowledge in structural analysis and design of pre-fabricated components among civil engineers and those related to construction encourage further implementation of IBS. According to Abdullah & Egbu (2010), they have observed that most local authorities in Malaysia are unlikely to change local building regulations to comply to IBS elements due to the current level of knowledge. Thus, it is quite certain that implementation of Modular Coordination (MC) concept through the amendment of Uniform Building by Low (UBBL) can be executed with a good knowledge and awareness especially among the supply chain members (Oleiwi et al., 2017).

The IBS construction project delivery involves relationships between many organisations and processes with the evolution of many specialised roles and embedded relationships. The fragmentation and adversarial relationships among players in the IBS supply chain has been identified as the main hindrance to IBS construction project delivery (Fauzi et al., 2017). However, with a good encouragement to apply new technology like GSCM in IBS can result in greater solution in integration and collaboration within the IBS supply chains and thus can increase the performance of IBS construction in the Malaysian construction industry.

Information substitution along the supply chain will have significant environmentally beneficial influence (Luthra et al., 2014). Knowledge of green thinking related to environmental policies, technology, practices, and programmes that can be shared across the supply chain need to be emphasised and featured (Hussin et al., 2013). Not only are operations effected by knowledge, but it also influences supply chain innovation. Innovation builds on and requires knowledge, knowledge generation, and knowledge exchange. Innovation from knowledge flows are especially pertinent to smaller organisations within the supply chain which typically lack the knowledge resources related to environmental actions for their operations (Hussin et al., 2013).

As sustainability awareness rises globally, the construction industry is under increasing pressure to improve efficiency and project delivery. The implementation of Industrialised Building Systems (IBS), for which utility components are built offsite, has the potential of promoting sustainability deliverables (Oleiwi et al., 2017). This can be achieved by better control of production environment, minimising construction waste, using efficient building material energy, and stabilising work conditions.

RESEARCH METHODOLOGY

The research methodology employed is qualitative and quantitative data collection through IBS key players interview and questionnaire. The key issues of successful GSCM implementation were sourced from a wide range of related literature and verified by the IBS key players (manufacturers, suppliers, IBS designers, contractors). The ten (10) good management practices have been obtained based on experience of the IBS practitioners through semi-structured interview. It is believed that the ten (10) good management practices and selected seven (7) key successful GSCM implementation are comprehensive and significance for this research.

Sample Composition

A total of 30 questionnaires are collected representing a preliminary stage of the research. The respondents' job title, the organisation they represent and working experience in IBS project deduced that the respondents' have good knowledge on the good management practices to implement GSCM in IBS construction. The details of the respondent's' profile are summarised in Table 1.

Table 1. Profile of Respondents

Characteristics	Number (N)	Percentage (%)
Job Title		
IBS Consultant	3	10.0
Contractor	7	23.3
IBS Sales Manager	2	6.7
Director	1	3.3
IBS Supplier	11	36.7
IBS Manufacturer Manager	6	20.0
Organisation		
Construction	14	46.7
Manufacturing	6	20.0
Production	10	33.3
Services	0	0
Years of experience in IBS project		
Less than 5 years	17	56.7
5 to less than 10 years	8	26.7
10 to less than 15 years	4	13.3
15 to less than 20 years	1	3.3
More than 20 years	0	0

Questionnaire Survey

The survey was conducted among IBS key players such as IBS consultants, contractors, IBS Sales Manager, Directors of IBS engineering firm, IBS suppliers, IBS manufacturer. All 30 respondents of the survey are registered with CIDB. Most of them have less than twenty years of experience in the construction industry. The respondents were well suited for this survey for reasons of being dispersed across the industry's supply chains, being relatively experienced in the construction industry and being employed by organisations involved in green building projects.

Semi-structured Interview

Semi-structured interviews were conducted with three professionals who have experience of more than ten (10) years in IBS projects, to validate the findings of the questionnaire survey, and they include a senior manufacturer manager, IBS contractor and a director of IBS engineering firm. They are mainly involved in residential and commercial building projects, across more than twenty IBS projects in Malaysia. As a result, ten (10) list of good management practices associated with the successful application of GSCM in IBS project implementation were identified for further examination and cross referencing.

Regression Analysis

Regression analysis is a powerful statistical method that allows to examine the relationship between two or more variables of interest. The core of this analysis is to examine the influence of one or more independent variables on a dependent variable. In this research, the dependent variables are the successful implementation of GSCM in IBS construction while the independent variables are the management practices for GSCM.

RESULTS AND DISCUSSION

Table 2 shows the regression analysis for all ten (10) management practices towards the successful implementation of GSCM in IBS construction. From the analysis, factor loading for ten (10) good management practices less than 0.05 are indicates significance results.

Successful GSCM Implementation 1: Information and IT Implementation to Apply GSCM in IBS

The most significant management practices to get the information and IT implementation in applying GSCM in IBS is high commitment from top management with a factor loading of (0.015). It is followed by extensive planning and scheduling and effective communication channel among the supply chain parties with a factor loading of (0.027) and (0.034) respectively. This is agreed by the industry expert as full commitment from the top management can ease the acceptance level of a new system and technology to the existing one among the construction players. With high commitments, extensive planning and scheduling also can be done at early stage, as a good practice to introduce GSCM in IBS technology (Nawi et al., 2016). Luthra et al. (2014) added that, by practicing a good and effective communication channel among the supply chain tiers, this will stimulate workers to work effectively towards the successful implementation of GSCM.

Successful GSCM implementation 2: Good Supplier Commitments Towards Applying GSCM in IBS Product

For good supplier commitments towards applying GSCM in IBS product, the most significant management practice is good management of suppliers and logistics in supply chain tiers with a factor loading (0.018). It is followed by good strategy and business approach with (0.038) significance, high commitment from top management (0.042) and close relationship with supplier (0.049). According to the industry experts, this is relevance because in most of the projects, by adopting a good management practices with suppliers, it optimizes the flow of products, information, finances, allowing companies to create a better relationship value and improve overall business efficiency especially in IBS project.

Successful GSCM Implementation 3: Parties Involved in Supply Chain Willing to Change the Current Practice to GSCM Practice

To get the parties involved in supply chain ready to change the current practice to GSCM, high commitment from top management (0.026) is critically important. Since IBS is not a popular choice by consultant and developers due to lack of knowledge on the performance of the IBS system, the authority needs to enlighten the view of some parties that still have bad perspective on this system. We can also practice good strategy and business approach (0.030) and having a good working collaboration (0.053) that relevance to help increase the willingness for the parties involved in supply chain change the current practice to GSCM.

Table 2. Regression Analysis for Importance of Management Practices Towards the Successful GSCM Implementation

	Information and IT implementation to apply GSCM in IBS	Good supplier commitments towards applying GSCM in IBS product	Parties involved in supply chain willing to change the current practice to GSCM practice	Raising awareness in the supply chain relationship	Organisation encouragement to apply GSCM	Implementation of green practice in IBS construction industry	Good awareness by the customer on the sustainable product
Good working collaboration	0.300	0.617	0.053	0.044	0.052	0.015	0.084
Effective communication channel	0.034	0.074	0.991	0.056	0.178	0.055	0.186
Good strategy and business approach	0.135	0.038	0.030	0.791	0.777	0.730	0.056
High involvement during design stage	0.296	0.327	0.262	0.314	0.020	0.725	0.185
Environmental friendly transportation	0.708	0.094	0.080	0.841	0.047	0.970	0.391
Close relationship with supplier	0.205	0.049	0.339	0.515	0.710	0.441	0.677
Good management of suppliers and logistics	0.442	0.018	0.922	0.061	0.916	0.029	0.596
Effective industry marketing strategy	0.490	0.504	0.254	0.511	0.904	0.016	0.057

	Information and IT implementation to apply GSCM in IBS	Good supplier commitments towards applying GSCM in IBS product	Parties involved in supply chain willing to change the current practice to GSCM practice	Raising awareness in the supply chain relationship	Organisation encouragement to apply GSCM	Implementation of green practice in IBS construction industry	Good awareness by the customer on the sustainable product
Extensive planning and scheduling	0.027	0.062	0.686	0.488	0.581	0.767	0.068
High commitment from top management	0.015	0.042	0.026	0.250	0.058	0.130	0.190

Successful GSCM Implementation 4: Raising Awareness in The Supply Chain Relationship

In raising the awareness in supply chain relationship, the significant practice is by having a good working collaboration (0.044) and effective communication channel (0.056). This is added by the industry expert that, awareness is the ability of our mind to control and to make emotional free judgements about things and draw clear conclusions in creating thoughtful methods of behaviour. Thus, to raise the awareness among the supply chain tiers, being conscious is one of the clues to increase the level of knowledge in GSCM towards the implementation.

Successful GSCM Implementation 5: Organisation Encouragement to Apply GSCM

To encourage the organisation to apply GSCM, the most significant management practice is high involvement of supply chain tiers during design stage (0.020). It is followed by environmentally friendly transportation (0.047), good working collaboration (0.052) and high commitment from top management (0.058). It is reported that, besides having a full support from the top management, the encouragement to apply GSCM need to be dominant among the managements (Malviya & Khan, 2015). Additionally, the CIMP 2006-2015 illustrates the full commitment of the Malaysian Government to implement IBS to speed up construction works and reduce the number of foreign workers in the construction industry. The Government has put forward regulatory requirements and incentives in order to reduce the dependency on foreign labour and promote IBS.

Successful GSCM implementation 6: Implementation of Green Practice in IBS Construction Industry

In the implementation of green practice in IBS industry, good working collaboration (0.015), effective industry marketing strategy (0.016), good management of suppliers and logistics (0.029) and effective communication channel (0.055) are those practices that relevance towards the successful GSCM implementation. The encouragement from the Malaysian Government shows the importance of green practice in upgrading the IBS system. With proper planning and implementation, IBS would be the best solution for sustainability problems in the construction industry. Additionally, with the integration of sustainability, the best standard of quality could be attained.

Successful GSCM implementation 7: Good Awareness by The Customer on The Sustainable Product

To have a good awareness by the customer on the sustainable product, the significant practices are good strategy and business approach (0.056) and extensive planning and scheduling (0.057). With all the incentives and promotion to use IBS, clients are aware of the advantage in applying IBS as a modular coordination. The willingness of the construction parties in promoting the system is probably the main reason for high awareness on the sustainable product. This is considering a good strategy and business approach to promote GSCM in IBS construction. With a proper planning and scheduling, this can be done and easily understood by the customer on the importance of having a sustainable product in construction.

CONCLUSION

The analysis results successfully examine the relationship between the good management practices towards successful implementation of GSCM in IBS construction. Despite assessing the successful implementation of GSCM in IBS construction is deemed crucial, identification of a good management practices towards the performance is equally important. This paper has presented the good management practices to understand how the practices influence the key issues of a successful adoption of GSCM in IBS construction. Further research can apply this method to investigate how the performance of GSCM can help to digitalize the supply chain management towards Fourth Industrial Revolution (IR 4.0) in IBS construction.

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PHYSICAL MODELLING APPROACH FOR ASSESSING THE EFFECT OF CLIMATE CHANGE ON GROUNDWATER AT COASTAL AREA

May Raksmey[1] and Siti Aishah Sahul Hamid [2]
[1]Department of Civil Engineering, Faculty of Engineering, Universiti Malaysia Sarawak (UNIMAS), Kota Samarahan, Sarawak, Malaysia.
[2]Master's Degree Student at Universiti Teknologi MARA (UiTM) Shah Alam, Selangor, Malaysia.

Abstract
Seawater intrusion has brought a great deal of attention globally with the worsening effect of rising sea level and changing of climates. Global climate change which has caused the unevenness increased in extreme events such as droughts and floods has anticipated water resources to become impaired. In this study, under the influence of climate change, a shallow unconfined groundwater was laid out orderly to investigate the saltwater intrusion phenomena in coastal aquifers. Experiments were conducted using laboratory rectangular model tank filled with fresh water, well-sorted sand and salt water into three separate chambers. In spite of the fact that numerous experimental procedures have been acquired to investigate the transient development of the saltwater interface in unconfined aquifers, none of them has tackled the possibilities of applying the climate change effect. The main focus of this particular investigation is to compare the relation of recharge and rising of sea level on the behaviour of the saltwater interface in the unconfined aquifer from laboratory experiment with the normal cases of flood and the sea level rise as well as the drought and sea level rise. Simulation on the projected rising of sea level due to the climate change leads to significant intrusion outcomes of 52.4% with a toe of the saltwater wedge (TOE) distance of 0.54 m; however, an increase in the magnitude of precipitation rates and freshwater head can ultimately recharge the groundwater. This mitigates the changes of 49% in the saltwater-freshwater interface (SFI) with a receding distance of 0.22 m.

Keywords: *Climate change; saltwater-freshwater interface; sea level rise; seawater intrusion; unconfined aquifer.*

INTRODUCTION

Presently, groundwater has been a primary source to provide consumable freshwater for a prominent percentage of the global population. However, global climate change which has caused the unevenness increased in extreme events such as droughts and floods has anticipated water resources to damage. It is important to understand the outcomes of climate change as it can contribute to bettering the knowledge of predicting possible future impact on groundwater resources, especially when its quality and quantity are being underlined globally. Groundwater makes up 98% of the freshwater resources accessible for human use (Fetter, 1988).

Nearly two thirds of the global population live within 400 km of the nearshore borderline, just over half live within 200 km, an area only taking up 10% of the earth's surface (Hinrichsen, 2007). According to Zhang et al. (2002), if a seaward hydraulic gradient present, the fresh groundwater in coastal aquifers will commonly discharges into the ocean. For an unconfined aquifer, a water table occurs. The aquifer above the water table is unsaturated and moisture movement occurs, including rainfall infiltration to the groundwater. The dominant groundwater flow occurs in the saturated zone below the water table. In the final part of the aquifer seawater is introduced at the seaward. Since seawater is

more dense than fresh water, it has a tendency to obtrude into the aquifer and beat below the less dense freshwater to forge a saltwater wedge. A freshwater-saltwater interface is established once the seawater comes into contact with the fresh water. The interface is a diffusion zone, where hydrodynamic dispersion occurs (Zhang et al., 2002). The physical parameter of the aquifer and flow rate condition is contingent upon the thickness of the interface (Volker & Rushton, 1982). Normally, the mixing of the saltwater-freshwater in coastal aquifers is likewise due to the determined factors such as tidal variations, extending over a relatively long-time climate change and tidal level varieties, cracking in coastal stone shaping and dependent on a particular season change in recharge rates and evaporation (Solinst, 2012). Recharge rates can as well be brought down in countries with rapid urbanization and thus impervious surfaces (Solinst, 2012). Building drainage channels can induce the water level to decrease and encourage intrusion occurrence (Barlow, 2003). A greater extent detailed investigation on coastal groundwater and seawater intrusion has been mentioned in Glover (1959); Henry (1959) and Reilly & Goodman (1985) studies.

In previous studies, the effect of the climate change (CC) on surface water has been widely investigated compared to the effect on groundwater. Principally, the changes in groundwater recharge are because of the groundwater reaction to climate change. As mentioned by Allen et al. (2003), a good hydraulic association of waterway and aquifer storage, and lower rates of disseminating groundwater recharge are some of the conditions where the stream level has higher impact on the changes of groundwater level than the changes in diffuse groundwater recharge. Moreover, the rising of sea level may be the cause of the increasing of groundwater salinity in riparian areas. A similar pattern in groundwater recharge is predicted when the pattern in stream overflow has been diagnosed. Unfortunately, according to Kundzewicz & Doll, (2009), in the 20th century, we will face undetected groundwater recharge or groundwater level pattern, which can associate to the climate change. This is also due to the limited data availability.

Increased groundwater usage has lowered potentiometric surfaces in coastal aquifers and caused infiltration of seawater into these formations. The result has been an increased rate of salinity of the unconfined coastal aquifer. Sea level rise, whether natural or induced, has also caused increased salinity of the subterranean estuary. Some of the anthropogenic changes affected over the last 100 years are equivalent to one third of a glacier and the sea level cycle in terms of changes in the freshwater-saltwater interface in the unconfined coastal aquifer (Bobba, 1993). This study target is to elucidate the rising of sea level, direct surface recharge and tidal effects on the behaviour of the saltwater wedge patterns in unconfined aquifers through laboratory-scale experiments.

The method has been established and anticipated by the former investigators (Hong et al., 2005; Robinson et al., 2007; Li et al., 2008; Chang et al., 2011). Laboratory experiments were carried on in the rectangular tank model under well dealt conditions using red dye to trace the brine dispersion under conditions of mean sea level and rising of sea level with different nearshore freshwater flow. The results were applied to observe the combined effects of fresh groundwater flux, direct surface recharge, density-driven flow and rising of mean sea level on saltwater intrusion in unconfined coastal aquifers. The comparison simulation from pervious researcher Chang et al. (2011) was then selected to look into and verified the experiment data because of its capability to employ dissimilar combinations of boundary conditions and common formulation of the direct problem.

METHODOLOGY

Material and Tank Model Preparation

The experiments have been carried out in a rectangular tank model in the Hydraulic Laboratory of Civil Engineering Faculty in Universiti Teknologi MARA, Shah Alam. The tank is 2.1 m long and 0.60 m wide, and it was used with a working water depth of 0.75 m. It has a rigid steel bed, and all four sides of the wall were built with glass panel so that the saltwater pattern can be observed and photographed when the experiment was conducted. Figure 1 shows the side view of schematic diagram of the experimental setup in the tank. The preliminary plan is to use commercial food colour to separate two different types of water: freshwater and precipitation (direct surface recharge, W) which is colourless and saltwater (dyed red) since it has been accomplished just like in the previous researches Zhang et al. (2002); Goswami & Clement (2007) and Luyun et al. (2009). The porosity was measured as proposed in the next subsection. In order to imitate the seawater concentration, salt solutions were prepared by mixing 60 L of distilled water with 800 g of sodium chloride and 150 g of red food colour dye. In other words, 13.3 g of sodium chloride and 2.5 g of the dye was needed to dissolve in a litre of distilled water. Both seawater and saltwater concentration are assumed to have a density of 1,025 kg/m^3 and 35 kg/m^3, respectively. Meanwhile, the imitation of freshwater and precipitation was replaced with distilled water as it is assumed to have a density of 1,000 kg/m^3. Another reason is because the compositions of contaminated particles, organic and ionic are lower compared to any other concentration. From the beginning until the end of the experiment, the freshwater level was constantly persisted.

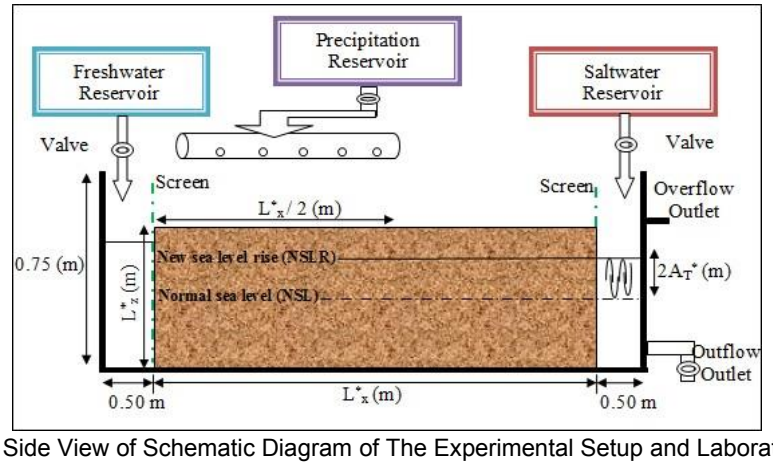


Figure 1. Side View of Schematic Diagram of The Experimental Setup and Laboratory Scale

The aquifer arrangement of the rectangular tank was divided into three separate chambers. The three chambers consist of two storage tanks of freshwater and saltwater, located at both ends. These two storage tanks functioned as landward and seaward borderline, respectively. Another one is the middle chamber, which consists of sorting sand. The middle chamber was levelled thoroughly with sorting sand to imitate an unconfined coastal aquifer. The sorting sand acts as the homogeneous porous medium. Before the sand can be levelled, sieve test analysis was carried out in order to determine the median size and the soil classification which can be obtained from the particle size distribution chart. The grain size obtained was 0.6 mm.

A set of the sprinkler system was placed at the top of the middle chamber, where the direct surface discharge flow was placed 0.30 m apart from the top layer of sand. The sprinkler pipe has 15 mm in diameter and 0.61 m long, with a nozzle. The sprinkler was connected to a rubber tube and a pump with the capacity of 0.3 HP to lift water from the precipitation storage located under the rectangular tank model. The recharge flows were chosen to be located in the middle chamber because it is expected that the saltwater wedge will occur there, two rectangular frame nets were constructed and used as a screen to strengthen the aquifer arrangement and further control the encroachment of sand from getting into other chambers. The rectangular frame was made from plywood (0.75 m × 0.60 m × 0.03 m) with a steel net attached. The green and blue colours plasticine at the three edges of the frame is used as a binder to fill the gap and stick the frame at its place.

Tests Conditions for Physical Tank Model Simulation

The experiment was first started with the distilled water filled in the rectangular tank. Then, for freshwater and precipitation storage tank, the same distilled water was used. To replicate the drought, flood and normal cases, three landward freshwaters with a height of 0.25 m, 0.40 m and 0.10 m was chosen in the left-hand-side constant head chamber. In representing a condition of normal case, flood and sea level rise case, and drought and sea level rise case, three different of direct surface water recharge from the top of the aquifer arrangement was established with a constant discharge per unit width. Those are 15 mL/min, 40 mL/min and 20 mL/min, respectively. For tidal forcing case, gravity flow was used to release the saltwater storage until it reached a constant discharge rate.

For tidal case, the new sea level rise (to simulate sea level rising due to the effect of climate change) and normal sea level (to imitate the normal condition before sea level rising), the water level was controlled with the outflow outlet by using the valve at seaward side. The flow was assumed to be constant throughout the experimental investigation for both normal sea level (NSL) and new sea level rise (NSLR) from seaward direction conditions. To simulate the rising of sea level and normal sea level condition, two different tidal elevations were chosen in the right-hand-side constant head chamber, at the height of 0.3 m and 0.25 m. From IPCC (2007), they predicted that before the year 2100, the world sea level was expected to rise by 0.1 m to 2m. Later, there was a theoretical forecast made by Vermeer & Rahmstorf (2009), mentioning that the worst-case scenario of sea level rise is double from the predicted value.

Three laboratory experiments were done according to the cases mentioned above. For each case, the experiment was run until it reaches the steady state condition. Subsequently, the observations of saltwater interface were recorded in a consecutive time period of 5 minutes, 10 minutes and 15 minutes. The red colour salt dispersion transport is then discovered by watching the movement of a clear point it moves from the right side of the sand flume meeting the physical model in the middle and end of left side of the sand flume mounted to the transparent flume (Figure 2). The colour images were then interpreted to discover the patterns that arise from the interaction of freshwater-seawater phenomenon inside the unconfined aquifer arrangement.

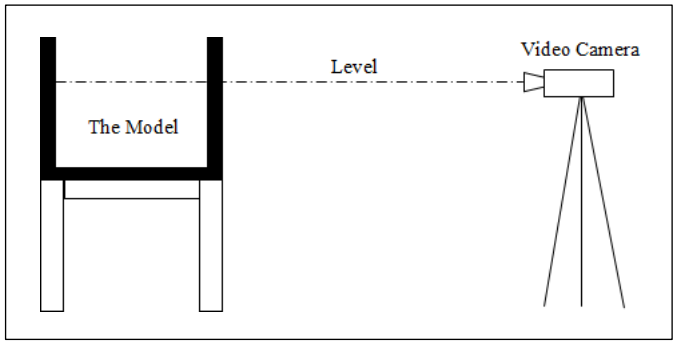


Figure 2. Sketch of The Measurement of The Saltwater Intrusion Using the Video Camera (Sony NEX-F3)

The porosity, P_o is measured of the volume of voids, V_V per total volume, V_T. The obtained value was expected to behave similarly throughout the actual flow inside the chamber. The value of hydraulic conductivity was taken out based on the successful correlation likened of d_{10}, median size grain and grain size distribution curve of this present study of sorting sand done by the previous researcher Shepherd (1989); Alyamani & Şen (1993); Odong (2007) and Salarashayeri & Siosemarde (2012). The test conditions and experimental parameters used in this study can be summarized in Table 1 and 2.

Table 1. Test Conditions Used for The Experimental Investigations

Parameter	Unit	Experiment 1 (NSL)	Experiment 2 (Flood + NSLR)	Experiment 3 (Drought + NSLR)
Saltwater elevation, H_s	m	0.25	0.30	0.30
Freshwater elevation, H_f	m	0.25	0.40	0.10
Precipitation, R	mL/min	15	40	20

Table 2. Model Parameters Used for The Experimental Investigations

Parameter	Symbol	Value	Unit
Horizontal length	L^*_x	1.10	m
Vertical length	L^*_z	0.65	m
Width length	L^*_w	0.60	m
Hydraulic conductivity	K	5.13×10^{-3}	m/s
Saltwater concentration	Cs	35	kg/m^3
Saltwater density	ρ_s	1025	kg/m^3
Freshwater density	ρ_f	1000	kg/m^3
Porosity	P_o	0.47	dimensionless

RESULTS AND DISCUSSION

Effect of Groundwater System During Normal, Draught and Flood with Sea Level Rise

The experiment was carried out by observing the patterns of saltwater-freshwater interface inside the laboratory rectangular model tanks with the influence of the hydrological cycle and rising of sea level. Three laboratory scale experiments were

established to observe the significance changes of saltwater wedge patterns due to the effect of climate change on unconfined groundwater in coastal aquifer. Figure 3 mapped intrinsically the location of the saltwater wedge patterns inside the two-dimensional experimental unconfined aquifer with the consecutively time period. The extended spatial locations of the red tracer were discovered by using Sony NEX-F3 in a high-resolution mode (Figure 2) as mentioned in the next subtopics for dissimilar experimental cases.

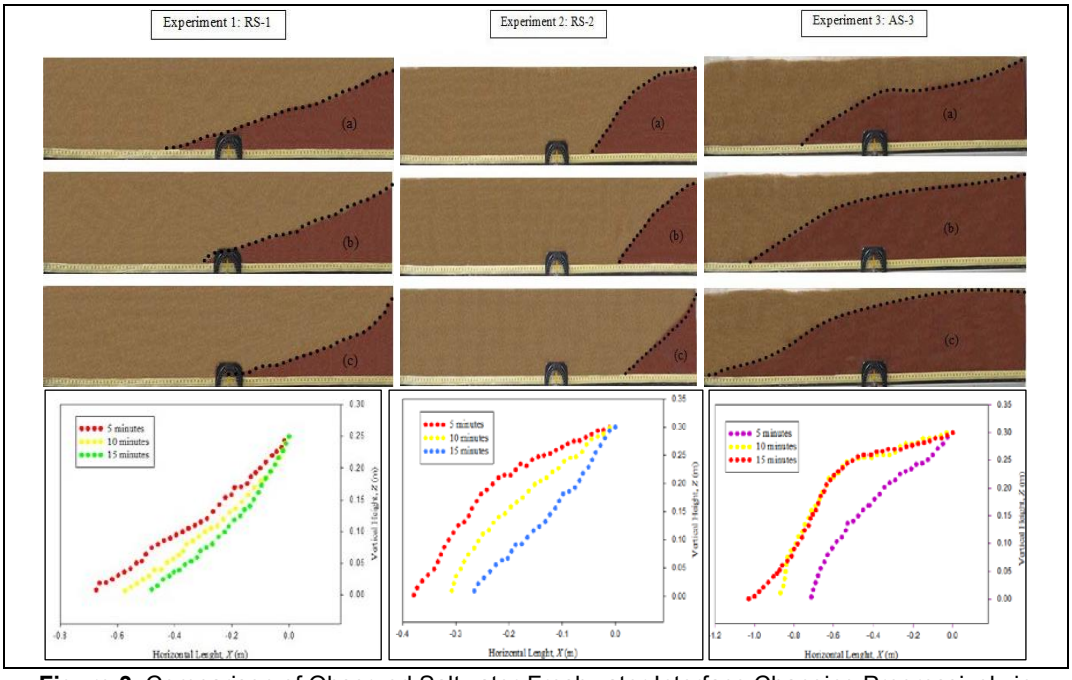


Figure 3. Comparison of Observed Saltwater-Freshwater Interface Changing Progressively in Receding (RS-1 and RS-2) and Ascending (AS-3) Manner with A Consecutive Time Interval of (a) 5 min, (b) 10 min and (c) 15 min

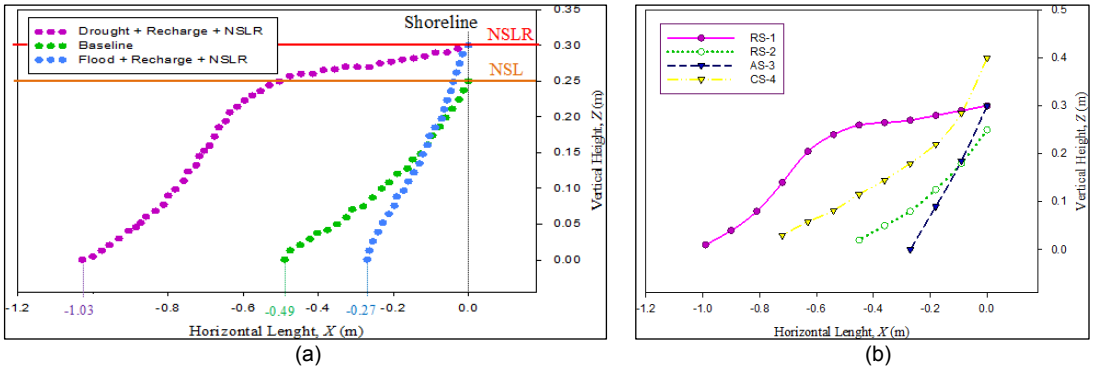
Figure 3 (RS-1) demonstrates the significant influence of the tide on seawater intrusion in the unconfined aquifer near coastal aquifer through laboratory observations. The magnitude of direct surface water recharge (R) near the coastal aquifer plays a crucial role in characterizing the flow and transport processes of saltwater wedge in the unconfined. Under normal condition of normal sea level, a constant head of freshwater ($Hf1$) was set a height of 0.25 m from inland aquifers in the left-hand side chamber. When it reached the steady state condition, a rate of 15 mL/min was introduced as direct surface recharge (R_1) above the unconfined aquifer to coastal discharge areas to the sea. Generally, a low density driven flow occurs along the aquifer with groundwater flows from areas with higher hydraulic head to areas with lower heads in the sea. Thus, Precludes saltwater from encroaching into the groundwater in coastal aquifers. Under the normal case consideration, it can be exemplified that the toe of the saltwater wedge location at 0.68 m, 0.58 m and 0.49 m with a specified length at landward from the normal sea level were established as a baseline comparison for the next two condition. The upper edge (UE) of the wedge takes over on its way at the mean shoreline throughout the aquifer for the three different experiments with a consecutive time period.

An initial condition (normal case) was run until the steady state was reached. The freshwater head ($Hf2$) at the left-hand- side chamber was then instantaneously raised to expedite the hydraulic level to push the saltwater wedge in receding front condition toward the saltwater boundary. A sea level rise was fixed at 0.05 m higher than the normal sea level. To reproduce the flood condition, sequentially a continuously uniform direct surface recharge (R_2) also was set at the greater rate of 40 mL/min. Under flood and rising of sea water level consideration, as adverted from technical report IPCC (2007), it summarized that by increasing in aquifer recharge and reservoir storage can rectify the intrusion of saltwater due to the tidal elevation of sea level into our groundwater resources in nearshore aquifer. As shown in Figure 3 (RS-2), the outcome leads the saltwater- freshwater interface to grow progressively in receding manner away from freshwater system at 15 min. The discovered mixing zone (interface) is thicker near the base of the unconfined aquifer and gets progressively thinner as the interface approaches toward the seaward. The extended toe of the saltwater wedge was recorded and measured along the horizontal direction approximately with a distance of 0.38 m, 0.31 m and 0.27 m drawback towards shoreline with 5 min, 10 min and 15 min of time interval, respectively as presented in Figure 3 (RS-2).

As demonstrated in Figure 3 (AS-3), an initial condition (normal case) was set to conceive the steady state condition. A sea level rise was set 0.05 m higher from the normal sea level. Then, the freshwater head ($Hf3$) at the left-hand-side chamber was lowered right away to hinder the gradient and reduced the freshwater discharge flow. To imitate the drought condition, sequentially a continuously uniform direct surface recharge (R_3) also was set at the lower rate of 20 mL/min. The outcome leads the saltwater-freshwater interface to grow progressively in ascending manner into freshwater systems at landward side at 15 min. The extended toe of the saltwater wedge was recorded and measured approximately at the distance of 0.72 m, 0.88 m and 1.03 m at landward with 5 min, 10 min and 15 min of time interval, respectively. This transient development of saltwater intrusion has been enunciated from previous investigation on analytical solutions, it manifested that when density-driven circulations instantaneously lowered to force a milder gradient, saltwater wedge pattern will advance more toward inland as the direct surface recharge diminishes (Chang et al., 2011).

Reference Xin et al. (2010) also noted that, more potent tidally periodic vibration flows are produced from seaward to inland with higher force of tidal level elevation in the aquifer near coastal area. This further enhances saltwater-freshwater sharp interface to widen and growth upward with a circular disseminated form.

Figure 4(a) shows the rising of sea level and higher magnitude of surface recharge, R has significant pattern variation in vertical and horizontal direction. Resulting the tendency of the toe of the saltwater wedge (TOE) to move towards the ocean and is expected to increase with a longer consecutive time period. While decreasing it surface recharge, promote the growth of TOE to move towards inland and then contaminated with high salinity into unconfined groundwater resources near the shore. Sea level rise makes existing water levels to be surpassed and low-lying coastal areas to be flooded with increasing in tidal effect magnitude.



(a) (b)

Figure 4. (a) Simulated Aquifer Response of Saltwater Wedge Patterns At 15 Min with Influence of Direct Surface Recharge (R) and Rising Sea Level (NSLR) (b) Comparison of The Observed SFI From the Experiments (Green: RS-1; Blue: RS-2; Purple: AS-3) and (Kuan et al. (2012) (Yellow: CS-4))

Comparison of Saltwater-Freshwater Interface

To compare each simulated case with Kuan et al. (2012) findings, only the maximum extent of the present interface in this study was laid out by considering the final dataset time recorded value at 15 min. These values are within the range of values reported in Kuan et al. (2012) which used a similar type of aquifer arrangement. The laboratory-scale simulations were accomplished for different freshwater elevation conditions that simulated minimum and maximum of regional groundwater flow scenarios. These freshwater elevations were chosen to simulate comparatively localization and lengthy of the saltwater-freshwater interface that encroaches deep into the aquifer with steady-state toe extending up to 0.49 m, 0.27 m, 1.03 m and 0.80 m from the seaward-side boundary under RS-1, RS-2, AS-3 and CS-4 conditions, respectively.

There were slightly different between this research and Kuan et al. (2012), where the maximal saltwater elevation for case RS-1, RS-2 and AS-3 at saltwater region (seaward side) was approximated to be about 0.35 m, which is much lower than the sea level rise recorded (0.40 m) in their findings. While the comparison of the SFI localization shows that at Z = 0.4 m, the saltwater intrusion was considerably higher than the observed SFI at Z = 0.35 m, except for case AS-3, as shown in Figure 4(b). The intrusion CS-4 surpassed about double the value presented under RS-1 and RS-2 conditions, respectively. This yield effects the withdraw on the toe of SFI which are about 39% of the sea level rise estimated for the two laboratory-scale experiments (note the experimental estimates of mean sea level rise are 0.037 m and 0.05 m). The observed localization of SFI in smaller experimental cross-sectional model results was presented in this study, as well as in Kuan et al. (2012), postulate that the differences in the saltwater-freshwater interface would depend on the effect of magnitude of direct surface recharge. When the various magnitude of direct surface recharge was introduced in this study, the mean sea level rise could be comparable to (or even be higher than) the saltwater elevation, which entails the localization of the toe of the saltwater-freshwater interface could be as far or even much farther from the freshwater region at landward side. By conducting this study, the experimental hypothesis that can be proved in a laboratory will be worthy, desirable and concerning as it can be used for field-scale observational datasets.

CONCLUSION

The phenomenon of clear saltwater-freshwater interface due to the saltwater intrusion has brought about a confirming result when the shallow unconfined aquifer is considered. The laboratory-scale experiment validates that tidal effect and precipitation increased as a variable can induce a significant decrement in saltwater intrusion in the unconfined coastal aquifer. An aquifer arrangement with a width of 0.60 m, a length of 1.10 m and height 0.65 m is able to simulate the projected rising of sea level due to climate change, which lead to significant intrusion outcomes of 52.4 % with a toe of saltwater wedge distance of 0.54 m for drought case within 15 minutes of observation.

However, as constituted for flood case, increases in the magnitude of precipitation rates and freshwater head and ultimately groundwater recharge, which mitigate these changes about 49 % in the saltwater-freshwater interface with a receding distance of 0.22 m. Comparative study to verify this study data shows yielding in withdrawal of the toe of SFI which are about 39% when R were brought into coastal aquifers. As a starting point for the present study, it can be concluded that the higher order of magnitude in direct surface recharge and constant freshwater head can be used to attenuate or eliminate the rising tide of salinity from further encroachment into our vital groundwater resources near coastal areas. It can also help prevent damage to marine ecosystems, shorelines, docks, moorings and other natural or man-made coastal structures.

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ANALYSIS OF RAINFALL EFFECT ON SLOPE FAILURE IN SABAH

Siti Jahara Matlan[1], Nazaruddin Abdul Taha[1], Salinah Dullah[1], Mohammad Radzif Taharin[1] and Hasmida Hamza[2]

[1]Faculty of Engineering, Universiti Malaysia Sabah, Kota Kinabalu, Sabah, MALAYSIA.
[2]Faculty of Engineering, Universiti Malaysia Sarawak, Kota Samarahan, Sarawak, MALAYSIA.

Abstract
The paper is aimed to characterize the relationship between the landslide events and the critical rainfall intensity-duration from the rainfall analysis. In this study, 10 landslide events that occurred in Ranau, Sabah area between years 2007 to 2015 were investigated and analysed using rainfall threshold based on soil water index (SWI) method. The analysis results showed that both short-intense rainfall (i.e., 1-day) and prolonged antecedent rainfall (i.e., 14, 30-day) played a significant role of the landslide initiations in Ranau, Sabah area. Overall results indicated that this method could simulate the rain infiltration response by the level of soil moisture in the area of landslide.

Keywords: *landslide; soil water index; slope stability.*

INTRODUCTION

In a majority of cases, the main triggering factor of landslides is heavy and prolonged rainfall. Generally, this takes by the rainfall associated with the particularly intense thunderstorm or a long duration rainfall event with lower intensity, such as cumulative effect of monsoon rainfall in Malaysia. The effect of antecedent rainfall on slope instability has been well accepted; however, the durations of antecedent rainfall suggested by various researchers differ considerably. Case studies indicate that geographical location also has an effect on the occurrence of rainfall induced slope failure (Gofar et al., 2007). Experiences in different regions of the world have resulted in different conclusions of defining the duration of the cumulative rainfall. From the foregoing, it is clear that the local rainfall pattern plays an important role in the mechanism of rainfall-induced landslide in an area. The cumulative antecedent rainfall and hourly rainfall intensity are among the basic rainfall variables (Mukhlisin et al., 2015; Ren et al., 2011).

Recently, physically-based models simulating the soil wetting dynamic response to spatial-temporal rainfall variability in complex terrain have been developed in order to enhance the predictability of shallow landslides (Capparelli et al., 2010; Greco et al., 2010; Segoni et al., 2010; Glade et al., 2000). These studies have highlighted that rainfall alone is not adequate to identify slope instability and that the initial soil moisture conditions play a significant role in the triggering of shallow landslides (e.g., Crozier, 1999; Capparelli & Tiranti, 2010; Tsai & Chen, 2010; Godt et al., 2006). Godt et al. (2006) argued that the landslide-triggering rainfall must be considered in terms of its relationship to antecedent rainfall.

With this in view, this paper aims to analyse the effect of rainfall and soil water index (SWI) to assess the critical rainfall pattern of landslide events in Ranau, Sabah area that occurred between years 2007 to 2015. The study finally characterizes the relationship between the landslide events and the critical rainfall intensity-duration from the rainfall analysis.

MATERIALS AND METHOD

Study Area

Ranau, Sabah is situated between 5°30'N to 6°25'N and 116°30'E to 117°5'E, is surrounded entirely by Kota Marudu to the north, Kota Belud to the northeast, Tuaran to the west, Tambunan to the southwest, Keningau to the south, Tongod to the southeast, and Beluran to the east (Department of Statistics, 2011). Laconically, Ranau's geography is characterized by undulating lands at most areas with a valley plain. The land use type in that area can be classified into four classes: forest, barren, built-up and agricultural area (Sharir et al., 2016). Ranau area is well known for its hilly topographic terrain and a favourite tourist spot due to its peaceful cold weather and beautiful mountain scenery.

Typically, this area is a highly landslide prone area within the country due to the hilly terrain as well as intense rainfall pattern (Figure 1). The development of hillsides in Ranau area also has caused disturbances to the characteristics of the natural slopes and consequently contributed to the landslide occurrences.



Figure 1. Landslide Prone Areas in East Malaysia (Rahmana & Mapjabilb, 2017)

The landslides in Ranau, Sabah area have been studied by several researchers (Simon et al., 2017; Tongkul, 2007; Simon et al., 2014; Rahim et al., 2017; Roslee et al., 2011). Despite of the fact that several studies have been carried out, landslide is still a recurring hazard at the study area concerned. It was either due to the proposed mitigation measures were not taken seriously by the authorities or the actual mechanism of the landslides has yet been revealed, is still unclear. Hence, rainfall characteristics should be considered in the analysis of such failure.

Landslide Data

In this study, 10 landslides that occurred in the Ranau, Sabah areas were investigated and analysed. The landslides data are obtained from the data source of JKR Sabah and also through literature surveys. Table 1 summarizes some of the landslide events that occurred in Ranau areas from 2007 to 2015.

Table 1. Date and Location of Landslide Event in Ranau, Sabah

No.	Date	Location
1.	03/10/2007	Jalan Kota Kinabalu – Sandakan (KM85.4)
2.	10/04/2011	Jalan Ranau – Kundasang (KM80)
3.	06/05/2012	Jalan Bundu Tuhan, Ranau
4.	05/11/2014	Jalan Tamparuli – Ranau (KM28)
5.	10/10/2014	Jalan Penampang –Tambunan (KM86)
6.	10/12/2014	Jalan Kota Kinabalu – Sandakan (KM91.3)
7.	01/12/2015	Jalan Kota Kinabalu – Sandakan (KM94.6)
8.	02/04/2015	Jalan Tenompok Bundu Tuhan, Ranau
9.	15/06/2015	Kampung Mesilau, Ranau
10.	29/04/2015	Jalan Himbaan Terolobou, Ranau

Rainfall Data and Analysis

Initially, a daily rainfall records were collected for this study from Ranau station through Department of Irrigation and Drainage (DID), Sabah. The rainfall patterns prior to the landslide occurrence were analysed and the soil water index (SWI) were calculated. The SWI is calculated using the tank model developed by Sugawara (1961) where the soil water index equals to the total storage volume of tanks laid vertically in series. The function of SWI is used to give an indication on the level of soil moisture in the area of landslide. A high level of SWI means that the rainfall has a significant role in destabilizing the slope.

This tank model is a simple concept that uses three tanks which are illustrated as reservoirs of a watershed that considering rainfall as the input and generate the output as the surface runoff, subsurface flow, intermediate flow, sub-base flow, as well as the phenomenon of infiltration, percolation, and water storages in the tank can be explained by the model. Figure 2 shows assumed flow patterns in tank model while Table 2 show the parameters of three series tank model used in this study.

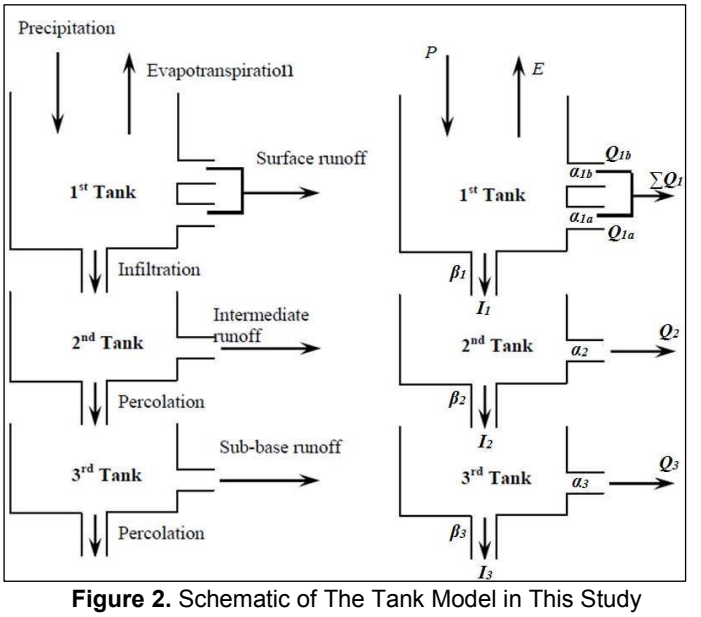


Figure 2. Schematic of The Tank Model in This Study

Table 2. Parameters of Three Series Tank Model Used in This Study (Okada et al., 2001)

Three series tank model	Height of tank holes (mm)		Coefficient of tank holes (mm)			
			Lateral		Vertical	
First Tank	H_{1b}	60	α_{1b}	0.15	I_1	0.12
	H_{1a}	15	α_{1a}	0.10		
Second Tank	H_2	15	α_2	0.05	I_2	0.05
Third Tank	H_3	15	α_3	0.01	I_3	0.01

Basically, the lateral flow discharge, $Q_i(t)$ and the vertical seepage volume $I_i(t)$ at one specific time can be calculated by the following equations:

$$Q_i(t) = \alpha(i)\,[WL_i(t)-H_i] \tag{1}$$

$$I_i(t) = \beta(i)\,[WL_i(t)-H_i] \tag{2}$$

where $WL_i(t)$ is the water table of corresponding tank, H_i is height of the lateral outlet, i is a number of lateral and vertical outlet for the respective tanks (i= *1a, 1b, 1, 2, 3*), $\alpha(i)$ and $\beta(i)$ are the lateral and vertical discharge coefficients respectively.

The SWI equals to the total storage volume of three tanks laid vertically in series. The storage volume on each tank can be calculated using the following equations:

$$\Delta S_1 = R(t)-Q_1(t)-I_1(t)$$

$$\Delta S_2 = I_1(t)-Q_2(t)-I_2(t)$$

$$\Delta S_3 = I_2(t)-Q_3(t)-I_3(t) \tag{3}$$

where $\Delta S_{1,2,3}$ is the fluctuation of water storage in tank (i.e., tank 1, 2 and 3 respectively), and $R(t)$ is the rainfall intensity at specific time.

RESULTS AND DISCUSSION

Figure 3 presents the rainfall intensity and soil water index graph of KM80 Jalan Ranau – Kundasang event (i.e., event no. 2, 10th April 2011). From the figure (Figure 3), it shows that there is a rainfall on the day of failure and also a cumulative rainfall during the last 14 days before slope failure. With regard to the SWI, the result shows that there is an increase in SWI value (i.e., 86.1 mm) on the day of failure. Besides, the SWI value of 14 days prior to the landslide incident was also higher (i.e., 135.2 mm) which was the highest SWI value recorded in the year 2011. The observation indicates that the combination of cumulative 14 days of rainfall and major rainfall on the day of landslide played a significant role in triggering the slope failure of KM80 Jalan Ranau – Kundasang on 10th April 2011. The results demonstrating that the rainfall-induced increase in pore pressure propagates mainly from the surface through the unsaturated zone, attenuate with depth and has its own velocity (Simoni et al., 2004) which is strongly dependent on the antecedent water content soil and retention curve.

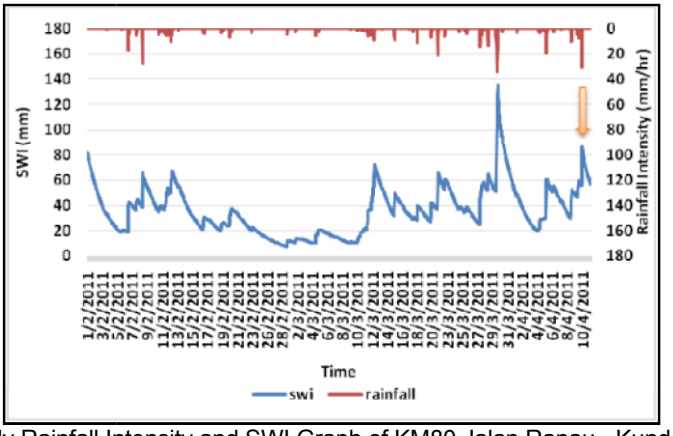


Figure 3. Hourly Rainfall Intensity and SWI Graph of KM80 Jalan Ranau - Kundasang Event (10 April 2011 – Event No. 2)

Meanwhile, observation on the result of landslide event no. 4 and 9 also shows that these landslide events was occurred during the short and intense daily rainfall with cumulative rainfall during the last 14 to 30 days before slope failure. These observations thus suggested that the amount of the antecedent and daily rainfall is the factor affecting the slope instability.

Table 3 shows the summary values of rainfall intensity and soil water index (SWI) for these three (3) landslides cases. In this case, it was concluded that event no. 2, 4 and 9 are related to the combination of antecedent (i.e., 14, 30-day) and major rainfall (i.e., 1-day). Under most circumstances, an increase in the SWI during the event indicated that the soil loss its strength due to infiltration of rainwater into the soil before it failed.

Table 3. Summary Values of Rainfall and Soil Water Index (SWI) For 3 Landslide Cases in Ranau, Sabah Area

No.	Event	Highest Rainfall Intensity Total Daily Rainfall on the Day of Failure mm/hr mm/day	Cumulative Antecedent Rainfall Prior to Landslide Event (mm)			Daily SWI (mm)	Highest Soil Water Index (SWI) Value Prior to Landslide Event (mm)		
			7 days	14 days	30 days		Within 2-7 days	Within 8-14 days	Within 15-30 days
2.	10/04/2011	30.2/50.8	90.3	264.9	454.3	86.1	60.6	135.2	72.9
4.	05/11/2014	11.5/25.6	58.4	114.9	172.9	65.7	51.4	57.0	37.2
9.	15/06/2015	13.5/17.5	42.5	50.5	143.7	46.4	34.1	12.2	59.7

*Events are related to the combination antecedent and major rainfall.

Figure 4 presents the rainfall intensity and soil water index graph of Jalan Bundu Tuhan Ranau event (i.e., event no. 3, 6th May 2012). From the figure (Figure 4), it shows that there is almost no rainfall on the day of failure but there is a cumulative rainfall during the last 30 days before slope failure. With regard to the SWI, the result shows that the level of SWI during the day of event is considered low (i.e., 25.3 mm). However, the SWI value of 15 days prior to the landslide incident was high and al so the highest SWI value recorded in the year 2012. The observation implies that the cumulative antecedent rainfall plays a significant role in destabilizing the slope and thus is the dominant triggering factor of the landslide occurrence at Jalan Bundu Tuhan Ranau on 6th May 2012. This result illustrates

that both the matric suction and factor of safety decreased steadily overtime until they reached the lowest values on the day of landslide occurrence. This situation also shows that a prolonged and continuous low intensity rainfall is occurred during these periods as it could result in a relatively greater amount of infiltration and eventually cause the slope more susceptible to failure (Lee et al., 2014).

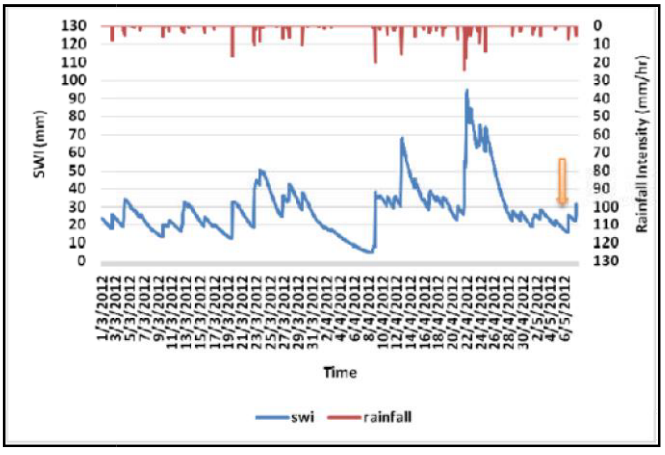


Figure 4. Hourly Rainfall Intensity and SWI Graph of Jalan Bundu Tuhan Ranau Event (06 May 2012 – Event No. 3)

Meanwhile, observation on the result of event no. 1, 5 - 8 and 10 also shows that these landslide events did not occurred during the highest daily rainfall. These observations thus revealed that the amount of the daily rainfall is not the factor affecting the slope instability. In the other words, the prolonged and continuous rainfall could play a role in building up the mechanism of landslide.

Table 4 shows the summary values of rainfall intensity and soil water index (SWI) for these seven (7) landslides cases. In this case, it was concluded that event no. 1, 3, 5 - 8 and 10 are related to the prolonged antecedent rainfall (i.e., 14, 30-day). The result shows that attention and cautious need to be done in relation to prolonged and continuous rainfall occurs within 14 to 30 days, in which it can cause the SWI value to increase while weakening the slope slowly, and consequently trigger landslides to occur at any time within that's.

Table 4. Summary Values of Rainfall Intensity and Soil Water Index (SWI) for 7 Landslides Cases

No.	Event	Highest Rainfall Intensity Total Daily Rainfall on the Day of Failure mm/hr mm/day	Cumulative Antecedent Rainfall Prior to Landslide Event (mm)			Daily SWI (mm)	Highest Soil Water Index (SWI) Value Prior to Landslide Event (mm)		
			7 days	14 days	30 days		Within 2-7 days	Within 8-14 days	Within 15-30 days
1.	03/10/2007	0	31	36	117.5	11.1	32.1	6.4	63.9
3.	06/05/2012	7.5/10.5	33	169.9	310.8	25.3	28.4	75.0	94.7
5.	10/10/2014	1	15.5	39.5	171.5	15.6	15.2	40.3	67.2
6.	10/12/2014	0	48	90.00	166.9	18.6	44.2	48.9	55.0
7.	01/12/2015	0	80	91.5	193.5	25.4	56.4	14.9	70.9
8.	02/04/2015	0	50	52	70.5	9.4	36.6	46.4	20.2
10.	29/04/2015	0	13	45	91.5	12.4	19.1	34.0	30.2

*Events are related to the prolonged antecedent rainfall.

CONCLUSION

Based on the analysis of rainfall data (i.e., Ranau rain gauge station), it is concluded that the critical duration of rainfall influencing the occurrence of landslides in Ranau, Sabah area can be categorized into two (2): (1) Landslides are triggered by the combination of antecedent and major rainfall and (2) Landslides are triggered by prolonged antecedent rainfall. These conclusions are supported by previous research (Mukhlisin et al. 2015; Lee et al., 2014; Jemec & Komac, 2011) which they found that both short-intense rainfall and prolonged antecedent rainfall were responsible for the landslide initiations in Malaysia. Overall results indicated that this method could simulate the rain infiltration response. The SWI method was demonstrated that the landslides were triggered by a down going wetting front (i.e., by the level of soil moisture in the area of landslide). However, separate analysis of the effect of slope parameters on the estimation of landslide is desirable and further studies are in need in this regard. Nevertheless, the results presented serve to point out the importance of landslide triggering by rain infiltration within a simple framework.

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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.

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CODIFICATION AND APPLICATION OF SEMI-LOOF ELEMENTS FOR COMPLEX STRUCTURES

(FULL NAME) Ahmad Abd Rahman[1,2], Maria Diyana Musa[2] and Sumiana Yusoff[2]
[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)

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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.

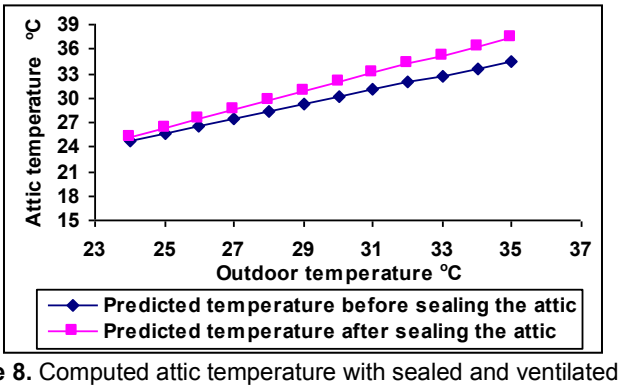


Figure 8. Computed attic temperature with sealed and ventilated attic

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)

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

References should be listed in **alphabetical order**, on separate sheets from the text. In the list of references, the titles of periodicals should be given in full, while for books should state the title, place of publication, name of publisher, and indication of edition.

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

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

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

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

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

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