SEISMIC DESIGN GUIDES FOR LOW-RISE MASONRY BUILDINGS IN MALAYSIA

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A project report submitted in partial fulfilment of the requirements for the award of Bachelor of Engineering (Hons.) Civil Engineering

Lee Kong Chian Faculty of Engineering and Science
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MAY 2016

DECLARATION

I hereby declare that this project report is based on my original work except for citations and quotations which have been duly acknowledged. I also declare that it has not been previously and concurrently submitted for any other degree or award at UTAR or other institutions.

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APPROVAL FOR SUBMISSION

I certify that this project report entitled **"SEISMIC DESIGN GUIDES FOR LOW-RISE MASONRY BUILDINGS IN MALAYSIA"** prepared by **CHERRIE LIEW QIANYI** has met the required standard for submission in partial fulfilment of the requirements for the award of Bachelor of Engineering (Hons.) Civil Engineering at Universiti Tunku Abdul Rahman.

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Date : 11 MAY 2016

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SEISMIC DESIGN GUIDES FOR LOW-RISE MASONRY BUILDINGS IN MALAYSIA

ABSTRACT

Unreinforced masonry panels are commonly used as internal or external walls in reinforced concrete building structures. Traditionally, infilled masonry has been used without taking into consideration as a load-bearing panel in Malaysia. External walls, as well as internal walls are subjected to the same seismic ground motion, and have to be designed to meet seismic requirements. According to Eurocode 6 and Uniform Building By-Laws 1984, a wall configuration was proposed. The response spectrum in the Malaysia National Annex draft, and finite element modelling (with linear elastic analysis method) with SCIA Engineer software were employed in this study. Masonry wall failure comprised of both in-plane failure and out-of-plane failure modes. Out-of-plane failure was found to be the controlling failure mode in this study. This study presented several influencing parameters helpful for the design of infilled masonry walls. Eurocode 8 is the additional rules to Eurocode 6. By taking into consideration the requirements in Eurocode 8, the proposed wall dimensions were verified. A few geometric requirements were proposed for the future design works in Sabah, namely the maximum height of wall panels of 2.7 m, the minimum masonry wall thickness of 200 mm and the minimum aspect ratio of 0.6, as base case. Several graphs were also presented for a preliminary seismic assessment of existing wall panels in Malaysia.

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LIST OF SYMBOLS / ABBREVIATIONS

a_g	design acceleration on type A ground, m/s^2
a_{gR}	reference peak ground acceleration, m/s^2
E	Young's modulus, Pa
g	acceleration of gravity, m/s^2
h	height of wall panel, m
h_{ef}	effective height of wall, m
L	length of wall panel, m
P	probability of exceedance, %
PGA	peak ground acceleration, g
PGV	peak ground velocity, mm/s
RP	return period, years
RSA_{max}	maximum peak ground acceleration, g
RSD_{max}	maximum peak ground displacement, mm
RSV_{max}	maximum peak ground velocity, mm/s
T	structural period, s
T_C	first corner period, s
T_D	second corner period, s
t_{ef}	effective thickness of wall, mm
γ_1	importance factor
$\sum G_k$	characteristic value of a permanent action
$\sum Q_k$	characteristic value of a single variable action
ACEM	Association of Consulting Engineers Malaysia
ATC	Applied Technology Council

BEM	Board of Engineers Malaysia
BS 5628	Code of Practice for the Use of Masonry - Materials and Components, Design and Workmanship
BS 8110	Structural Use of Concrete
BSI	British Standards Institution
DL	damage limitation
EN 1996	Design of Masonry Structures
EN 1998	Design of Structures of Earthquake Resistance
IEM	Institution of Engineers Malaysia
IEP	initial evaluation procedure
JKR 20800-132-23	Brickworks Standards in Malaysia
MS 29	Specification for Aggregates from Natural Sources for Concrete
MS 522	Portland Cement (Ordinary and Rapid Hardening)
MS EN 1990	Basis of Structural Design
MS EN 1991	Actions on Structures
MS EN 1991-1-1	General Actions - Densities, Self-Weight, Imposed Loads for Buildings
MS EN 1992	Design of Concrete Structures
MS EN 1992-1-1	General Rules for Structures
NC	near collapse
RC	reinforced concrete
RSP	rapid screening procedure
SD	significant damage
UBBL	Uniform Building By-Laws 1984
USGS	United States Geological Survey
UTM	Universiti Teknologi Malaysia

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CHAPTER 1

INTRODUCTION

1.1 Background

Malaysia is generally perceived as an earthquake-free zone. In recent years, earthquakes and tremors appeared to be experienced much more frequently in Malaysia. These earthquake tremors in Malaysia are mainly originated from local earthquakes, and distant earthquakes from Sumatera with the two major sources of Sunda Arc subduction fault zone source off-shore of Sumatra and Sumatran strike-slip fault source (Looi, et al., 2013). Regional earthquake zone extends from outside geographical boundaries to within the state boundaries. In Peninsular Malaysia, there were a series of weak earthquakes in Bukit Tinggi from year 2007 to year 2009 which were attributed by the Bukit Tinggi fault. As for the local earthquakes in Sarawak and Sabah, the faults were identified at Tubau and Kelawit, and Mensaban, Perancangan, Lahad Datu, Keningau, Danum, Binuang, Tabin and Beluran respectively (Yan, 2010).

1.2 Problem Statement

Since Malaysia lies outside the zone popularly known as Pacific Ring of Fire, where a large number of earthquakes and volcanic eruptions frequently occur, large magnitude earthquakes have not been reported to occur here. However, in the earthquake incident in Sabah (June 2015), many buildings were damaged and eighteen people were killed by the rock-falling near the peak of Mount Kinabalu. According to a news portal on the internet, The Malay Mail Online, this earthquake has set the record of being the strongest earthquake to have hit Malaysia in thirty-nine years since the year of 1976 (The Malay Mail Online, 2015). Since then, this incident has been a wakeup call for Malaysia engineers, to motivate the authorities to implement seismic design codes on buildings, especially buildings in Sabah.

Upon the implementation of Eurocodes, Eurocode 8 will provide guidance for seismic design of new buildings in Malaysia, particularly in seismically active Sabah. Since most buildings in Sabah are low-rise masonry buildings, Eurocode 6 for masonry, in addition to Eurocode 2 for concrete, should also be implemented in Sabah too. However, currently structural designers are generally unfamiliar with these Eurocodes to carry out seismic design with both Eurocodes. Moreover, the predecessor of Eurocode 6, namely BS 5628, has not even been widely adopted and applied in Malaysia.

In addition, Eurocode 6 covers unreinforced masonry, reinforced masonry, confined masonry and prestressed masonry. Confined masonry construction is commonly practised in many developing countries such as Italy, Mexico, Chile, Peru, Indonesia and China. This method is widely applied in low-rise residential buildings. Confined masonry construction allows in-plane resistance to be mobilized to resist horizontal seismic action. Malaysia and many developing countries are using “infilled masonry”, where the wall is just subject to permanent action for non-seismic designs. Depending on the detailing to interact with reinforced concrete frames, infilled masonry can exhibit as a “confined masonry” to resist horizontal seismic forces. For new masonry design, engineers should take advantage of this structural feature to resist the seismic actions.

The Eurocodes are expected to be implemented in Malaysia with a recommended transition period of three years. Until today, local reinforced concrete buildings are generally still designed according to British Standards such as BS 8110 and prevailing Uniform Building By-Laws 1984 (UBBL), without specific seismic design requirements and without taking into account seismic actions. Seismic assessment to determine the performance of existing masonry buildings, especially in

Sabah, has become the first priority. Retrofitting of those highly vulnerable buildings can then be identified and considered, and executed in other future studies.

1.3 Aims and Objectives

The aim of this study is to reduce the damages caused during earthquake tremors in Malaysia, particularly in Sabah. This study covers the assessment for existing masonry buildings and design for future masonry buildings design.

For future buildings, a few simple influencing parameters requirements are proposed for designing new masonry buildings, that are efficient to resist seismic actions. The proposed parameters can be applied in seismic performance assessment of existing masonry buildings. Charts are developed to aid designers at preliminary assessment work.

1.4 Scope of Study

The scope of study includes the application of structural engineering, design and analysis software such as SCIA Engineer and Microsoft Excel spreadsheet. This study included finite element modeling of reinforced concrete frames with masonry infills under seismic action. Simplified calculation is also implemented. Likewise, Microsoft Excel and simple calculations proposed aim to aid in further studies of seismic performance assessment for retrofitting of existing buildings.

This research has drawn on reports of damage caused by the recent Ranau earthquake. Appropriate assumptions (adopted from numerous previous experimental researches) have also been made throughout this study to ensure that the proposed parameters are practical when applied.

1.5 Previous Work

UTAR's Bachelor of Engineering (Hons.) Civil Engineering Final Year Project students have conducted several seismic researches, such as the dynamic properties of the residual soil in Malaysia with shaking table test and numerical modeling of the interaction effect between reinforced concrete frame and infilled masonry.

1.6 Significance of Research

The outcome of this research served as a reference for further studies of design guideline for new buildings and assessment guideline for existing low-rise masonry buildings. This research mainly learnt from the damages reported in recent Ranau earthquake in Sabah, thus, this research will assist Malaysian engineers in the design and retrofitting jobs (especially in Sabah) in the future.

1.7 Layout of Report

In Chapter 1 Introduction, the earthquake condition in Malaysia is introduced. Problem statements, aims and objectives of study, scope of work, previous work and significance of works are also discussed.

In Chapter 2 Literature Review, the seismic actions in Malaysia are discussed. Failure modes of masonry walls and influencing parameters of masonry wall under seismic actions, just to name a few, are also discussed in this chapter. This information is gathered from suitable standards, articles and journals.

Chapter 3 Methodology describes the workflow of project. Besides, the configuration of masonry panel, design response spectrum, type of analysis and software adopted in this study, is discussed too.

Chapter 4 Results and Discussion further discusses the results obtained from SCIA Engineer software. For new masonry design, several graphs are plotted, and by comparing with Eurocode 8 (considering seismic requirements), interpolation is done to obtain the appropriate configuration of new masonry walls, as base case. As for existing masonry, the plotted graphs function as a preliminary check for seismic performance assessment.

Last but not least, Chapter 5 Conclusion and Recommendations ends this study with the conclusions with respect to the aims and objectives of this study. Suitable recommendations are proposed too.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview of Eurocodes

Eurocodes are basically a set of standards with common rules of structural design within the European Union. There are several specific clauses with left open parameters in Eurocodes, and these clauses are to be filled up in the National Annex of each country by adapting to the country's social and economic situations.

In April 2014, the Institution of Engineers Malaysia (IEM), the Board of Engineers Malaysia (BEM) and the Association of Consulting Engineers Malaysia (ACEM) have initiated a proposal to implement Eurocodes with a recommended transition period of three years. Thus, these new codes will soon replace the conflicting British Standards, which have been withdrawn by British Standards Institution (BSI) since 31 March 2010.

2.1.1 Overview of Eurocode and Eurocode 1

MS EN 1990: "Basis of Structural Design" is often known as Eurocode or EC 0. It is an essential code as it introduces the basic principles and requirements for the safety, serviceability and durability of structures. It is intended to be applied with other Eurocodes for the structural design of buildings and civil engineering works.

The Malaysia National Annex of Eurocode was prepared by the Institution of Engineers Malaysia (IEM) Technical Committee on Code of Practice for Design of Concrete Structures. It basically provides the nationally determined parameters such as the basic requirements (indicative design working life), basic variables (design values of actions and factors of actions) etc.

Eurocode 1, MS EN 1991: “Actions on Structures”, is divided into ten parts. The most commonly adopted code is part 1-1 (BS EN 1991-1-1: “General Actions – Densities, Self-Weight, Imposed Loads for Buildings”). As its title named, this code and its Malaysia Annex provide density of different materials, which is subsequently used to calculate the self-weight (permanent actions) of buildings.

2.1.2 Overview of Eurocode 2 and Eurocode 6

Eurocode 2, MS EN 1992: “Design of Concrete Structures”, will soon be replacing the existing BS 8110 in Malaysia. This code complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification of Eurocode. It is mainly applied to the design of buildings and civil engineering works in plain, reinforced and prestressed concrete. This code is intended to be used in conjunction with Eurocode 8 for design of plain, reinforced and prestressed concrete structures which are built in seismic regions.

Eurocode 6, denoted in general by EN 1996: “Design of Masonry Structures”, applies to the design of buildings and civil engineering works, or parts in unreinforced, reinforced, prestressed and confined masonry. It consists of four documents: rules for reinforced and unreinforced masonry, structural fire design, selection and execution of masonry and simplified calculation methods for unreinforced masonry structures.

In Malaysia, majority of the masonry walls are designed and functioned as partitions to divide the space. Engineers often ignore the structural resistance of these infill walls in structural design. These walls, if poorly-detailed and executed, will perform poorly during earthquakes. In actual fact, masonry walls should be considered as structural elements during the design process. It would be a good practice, especially in Sabah, as most buildings in Sabah are low-rise masonry buildings. For seismic design of low-rise masonry buildings in Malaysia, it would be appropriate to draft the design guidelines based on Eurocode 6 and Eurocode 8, in addition to Eurocode 2 and UBBL.

2.1.3 Overview of Eurocode 8

MS EN 1998: “Design of Structures for Earthquake Resistance”, provides guidelines of design and execution of buildings and civil engineering works in seismic regions. The main purpose of Eurocode 8 is to protect human lives, to limit damage and to ensure that the structures for civil protection remain operational after earthquake. Thus, the fundamental requirements for design and execution are basically no-collapse and damage limitations.

One of the expected significant changes after the implementation of Eurocodes is the replacement of BS 8110 by MS EN 1992-1-1 for structural concrete buildings. For seismic design of low-rise masonry buildings, new design will need to comply with Eurocode 8 Clause 9 (Specific Rules for Masonry Buildings) and the future Malaysia National Annex. As for existing buildings, assessments should be done to identify the need of retrofitting.

2.2 Seismic Activities in Malaysia

Referring to Figure 2.1, which is a figure modified from United States Geological Survey (USGS), Malaysia is geographically located outside of the Ring of Fire. The red dots define the Pacific Ring of Fire, forming along the tectonic plate boundaries. Thus, Malaysia is perceived as an earthquake-free zone. Despite this, Malaysia has been experiencing a series of earthquake tremors in both East and West Malaysia

(Peninsular Malaysia). In the year of 2007, approximately twenty-four tremors of magnitude 0.3 - 4.2 have been documented in Bukit Tinggi, Pahang. In the year of 2009, places such as Jerantut in Pahang, Manjung in Perak and Kuala Pilah in Negeri Sembilan have also reported to have experienced earthquakes. In June 5, 2015, another large earthquake struck Ranau, Sabah. Yet, this earthquake was not the highest record ever, the deadliest earthquake was in Lahad Datu with the magnitude of 6.2 back then in year 1976 (Earthquake Track, 2015).

Figure 2.1: The Location of the Ring of Fire

The Ranau earthquake has been a cause célèbre recently. Sabah, located on the southeastern edge of Eurasian Plate, which is bordered by the Philippine Plate and the Pacific Plate, is currently receiving compression forces from the interaction of three main tectonic plates. According to Universiti Malaysia Sabah's geologist, Dr. Felix Tongkul (2015), the Philippine Plate and Pacific Plate are moving westwards at a rate of about 10 cm a year, colliding with the Eurasian Plate. The focus of this earthquake was approximately underneath the peak of Mount Kinabalu.

The Ranau earthquake was reported to have caused severe damage such as building cracks, road cracks, rockfalls and mudslides. This earthquake in Sabah was estimated to cost approximately RM100 million of damages (Mariah, 2015). Thus, East Malaysia, especially Sabah has been perceived to be a moderate seismic region.

2.3 Brickworks Standards in Malaysia (JKR 20800-132-23)

The cement used is Ordinary Portland Cement which complied with MS 522 while sand complied with MS 29. The suggested proportion is one part of cement to six parts of sand, and with the addition of approved mortar plasticizer, to increase the strength of mortar yet maintaining the workability of fresh mortar.

As for bricks and blocks, there are various types of bricks and blocks, ranging from clay bricks, cement sand bricks and hollow blocks, autoclaved aerated concrete block to patented block. Clay bricks are the most common bricks. As for cement sand bricks and hollow blocks, these consist of a mixture of sand and cement at a ratio of six parts of sand to one part of cement. The clay bricks and hollow blocks should at least have a compressive strength of 5.2 N/mm^2 and 2.8 N/mm^2 respectively. Autoclaved aerated concrete blocks, consist of a mixture of ordinary cement, sand and lime, and are high pressure steam cured.

2.4 Application of Eurocode 8

Eurocode 8’s purpose is to ensure that during earthquake, human lives are protected, damage is limited and structures important for civil protection remain operational. Besides, the fundamental performance requirements of structures designed and constructed are no-collapse and damage limitation requirements. The probability of exceedance and mean return period for Eurocode 8 and Malaysia National Annex draft varies as shown in Table 2.1.

Table 2.1: Probability of Exceedance and Mean Return Period for Eurocode 8 and Malaysia National Annex Draft

Requirements	No-collapse Requirement		Damage Limitation Requirement	
	Eurocode 8	Malaysia National Annex Draft	Eurocode 8	Malaysia National Annex Draft
Probability of Exceedance, P	10 % in 50 years	2 % in 50 years	10 % in 10 years	10 % in 50 years
Mean Return Period, T	475 years	2475 years	95 years	475 years

For no-collapse requirement, with the reference return period of four hundred and seventy-five years, a 1.0 factor, importance factor is assigned when linear analysis is performed. The design ground acceleration on Type A ground calculation is as shown in Equation 2.1.

$$a_g = \gamma_1 a_{gR} \tag{2.1}$$

where

a_g = design acceleration on Type A ground, m/s^2

γ_1 = importance factor

a_{gR} = reference peak ground acceleration, m/s^2

When the return period varies from four hundred and seventy-five years, the importance factor varies according to the importance classes (importance for public safety and civil protection, and consequences of collapse).

In Eurocode 8, national territories are subdivided into seismic zones according to the local hazard. The parameter used to describe the hazard is the reference peak ground acceleration on Type A ground. Type A ground is basically ground with high average shear wave velocity, and rock or other rock-like geological formation with at most 5 m of weaker material from the surface. The characterization of seismic zones is summarized in the table below. The peak ground acceleration (PGA) for both Peninsular and Sarawak with four hundred and seventy-five years return period (RP) is 0.07 g, whereby indicates that both Peninsular Malaysia and Sarawak are in low seismicity zone.

Table 2.2: Characterization of Seismic Zones according to Eurocode 8

Seismic Zones	PGA with 475 Years Return Period	
	Bed Rock	**Soil**
Very low	< 0.04 g	< 0.05 g
Low	< 0.08 g	< 0.1 g
Moderate to High	> 0.08 g	> 0.1 g

2.4.1 Seismic Actions in Malaysia

Earthquake actions adopted for design purposes are based on the maximum acceleration response of the structure under earthquake motions. An elastic response spectrum is constructed after conducting studies on the potential peak ground acceleration in a specific seismic-affected region. This elastic response spectrum is taken as being the same shape for both no-collapse and damage limitation requirements. The elastic response spectrum is as shown in Figure 2.2.

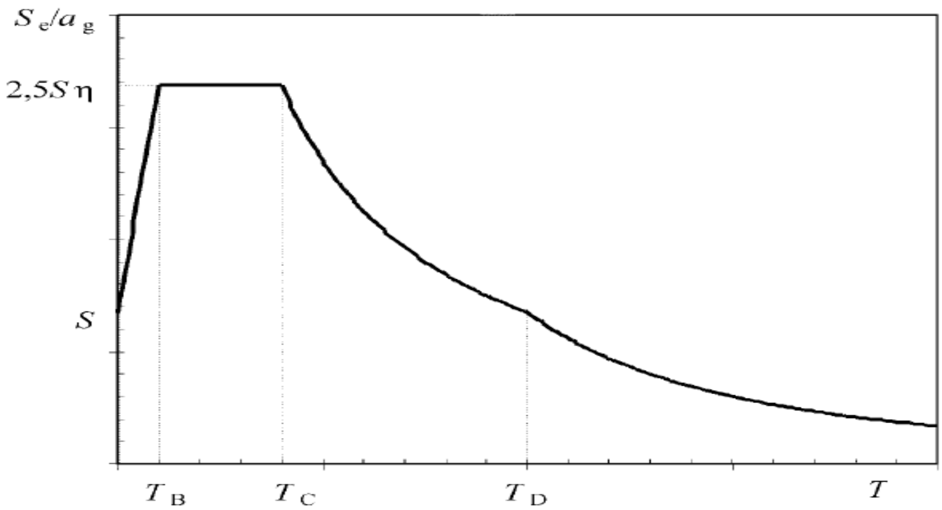

Figure 2.2: Shape of the Elastic Response Spectrum

The values of T_B, T_C and T_D and S are to be found in National Annex. A behaviour factor is applied to take into account the influence of 5 % viscous damping. In Malaysia, a hybrid response spectrum model (Figure 2.3) was formed by combining both far field and local earthquake hazards. Thus, the long period range, t > 2 s ($T > T_C$ in Figure 2.2) is controlled by the considerations of distant earthquakes while the short period range of t < 1 s ($T < T_B$ in Figure 2.2) is controlled by the local earthquakes. For the transition period of 1 s to 2 s ($T_B < T < T_C$ in Figure 2.2), the straight line in the graph is caused by the assumption of 5 % viscous damping made.

According to the article in IEM's monthly bulletin, Jurutera of the month of April 2013, the recommended hybrid model is as shown in Figure 2.3. It is also recommended that the reference Response Spectral Acceleration (RSA) for Peninsular Malaysia and Sarawak is 0.10 g while the RSA for Sabah is 0.18 g (Looi, et al., 2013).

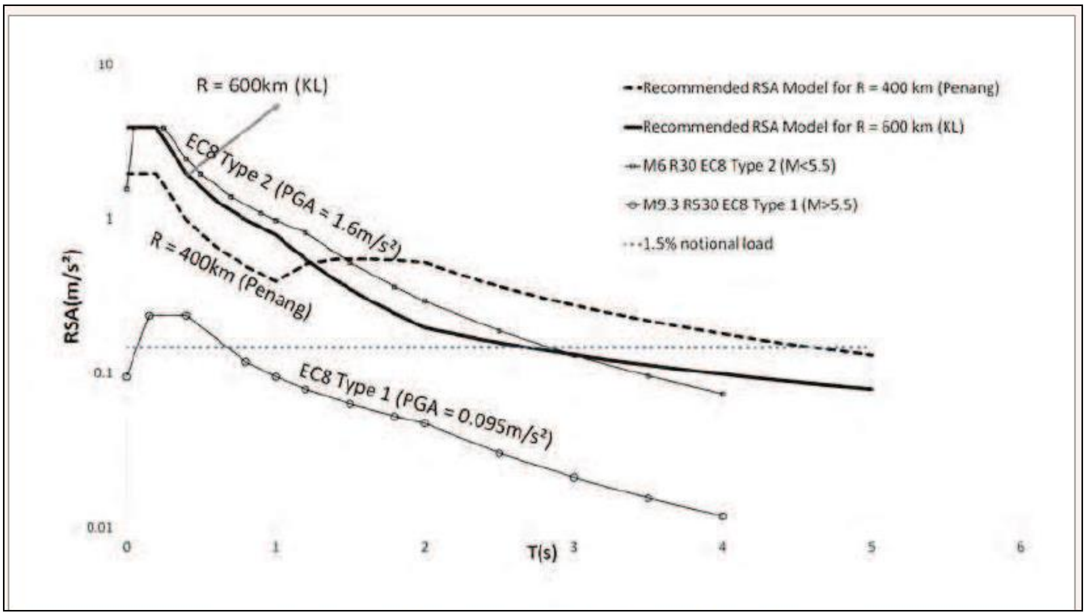

Figure 2.3: Recommended Hybrid Response Spectrum Model (Looi, et al., 2013)

2.4.2 Structural Analysis in Eurocode 8

There are two method of analysis in this code, which are linear-elastic analysis and non-linear analysis. For linear-elastic method, lateral force method is applied when the building is not considerably influenced by the contributions from modes of vibration higher than the fundamental mode in each principal direction, while modal response spectrum analysis is the substitute of lateral force method when lateral force method could not be applied. As for non-linear analysis, static (push over) analysis and dynamic (time-history) analysis are available.

2.5 Masonry Construction Systems

Masonry systems are divided into two types: load-bearing and non-load-bearing, depending on the types of construction. To classify them, material used and construction scheme used are considered. In Malaysia, it is a common practice for walls to be designed as non-structural elements, contributing only permanent actions to the structure.

2.6 Unreinforced Masonry Infill Panels

Unreinforced masonry structures are the conventional construction systems commonly used in Malaysia. This system basically consists of solid or hollow clay brickwork, using mortar as bonding layers. Unreinforced masonry infill panels are widely treated as non-structural members, and are used as interior partitions and non-load bearing external walls. This is primarily due to the complexity on the modelling and designing of infill walls as structural elements.

Masonry infill is constructed after constructing the frame (columns and beams). As such, unreinforced masonry structures are typically brittle in nature and do not perform well in earthquakes, causing the structures to be vulnerable to damage and collapse. Masonry units are constructed using bonding arrangements to obtain a single structural element. Thus, good workmanship and construction practices should be adopted.

In the event of an earthquake, ground motions cause inertia forces to be generated in the structural mass, and they react on the structural components to cause deformation and damage. Unreinforced masonry walls normally fail via in-plane and/or out-of-plane mechanisms.

2.6.1 Equivalent Diagonal Strut

The interaction between infill panels and frame is as shown in Figure 2.4. When a horizontal seismic action is applied, the frame experiences flexural deformation while the infill experiences shear deformation. This subsequently forms the equivalent diagonal strut.

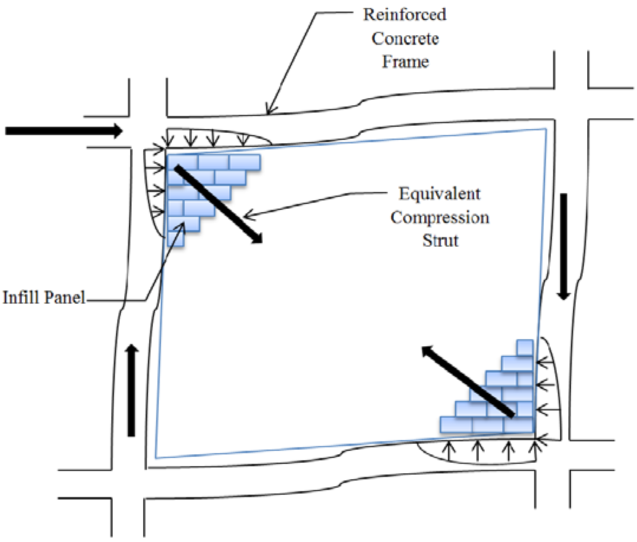

Figure 2.4: Infill Walls Resist Frame Deformation through Diagonal Compression

In cases where the infill is much stronger than frame, the shear force from the infill is transferred to the frames, causing the frame to experience shearing too. This large shearing force could cause shear failure in the adjacent columns and beam-column joints (Semnani, Rodgers and Burton, 2014).

2.6.2 Openings in Infills

Openings are commonly found in most masonry buildings. Typical opening in infills are door and window openings. Presence of these openings influences the strength and stiffness of frame. For example, with the presence of large opening, the diagonal strut as mentioned in the above subsection is significantly reduced. According to Eurocode 8, tie-beams and tie-columns are needed to frame the openings.

2.6.3 Short Column Effect

Short column effect is also commonly known as captive column effects. The locations of the openings, mentioned in the above subtopic, would cause the formation of short column effect. For instance, the opening adjacent to the column case, the partial-height infill restrains the deflection of the column, causing short column effect.

According to the Conceptual Seismic Design Guidance for New Reinforced Concrete Framed Infill Buildings by GeoHazards International, under a seismic loading, all columns are supposed to deflect similarly (Semnani, Rodgers and Burton, 2014). In other words, the short columns are required to deflect the same amount as the full-height columns. Thus, short column would experience larger shear force, whereby results in brittle shear mode of failure.

Figure 2.5: Shear Damage due to Short Column Effect in Indonesia (Photo credits: Tim Hart, Lawrence Berkeley National Laboratory)

2.6.4 Soft Storey Effect

Soft storey is the irregular building configurations as a result of the absence of infill walls or presence of large openings in infill walls. This configuration is generally found in many buildings in Malaysia, to be utilized as car parks. Figure 2.6 shows the concentrated deformation of columns in ground level of a structure.

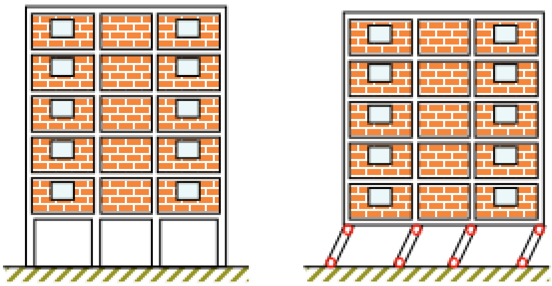

Figure 2.6: Soft Storey Effect

In fact, for a building with complete infill panels (without soft storey), the ground level masonry infill walls resist the highest lateral loading in a seismic action. Thus, it is a norm for many structures to fail at the ground level during earthquakes in this case. Nonetheless, in cases where soft storey exists, the level with soft storey (despite which level the soft storey existed at) would fail before the ground level fails. Reinforced concrete columns in soft and weak storey can experience lateral deformations that are more than ten times those in the storeys with substantial infill walls.

2.6.5 Irregularities due to Masonry Infills

Irregularities due to masonry infills include plan irregularities and vertical irregularities, which is also known as elevation irregularities.

(a) Irregularities in Plan

According to Eurocode 8-1: Clause 4.3.6.3.1, such irregularity includes buildings without geometric shape, and symmetrical geometric-shaped buildings without re-entrant corners or wings. For the first case, irregular, unsymmetrical and non-uniform arrangements of infills would cause severe irregularities. As for second case, the distribution of mass and position of seismic-force-resisting elements also contributes to the irregularity of plan. A factor of 2.0 is used to double the accidental eccentricity (Mahdi and Khorramiazar, 2012).

(b) Irregularities in Elevation

This vertical irregularity is basically drastic reductions or absence of masonry infill walls in one or more storeys as compared to the others, which eventually increases the seismic effects (mass, stiffness and strength) acting on the vertical elements. According to Eurocode 8, in such cases, a magnification factor is adopted to assess the need of modification of action effects. Some typical examples of irregularities in elevation are soft storey and openings near to columns.

2.6.6 Failures of Infill Panels

There are two types of failure of infill panels, which are in-plane failure and out-of-plane failure.

(a) In-Plane Failure

The main reason of the occurrence of in-plane failure is due to the failure of the infill materials, which are the masonry unit and mortar. Masonry walls are subjected to large shear stress, causing brittle failure and cracking to form along the diagonal strut (Mahdi and Khorramiazar, 2012).

(b) Out-of-Plane Failure

As for out-of-plane failure, this failure is caused by the weak bonding of infill panel and frame. Thus, when large lateral force is acted on the masonry walls, the infill panel is separated from the frames, causing collapse of masonry walls. Besides, when a masonry wall has undergone in-plane failure and cracked, any lateral force further applied to the wall would cause the wall to collapse too. In other words, in-plane failure would subsequently lead to out-of-plane failure (Mahdi and Khorramiazar, 2012).

2.7 Reinforced Masonry

Reinforced masonry construction consists of reinforcing steel bars or mesh added into the brick and mortar system. Reinforcement transfers tensile stresses across cracks thus holding the structure together against excessive deformation, disintegration into parts, and overall collapse of whole structure. Thus, seismic resistance to lateral loading are further improved as compared to unreinforced masonry structures.

2.8 Confined Masonry

Confined masonry construction is widely applied in many developing countries such as Italy, Mexico, Chile, Peru, Indonesia and China, especially for low-rise residential buildings. However, it is still not a norm to apply such structural system here in Malaysia. Confined masonry construction basically is defined as the confinement of masonry walls on all sides with vertical or horizontal confining elements. It consists of masonry wall, horizontal reinforced concrete (RC) elements (ring beam or tie-beams) and vertical RC elements (tie-columns), as shown in Figure 2.7. Typically, tie-columns are smaller in cross-sectional dimensions, consisting of a rectangular section with cross-sectional dimensions corresponding to the wall thickness. According to Eurocode 8 Clause 9.5.3 (3), the minimum cross-sectional dimension of both tie-beams and tie-columns are 150 mm. Thus, the suggested dimension is 150 mm to 200 mm.

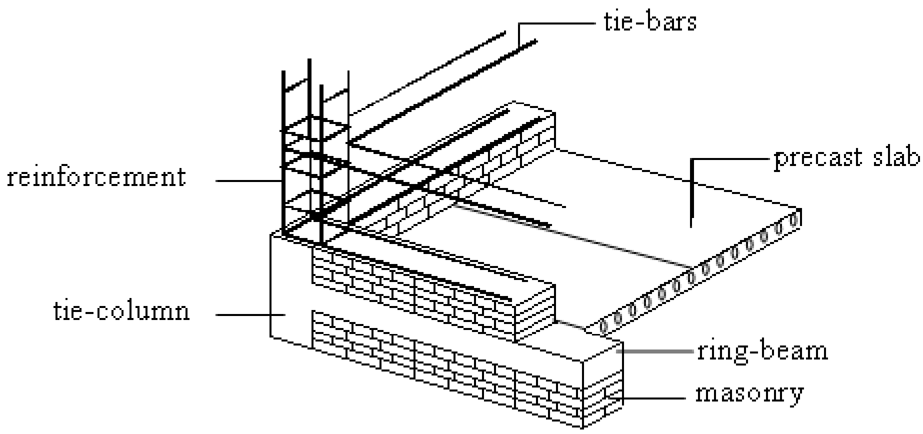

Figure 2.7: A Typical Construction Detail of Confined Masonry Structures

Confined masonry walls function as vertical and earthquake (lateral) load carriers by restraining the masonry panel. The horizontal tie-bars help to improve the structural integrity between the tie-columns and brickwork. The significant differences between confined masonry and other masonry system are in terms of construction sequence and seismic performance.

In terms of construction sequence, masonry walls and tie-columns are constructed first, followed by the cast in-place beams on top of the wall. This is slightly different with the current typical practice in Malaysia, which is to construct the RC frame first, and then followed by filling in with the masonry panel. The sequence of casting confined masonry (casting masonry wall panel first) helps to enhance the bonding between the tie-system and the masonry wall panel.

As for seismic performance, unlike most infills in RC frames bearing only its self-weight, masonry walls in confined masonry construction mostly support gravity loads. Adding on, due to the difference in construction sequence, confined masonry construction could also prevent having gaps between the masonry wall and concrete beams (on top of the masonry). Gaps are commonly found in infilled masonry, causing less bonding between masonry wall and concrete beam, at the same time, allowing the beams to deflect without transferring the gravity loads to the wall below. These gaps unduly minimize the capability of masonry walls and beams. Speaking of lateral seismic loads, confined masonry wall also acts as shear wall, which is much similar to unreinforced or reinforced masonry walls (EERI, 2011).

2.8.1 Confinement of Concrete

Three fundamental functions of reinforcement are to transfer tensile stress across cracks, to control cracks and to confine concrete. The strength envelopes of concrete in compression and tension are summarized in Figure 2.8. Confined concrete which is under triaxial compressive stress state, has higher strength and ductility. A stress-strain graph of confined and unconfined concrete adopted from Eurocode 2 is also shown in Figure 2.9.

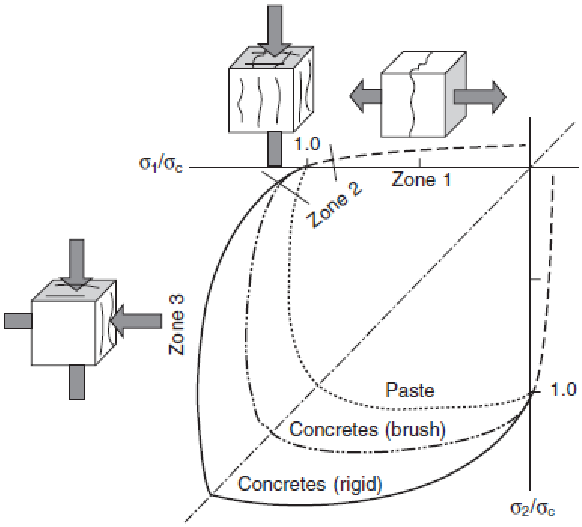

Figure 2.8: Biaxial Failure Envelope of Paste and Concrete (Newman and Ban, 2003)

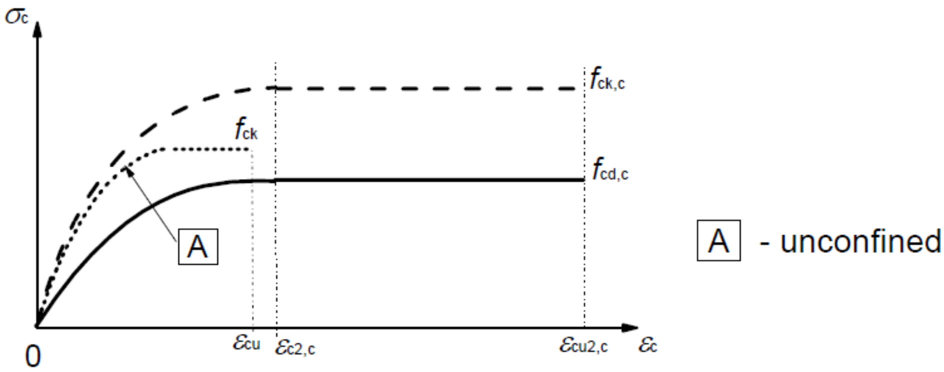

Figure 2.9: The Graph of Stress versus Strain in Eurocode 2

2.8.2 Tearing Failure of Floor Diaphragms

Cyclic tensile forces are generated in floor diaphragms under earthquake loads. The masonry infill walls and frame resist the lateral seismic actions by strut and tie action. The tensile forces in the diaphragms are very high, especially at the first floor of the building. To resist these tensile forces, sufficient reinforcements need to be provided.

Premature failure is prevented by making sure that the horizontal ties do not yield earlier than the occurrence of the ductile beam-sway mechanism for the building.

For cast-in-place RC floor slabs, the flexural reinforcement provided is often sufficient for serving as ties as compared to precast construction method. However, for restraining precast floor systems, sufficient ties or tie beams have to be installed to prevent the premature tearing failure of the floor diaphragm. In fact, from the outcome of the field investigation in Dujiangyan, tearing failures of floor diaphragm are pretty much uncommon (Su, et al., 2011).

2.8.3 Tension Failure of Columns

Under the seismic actions, reversed cyclic axial forces are generated in tie columns. Such actions could cause the yielding of vertical ties and de-bonding between tie-beams and infill masonry walls.

2.9 Seismic Performance in Eurocode 8 Part Three

Retrofit, in simple words, means modifications by adding in new technology to existing structures to increase their resistance to seismic activities, ground motions or soil failures caused by earthquakes. Often, retrofitting is focussed on structures with open frames, small columns, soft storeys and/or low-grade bricks, as they could have experienced more severe seismic damage. In the recent Ranau earthquake, a handfull of buildings suffered typical damage such as cracking, as shown in Figure 2.10. These incidents have implied that not all the existing buildings in Sabah are capable to reach a minimum required level of performance in earthquakes with the magnitude of as high as 6.0.

Relevant seismic performance on existing structures to identify the need of retrofitting, following by appropriate retrofitting to repair and strengthen weak structures should be executed by engineers. Eurocode 8 part three deals with the assessment and interventions of buildings.

Figure 2.10: Typical Damage of Buildings

2.9.1 Guidelines in Eurocode 8 Part 3

The limit states associated with the state of damage in a structure are Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL). The return period (RP) of Limit States of NC is two thousand four hundred and seventy-five (2475) years, SD is four hundred and seventy-five (475) years and DL is two hundred and twenty-five (225) years respectively. In other words, the no-collapse requirement in Eurocode 8 part one is roughly approximate to limit state of SD.

In Eurocode 8-3 Annex C, the assessment and design of retrofitting are recommended covering in-situ assessment such as examination of the geometry, details and materials. This includes the checking of the physical condition of masonry elements, location and size of walls (with or without openings), identification of types of walls (unreinforced masonry, confined masonry or reinforced masonry) and in-situ testing of materials used. The few possible tests such as Schmidt rebound hammer test (to evaluate hardness), hydraulic flat jack test (to measure in-situ shear strength and vertical compressive stress) and diagonal compression test (to estimate shear strength and shear modulus). In most actual cases, in-situ direct experimental measurement of material parameters is not feasible as it is dependent on the quality of material and construction, thickness of wall and availability of adequate experimental equipment (Magenas and Penna, 2009).

There are basically four types of method to analyze, which as lateral force analysis, modal response spectrum analysis, static (pushover) analysis and time history (dynamic) analysis. For the limit states of NC and SD, non-linear analysis is suggested, while for the limit state of DL, both linear and non-linear analysis is suitable.

2.10 Applied Technology Council (ATC)

Several studies have been conducted to assess the seismic vulnerability of buildings by determining the potential earthquake hazard and identify the unacceptable components. Universiti Teknologi Malaysia (UTM) has adopted "Applied Technology Council" (ATC) for visual screening on conditions of the building through site walk. This seismic inspection involves rapid screening procedure (RSP) survey mainly through seismic inspection method.

The first approach, ATC-21 survey is basically a preliminary tool to assess the building's capability to resist seismic threats just by judging from its external appearance, to conclude the earthquake hazard risk of building collapse. Buildings failing to pass ATC-21 will then have to undergo ATC-22 evaluation where the structural integrity as well as non-structural implications is taken into consideration. Yet, the assessment is still concentrating on qualitative evaluation based on the score sheet contained on the checklist.

ATC will depend on the quality of strong motion data such as quantity and the distribution of parameters of attenuation function developed by empirical method. Besides, this evaluation is also subjected to many uncertainties and primarily depends on the personal judgment of the inspector in-charged (Ramli, Ismal and Suhatril, 2008). This assessment is subjective. For instance, the assessment of a similar building could vary significantly by the assessment conducted by different inspectors. Thus, this approach is still less appropriate to be used to develop the procedure of assessment in Malaysia. A comparison of Eurocode 8 part three versus ATC is performed and the result is illustrated in Table 2.3.

Table 2.3: Table of Comparison of Eurocode 8 Part 3 and ATC

	Eurocode 8 Part 3	**ATC**
Approach	Quantitative Approach (Objective)	Qualitative approach (Subjective)
Procedure	1. Three requirements / limit states: Near Collapse (NS) – 2475 years RP, Significant Damage (SD) – 475 years RP and Damage Limitation (DL) – 225 years RP. 2. Non-linear modeling is adopted 3. Determining failure mode of building components	1. ATC-21- a visual screening considering the building’s capability to resist seismic threat to identify if building is hazardous 2. Fails ATC-21: ATC-22 - a visual screening considering structural integrity and non-structural implications

2.11 Potential Failure Modes

The main failure mode of an infill panel is brittle failure, which consists of both buckling failure and compression failure.

According to Eurocode 6 Clause 5.5.1.4, in the analysis of structural members, the slenderness ratio of a masonry wall shall be obtained by dividing the value of the effective height, h_{ef} by the value of the effective thickness, t_{ef} . A reduction factor was introduced for slenderness and eccentricity. When the slenderness ratio calculated is greater than twelve, reinforced masonry could be designed using the principles and application rules for unreinforced members, taking into account the second order effects by an additional design moment. Eurocode 6 Annex G included a detailed explanation of the reduction factor for slenderness and eccentricity of unreinforced masonry. When the Eurocodes are implemented, this clause has already been adopted in the design of masonry walls, thus, checking of buckling failure is no longer essential.

As for compression failure of diagonal strut, in masonry-infill panel, this failure commonly occurs in the direction of the principal compressive stress. In other words, the infill failure force is determined by the compression strength of strut. One of the commonly found crack pattern is zigzag (stepped), where the cracks follows the diagonal path along the mortar. As such, it can be deduced that the mortar strength is weaker than the masonry unit strength. On the other hand, when the crack passes through the masonry units under diagonal path, it can be concluded that the masonry units strength are equivalent (or smaller) than mortar strength. This phenomenon is similar to the concrete fracture phenomenon as shown in Figure 2.11 where the aggregate particle (in Figure 2.11) is comparable to a masonry unit in a masonry-infill panel.

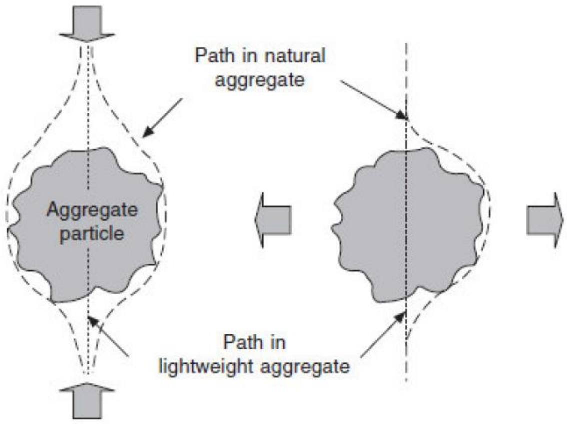

Figure 2.11: Fracture Paths (Newman and Ban, 2003)

2.12 Influencing Parameters

There are several influencing parameters that would affect the behaviour of infill panels, namely aspect ratio of panel, gravity load, opening on panel etc (Stavridis, 2009). From previous findings, the influencing parameter is summarized in Table 2.4 with respect to the initial stiffness, strength, infill and failure mechanism.

Table 2.4: Effect of Design Parameters on Infilled Masonry Walls

Parameter	Initial Stiffness	Strength	Infill	Failure Mechanism Windward Column	Failure Mechanism Leeward Column
Aspect Ratio	Significant	Significant	No effect	Significant	No effect
Vertical Load	No effect	Significant	No effect	Some effect	Minor effect
Ratio of Longitudinal Reinforcement	No effect	No effect	Some effect	Some effect	No effect
Area of Transverse Reinforcement	No effect	No effect	No effect	Significant	No effect
Spacing of Transverse	No effect	No effect	No effect	Significant	Minor effect

2.12.1 Aspect Ratio

Aspect ratio, in simple words, is the proportional relationship of width and height of infill panel. Such ratio is easily identified by dividing the height of wall panel by length of wall panel. Previous studies have found out that there are significant differences in the behaviour of infill panels in terms of the initial stiffness, lateral strength and drift at peak lateral force. Figure 2.12 illustrates the effect of different aspect ratios. For instance, panel with higher aspect ratio or longer length has higher stiffness value, and the panel reaches its strength at a higher drift (Stavridis, 2009).

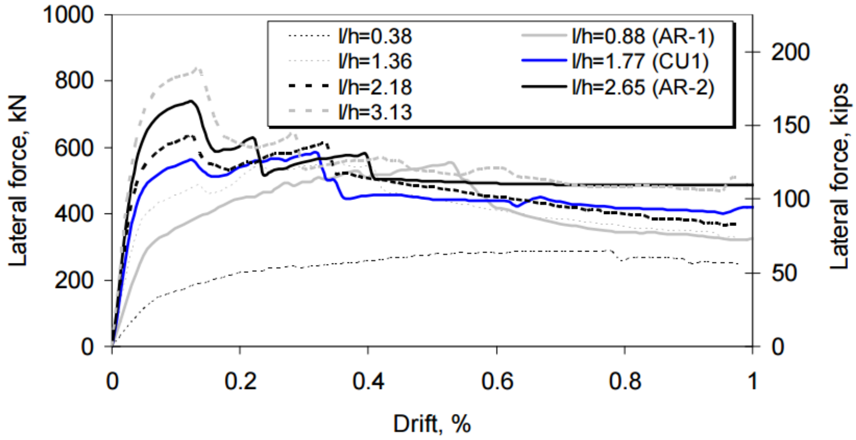

Figure 2.12: Effect of Different Aspect Ratios (Stavridis, 2009)

2.13 Autoclaved Aerated Concrete (AAC)

Autoclaved aerated concrete (AAC), is basically a mixture consisting of cement, lime, water and aluminium powder. The density of AAC is relatively low as compared to masonry unit, as AAC is relatively more porous due to the chemical reaction between aluminium and concrete. Hence, AAC is a low weight material with large volume. Besides, AAC has low value of Young's Modulus, *E* value, with high strain value, signifying that AAC has a high deformability limit. These material properties aid AAC block wall panel to deform more than ordinary masonry unit wall panel before failing (Costa, et al., 2008).

2.14 Summary

In summary, the focus of this study was to investigate several parameters to aid future researches on design guidelines for new buildings and procedures for seismic performance assessment for existing buildings in Malaysia, particularly in Sabah. This study was done based on several available materials, namely Eurocode 6, Eurocode 8 part one and three, Applied Technology Council (ATC-21 and ATC-22) and a number of journal papers. In addition, critical evaluation was made throughout the study.

CHAPTER 3

METHODOLOGY

3.1 Configuration of Masonry Panel

The infilled masonry wall in this project is a single-story, single-bay wall. Figure 3.1 illustrates the example of configuration for panel with aspect ratio of 1.0. Since the height of storey does not vary much in real structures, the heights of the panels were maintained at 3.2 m for all cases while only the length of the panels was varied. The thickness of wall panels was 170 mm.

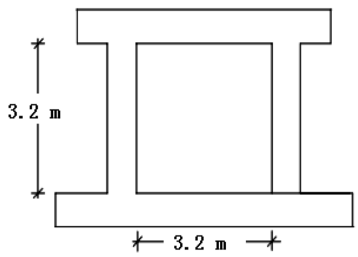

Figure 3.1: Example of Configuration for Panel with Aspect Ratio of 1.0

The analysis was performed by applying two vertical loads of 400 kN each at the nodes (N2 and N4) as illustrated in Figure 3.2. This was to simulate the presence of loading above.

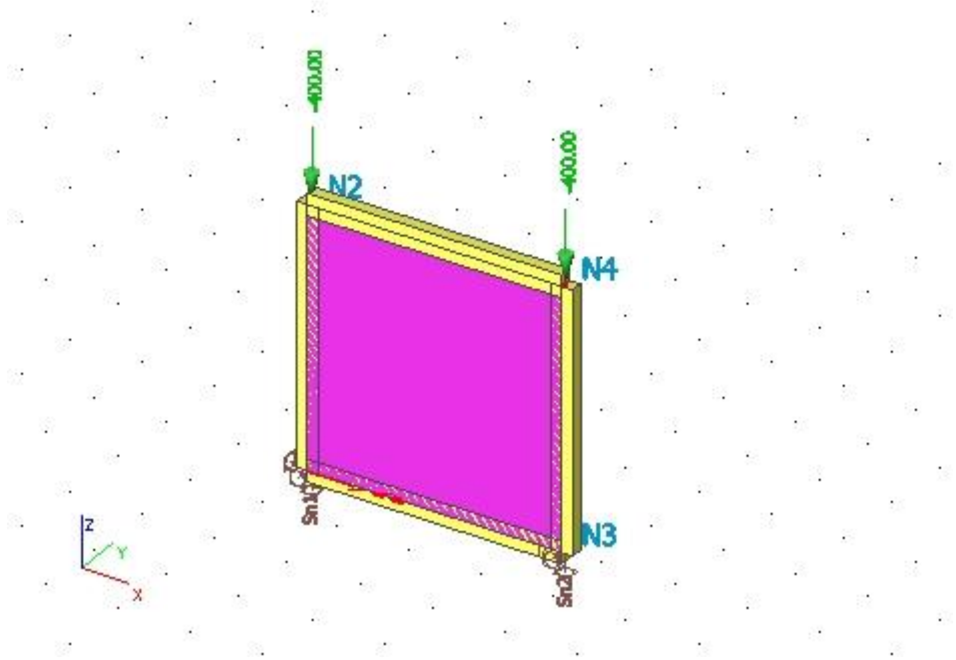

Figure 3.2: Model Used for Simulation in SCIA Engineer Software

3.2 Eurocodes

Figure 3.3 illustrates the outline of this study, which is to focus on Eurocode 2, Eurocode 6 and Eurocode 8. At the early stage of this project, attention was paid on these Eurocodes. Important Clauses such as Clause 5 in Eurocode 6, Clause 3 in Eurocode 8, Clause 4 in Eurocode 8 and Clause 9 in Eurocode 8, just to mention a few, were considered.

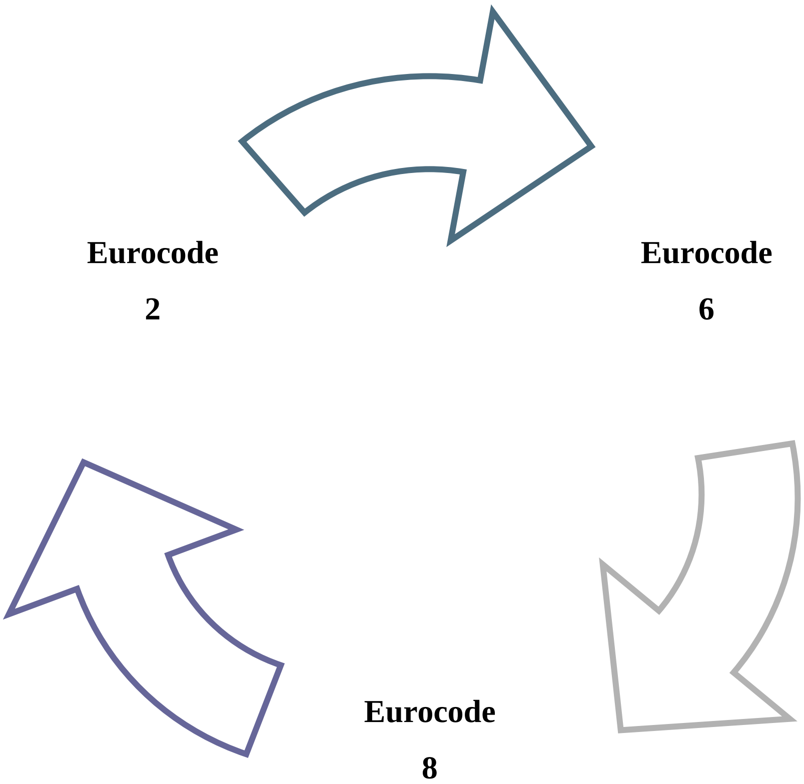

Figure 3.3: Outline of Study (Eurocodes)

3.3 Rules for Masonry Buildings

The dimensions proposed for the wall panels were 3.2 m for height, 170 mm for thickness and 2.55 m to 6.4 m for length of wall panels. These dimensions were proposed based on EN 1996 (design code for masonry structures), Uniform Building By-Laws 1984 (UBBL) and several past research findings.

According to UBBL, wherever references are made to the thickness of any brick wall, the maximum or minimum thickness of such wall shall not exceed the nominal thickness plus or minus the maximum tolerance permissible under any standard specification. Thus, 170 mm thickness was proposed in this project. According to EN 1996-1 Clause 5.5.1.3, the effective thickness, t_{ef} of a single-leaf wall should be taken as the actual thickness of the wall. Hence, the effective thickness of the proposed wall in this project was also 170 mm.

The height of rooms in residential buildings shall be not less than 2.5 m, as mentioned in UBBL. The maximum value of ratio of h_{ef} / t_{ef} in EN 1996-3 is 27, and the clear storey height shall not exceed 3.2 m (or 4.0 m for cases where the overall height of the building is greater than 7.0 m). In cases where walls are acting as end support to floors, the effective height, h_{ef} for these walls are equivalent to the clear storey height. With these criteria, the proposed height of the wall panels was 3.2 m, which was equivalent to the maximum clear height storey in Eurocode 6. After referring to both Eurocode 6 and UBBL, the proposed lengths of wall panels ranged from 1.55 m to 6.4 m too.

3.4 Design Response Spectrum

The design response spectrum was generated based on unified response spectrum model, of a return period of two thousand and five hundred years by taking Kuala Lumpur as reference. The equations below define the displacement spectral ordinates and acceleration spectral ordinates (Looi, et al., 2013).

Response Spectral Displacement (RSD)

$$T \le T_C : S_D(T) = S_D(T_D) * T^2 / (T_C T_D) \tag{3.1}$$

$$T_C \le T \le T_D : S_D(T) = S_D(T_D) * T / T_D \tag{3.2}$$

$$T_D \le T \le 2 : S_D(T) = S_D(T_D) + [S_D(2) - S_D(T_D) * (T - T_D) \tag{3.3}$$

$$T \ge 2 : S_D(T) = S_D(2) + 10 * (T - 2) \tag{3.4}$$

Response Spectral Acceleration (RSA)

$$RSA = RSD * (2\pi / T)^2 \tag{3.5}$$

where

T = structural period, s

T_C = first corner period, s

T_D = second corner period, s

Subsequently, in another IEM workshop on a "2-day Workshop on Recommended Earthquake Loading Model in the Proposed National Annex to Euro Code 8 for Sabah, Sarawak and Updated Model for Peninsular Malaysia" held in July 16-17, 2014, the recommended earthquake design spectrums for no-collapse requirement (2475 years return period) are summarized in Table 3.1 (Hee, 2014). According to Table 3.1, the maximum response spectral acceleration (RSA_{max}) is 0.25 g and 0.45 g for Peninsular Malaysia and Sarawak, and Sabah respectively.

Table 3.1: Design Parameters of Response Spectrum for Malaysia

Parameter	**RSA_{max}**	**PGA**	**SD(T_D)**	**Slope m**	**T_C**	**T_D**
Unit	**g**	**g**	**mm**	**mm/s**	**s**	**s**
P. Malaysia	0.25	0.1	24	9.6	0.3	1.25
Sarawak	0.25	0.1	24	0	0.3	1.25
Sabah	0.45	0.18	42	60	0.3	1.25

As for damage-limitation requirement (475 years return period), the mean and design values is shown in Table 3.2. A reduction factor of 2.5 is taken into account to obtain the design parameters of this damage-limitation requirement response spectrum.

Table 3.2: Mean and Design Values (Draft NA of MS EN 1998-1)

	500 years		2500 years	
	Mean	**Design**	**Mean**	**Design**
RSD max (mm)	6	12	12	24
RSV max (mm/s)	27	54	60	120
RSA max (g)	0.05	0.1	0.125	0.25
PGV (mm/s)	15	30	33	66
PGA (g)	0.02	0.04	0.05	0.1

With these parameters, two graphs are plotted as depicted in Figure 3.4 and 3.5.

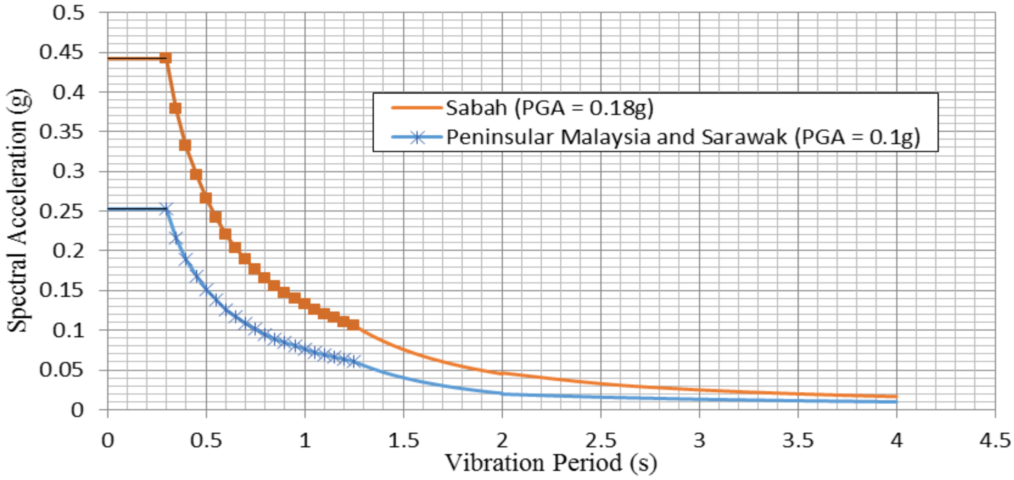

Figure 3.4: Design Response Spectrum (No-collapse Requirement) in Malaysia

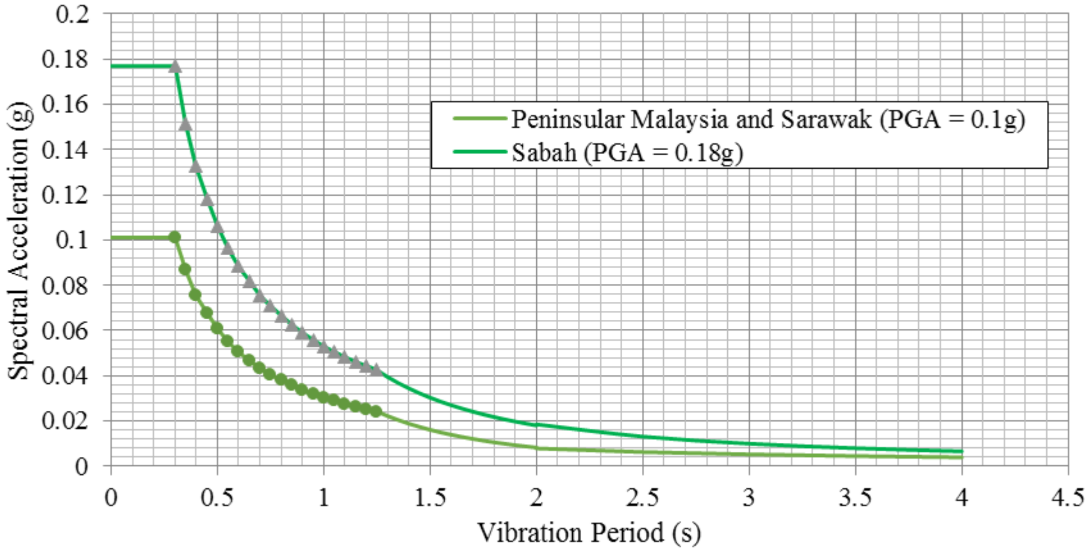

Figure 3.5: Design Response Spectrum (Damage Limitation Requirement) in Malaysia

3.5 Load Combinations

According to Eurocode 8 Clause 3.2.4 (2)P, the induced inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the combination of actions in Equation 3.6.

$$\Sigma G_{k,j} \text{ '+' } \Sigma \psi_{E,j} \text{ x } Q_{k,i} \tag{3.6}$$

where

$G_{k,j}$ = gravity actions, kN

$Q_{k,i}$ = variable actions, kN

$\psi_{E,i}$ = combination coefficient for variable action i

From Eurocode 8 Clause 4.2.4 (2)P, the combination coefficients ψ_{Ei} is computed from Equation 3.7.

$$\psi_{E,i} = \varphi \times \psi_{2,i} \tag{3.7}$$

where

ψ_{2i} = combination coefficients (taken as 0.3 herein)

φ = coefficient relates to the type of variable actions (taken as unity herein)

3.6 Finite Element Analysis with SCIA Engineer Program

The results were obtained with SCIA Engineer program. The seismic design functionality in SCIA Engineer includes tools for effective modeling and analysis of buildings under seismic actions according to design code principles. Non-linear analysis models the plastic behaviour of materials, which is inconsistent to be adopted in the masonry wall cases (with brittle failure which occurs rather abruptly and violently with little ductile deformation before failure). Thus, linear analysis is conducted, instead of non-linear simulation.

3.7 Flow Chart of Work

Figure 3.6 illustrated and summarized the workflow for this project.

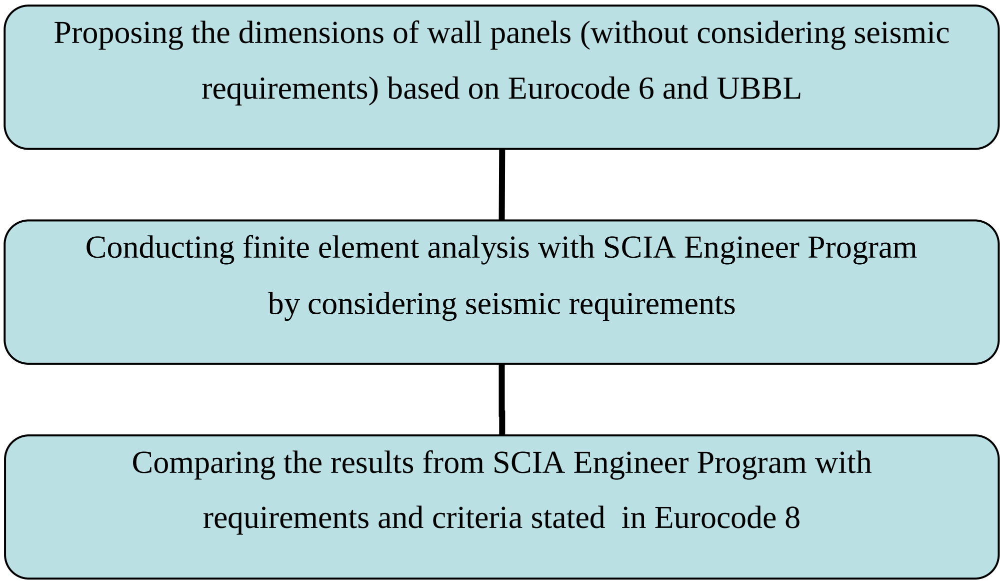

Figure 3.6: Workflow of Project

3.8 Conclusion

In short, after taking into consideration the requirements in EN 1996 and UBBL (without seismic), the configuration of wall panel was proposed. Sabah's response spectrum was adopted in this study. The proposed wall panel was simulated and analyzed with SCIA Engineer program under linear elastic analysis. Checking was done with EN 1998 (with seismic) to verify the sufficiency of proposed dimensions. Charts were also proposed for preliminary seismic assessment purposes.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter discusses the results obtained through SCIA Engineer software and manual calculation using Microsoft Excel.

4.2 Deformation of Wall Panels

From the results of wall panels (middle plane) deformation in SCIA Engineer software, it has been noticed that in-plane failure was not the controlling factor. Out-of-plane effect was clearly illustrated in the results adopted from SCIA Engineer software in Figure 4.1. Even so, this did not imply that in-plane effect did not occur, as out-of-plane failure was normally initiated by in-plane failure, when a masonry wall has undergone in-plane failure and cracked, any lateral force further applied to the wall would cause the wall to collapse as Figure 4.1 too.

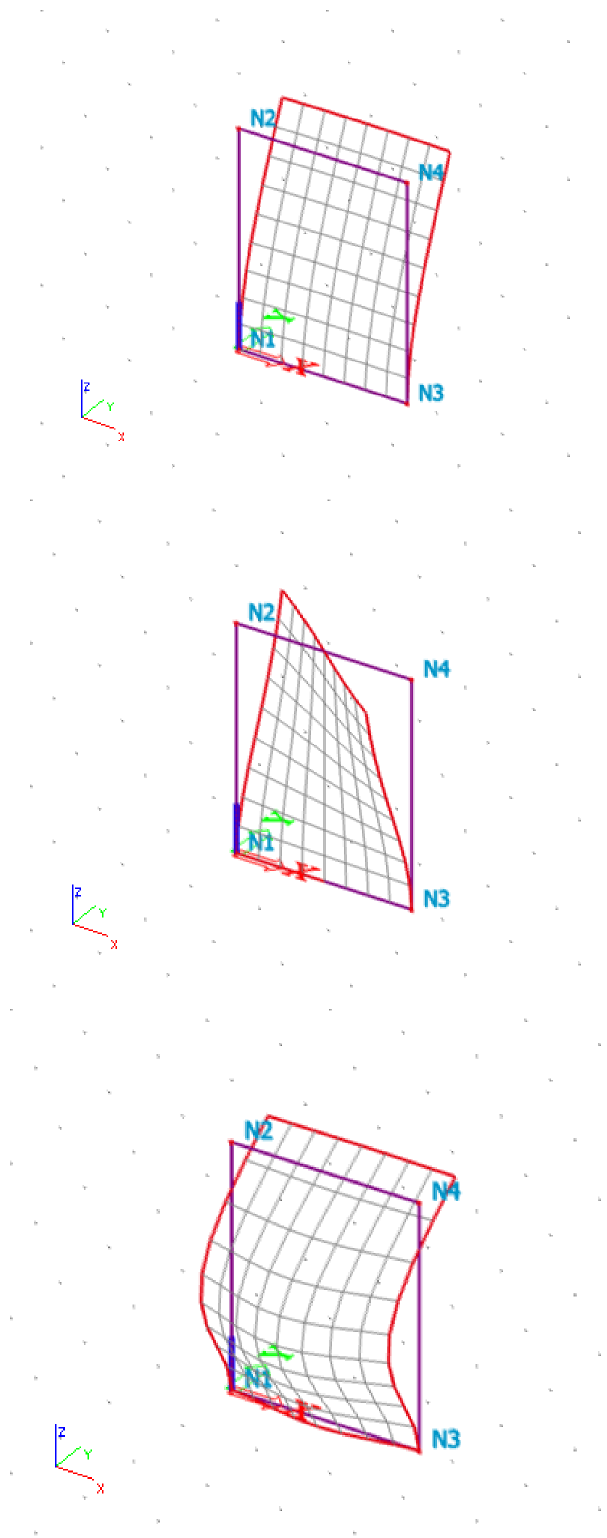

Figure 4.1: Examples of Out-Of-Plane Effect

4.3 Deformation of Nodes

It has been noticed that the deformation of (corner) nodes of wall panels of different aspect ratios have a typical out-of-plane overall deformed configuration. For comparison purpose, the deformed nodes and their initial state were as shown in Figure 4.2. The nodes at the lowest part of the wall panel were fixed (without deformation) while the nodes at the highest part have the largest deformation.

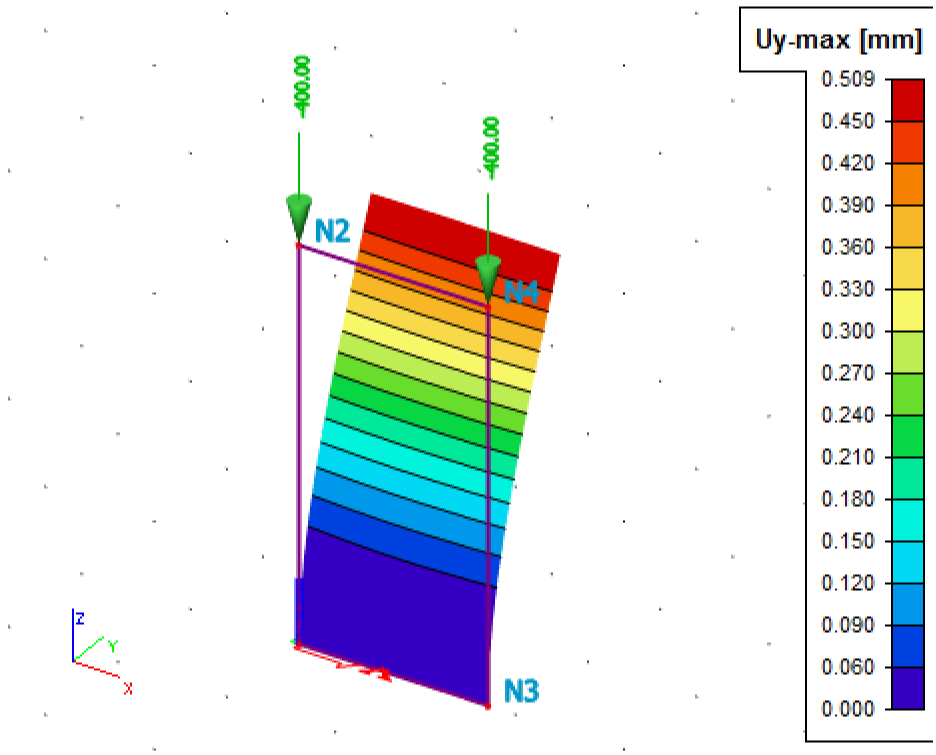

Figure 4.2: The Typical Deformation of Nodes

4.4 Effect of Aspect Ratio

As mentioned earlier in Chapter 3, since the storey height was not expected to vary drastically in real structures, the effect of the aspect ratio has been examined by varying the length of the frame. The analysis initially involved the lengths of wall panel of 2.55 m to 6.4 m. Thus, Figure 4.3, Figure 4,4 and Figure 4.5 illustrated the effect of aspect ratio of 0.5, 0.75, 1.0 and 1.25.

Response of masonry walls are dependent to aspect ratios. In SCIA Engineer software, the maximum deformation of each aspect ratio in the most prominent case was selected, and was plotted in Figure 4.3. In Figure 4.3, the smaller the aspect ratio of wall panel is, the longer the wall is, the stiffer the wall panel is.

As for Figure 4.4, this figure illustrated the relationship between drift and aspect ratios of wall panel. Figure 4.4 serves as a checking chart to determine the maximum drift of infilled wall panel in different aspect ratio cases and aid retrofitting works for existing buildings, especially buildings in Sabah. For instance, simple actions such as by measuring the dimensions (length and height) of an existing wall panel and by interpolating the graph in Figure 4.4 as according to the calculated aspect ratio, an engineer is able to predict the deflection of a specific wall panel subjected to Sabah's ground acceleration motion.

In EN 1998 Clause 4.3.3.2 (1), under damage limitation requirement, the drift limits are 0.5 %, 0.75 % and 1.0 %, for non-structural elements of brittle materials, ductile non-structural elements, and non-structural elements fixed in a way respectively. Since masonry units are brittle materials, the drift limit of 0.5 %, was used in this study, and was as plotted in Figure 4.4. With this limit, the maximum allowable height of wall panels was limited to 2.5 m, which was only the minimum height for residential buildings according to UBBL. Thus, for conservative designs, the minimum wall thickness of 170 mm was insufficient in Sabah to meet the requirement of UBBL.

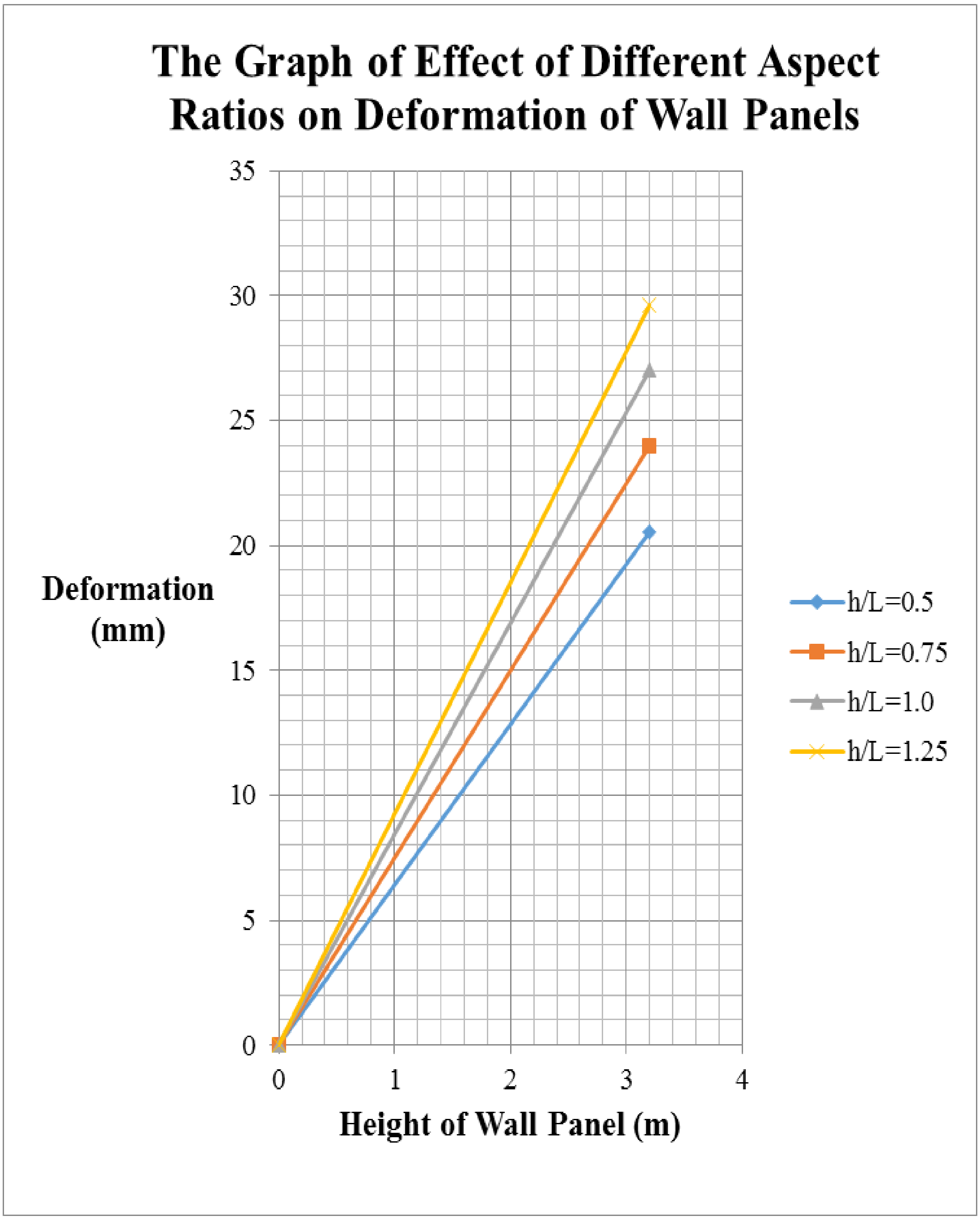

Figure 4.3: The Effect of Different Aspect Ratios on Deformation

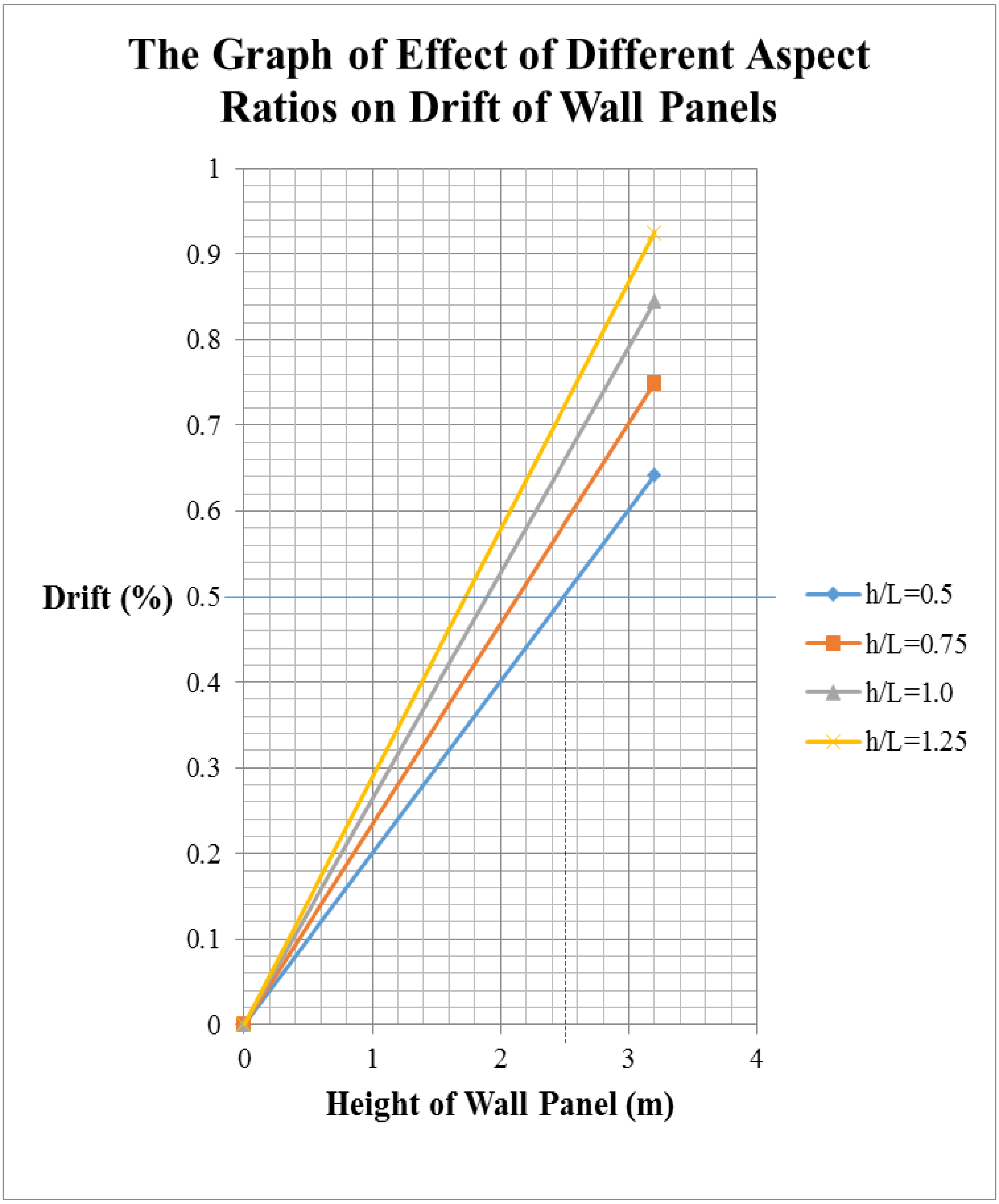

Figure 4.4: The Effect of Different Aspect Ratios on Drift

From the past research findings by Lim (2015), through equivalent diagonal strut analysis, the recommended wall thickness was 170 mm for Peninsular Malaysia and Sarawak. If 170 mm thickness were to be used in Sabah for economical design, higher strength masonry units should be adopted. In this study, instead of using higher strength masonry units, thicker walls of 200 mm thickness were proposed and analysed with SCIA Engineer in the following part.

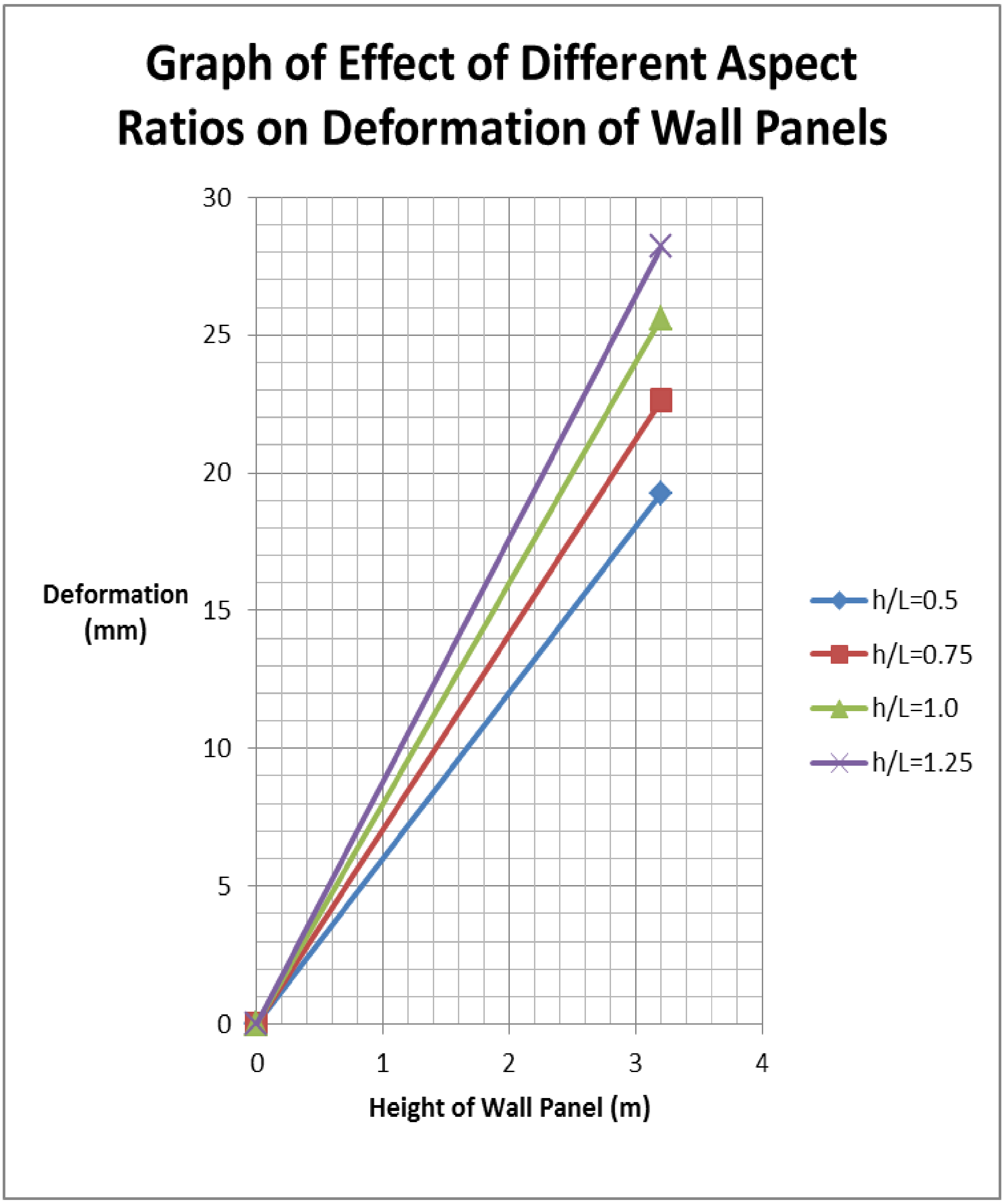

Figure 4.5: The Effect of Different Aspect Ratios on Deformation (200 mm Thickness Case)

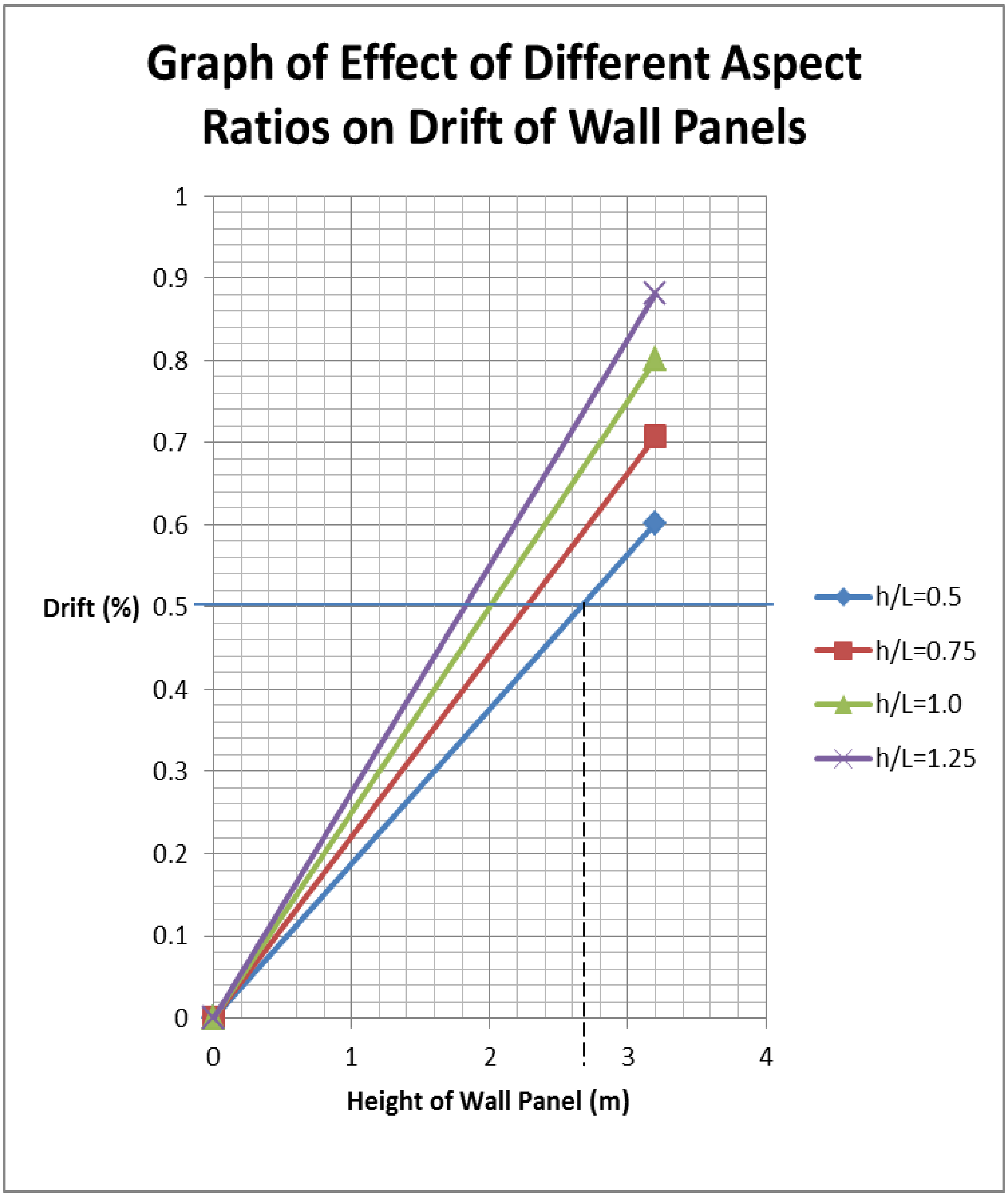

Figure 4.6: The Effect of Different Aspect Ratios on Drift (200mm Thickness Case)

For 200 mm thickness case, Figure 4.5 and 4.6 illustrated the results on deformation and drift adopted from SCIA Engineer software. From Figure 4.6, with the drift limit of 0.5 %, the allowable maximum height of wall panels was limited to 2.7 m, which was higher than the minimum height for residential buildings, as the height of rooms in residential buildings shall be not less than 2.5 m according to UBBL.

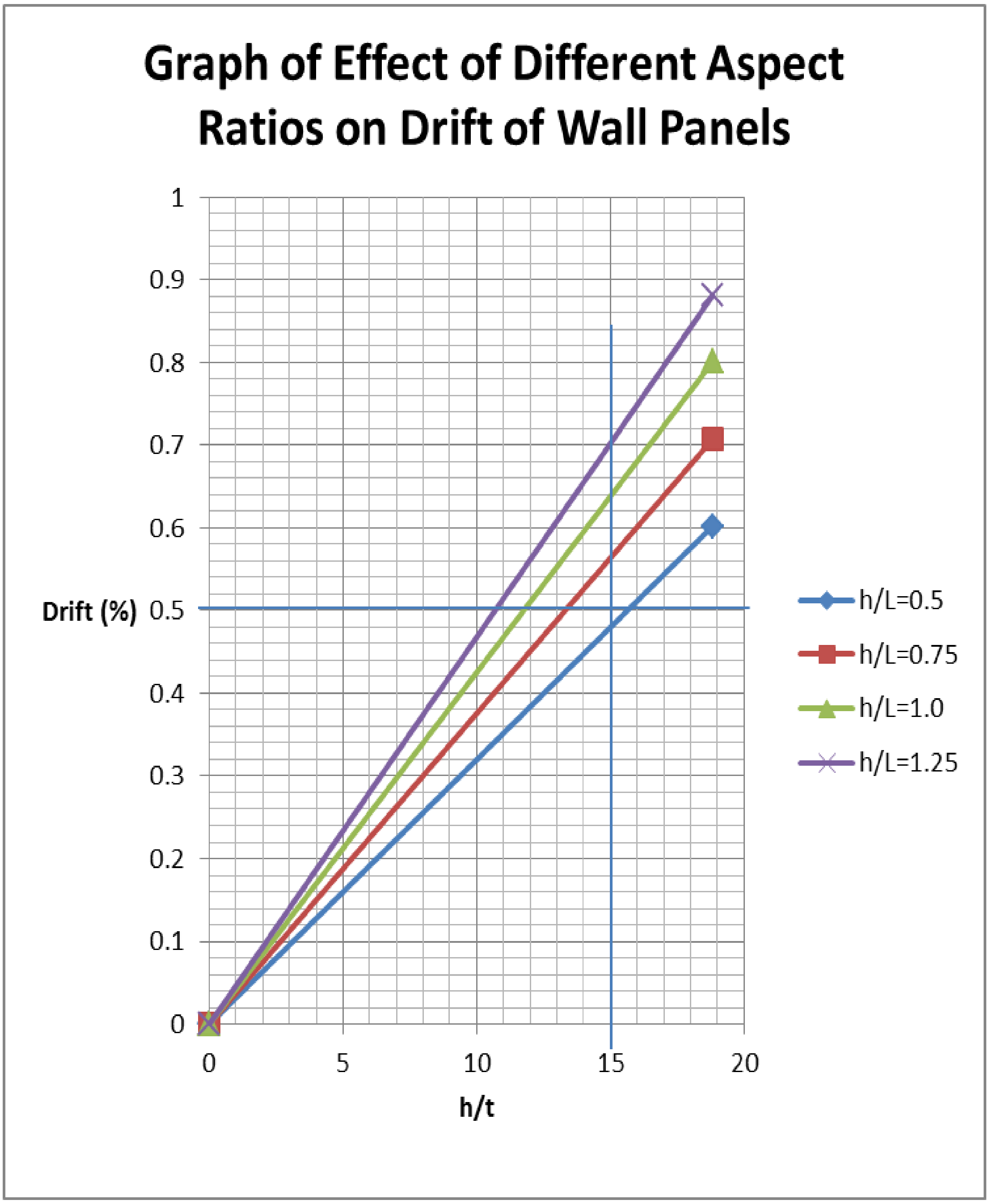

Figure 4.7: **The Effect of Different Aspect Ratios on Drift (with h/t on X-axis) (200 mm Thickness Case)**

The graph in Figure 4.7 was plotted to further verify the dimensions proposed with EN 1998. Since this code is the additional rules for EN 1996, the proposed dimensions should also meet the requirements in EN 1998 too. According to EN 1998-1 Clause 9.5.1 (5), there are certain geometric requirements in cases of low seismicity areas like Malaysia, namely a minimum effective thickness of unreinforced wall panel of 170 mm and a maximum value of ratio of h_{ef}/t_{ef} of 15.

In this part of the study, the proposed thickness (200 mm) has met the minimum thickness requirement of 170 mm. As for the ratio of h_{ef}/t_{ef} of 15, as shown in Figure 4.7, with the limit of 0.5 % drift, h_{ef}/t_{ef} of this study has a value of 16 (at h/L =0.5 case), and has exceeded the limit of 15. Through interpolation, it was found out that, to meet the required ratio of h_{ef}/t_{ef} of 15 in EN 1998, the aspect ratio was limited to 0.6.

Indeed, for long spanning walls, a U-type wall failure would easily occur if the top portion of the wall were not restrained well. This failure would extend to the full height of the wall as shown in Figure 4.8. Thus, walls should not have too low aspect ratio (h/L). In this study, with the geometric requirements of EN 1998, the aspect ratio was limited to 0.6 (with the limited maximum length of wall panel of 5.3 m in this case).

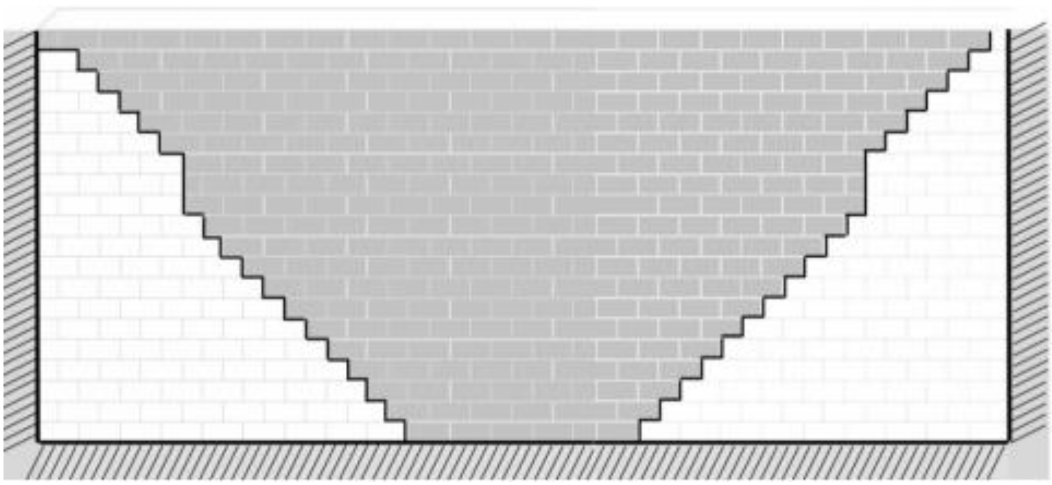

Figure 4.8: Schematic Example of Cantilever U-type Wall Failure

Since these wall dimensions was proposed based on EN 1996, UBBL and other research findings, and has met the requirements in EN 1998 too, it could be concluded that, the recommended minimum wall thickness was 200 mm and the minimum aspect ratio was 0.6 (as interpolated in Figure 4.7) for infilled wall panel designs in Sabah.

4.5 Internal forces and Stresses of Wall Panel

In this analysis, the infilled panel is modelled by a wall element (shell element is categorised as 2D element in SCIA Engineer) with membrane forces and shear forces. Since Kirchhoff bending theory for thin plate was adopted instead of Mindlin bending theory, this analysis focussed on the out-of-plane failure mode of wall panels, considering only normal forces action on the wall instead of the relatively small shear forces. Figure 4.9 illustrated the typical internal forces pattern (membrane force) of wall panels, while Figure 4.10 illustrated the internal stress contour.

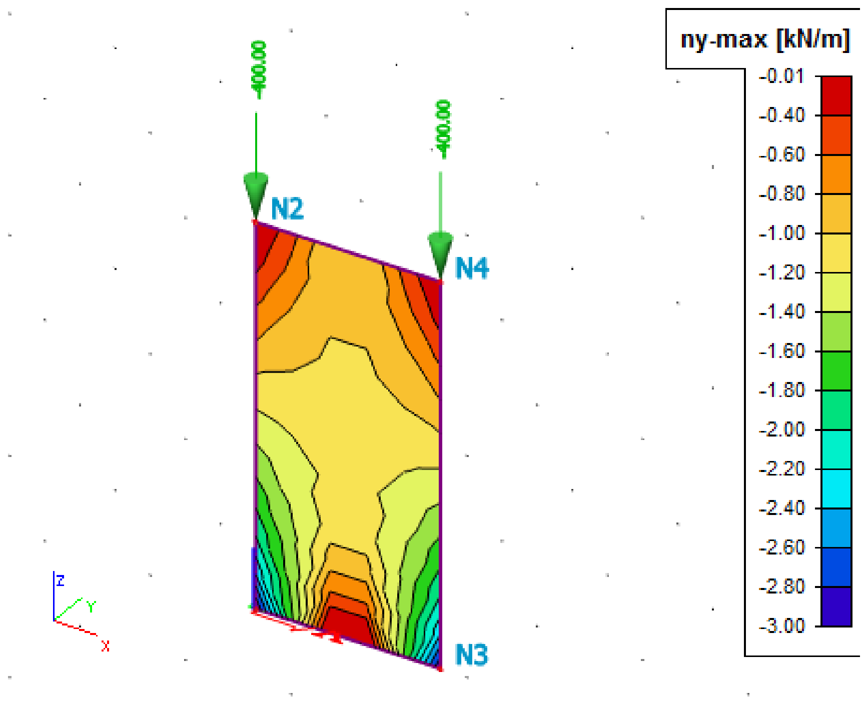

Figure 4.9: Internal Force (Membrane Force)

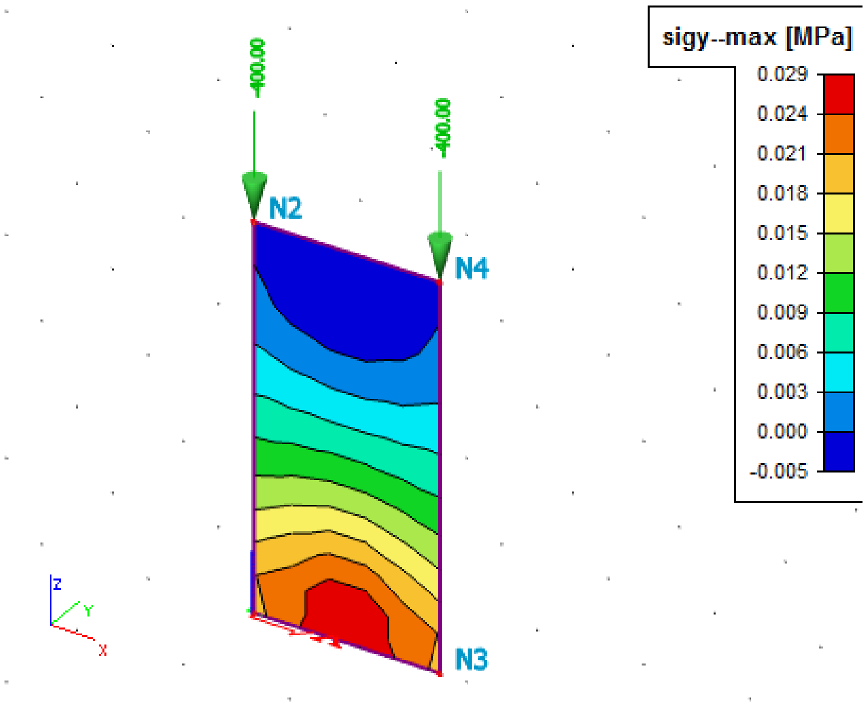

Figure 4.10: Internal Stress Contour

4.6 Autoclaved Aerated Concrete

Autoclaved aerated concrete (AAC) is a worldwide construction material used as an alternative to masonry units. Table 4.1 summarized several existing AAC products in Malaysia, namely Leichtbric, Starken and Alamcon.

The ordinary masonry units have a smaller dimension of 225 mm *x* 100 mm *x* 65 mm and each AAC block is roughly equivalent to seven pieces of masonry units. Since the density of AAC is also relatively low as compared to the ordinary masonry units, AAC is relatively lighter in mass. As a lightweight material, AAC wall panel would probably cause less harm when collapsed. AAC has a low value of Modulus of Elasticity, thus having a high value of strain and deformability. In addition, AAC has poor thermal conductivity too. In fact, these properties of AAC aid wall panels to perform well in seismic areas. Thus, this technology serves as an alternative to increasing the wall panel thickness.

Table 4.1: Comparison of Existing AAC Product in Malaysia

Parameter	Leichtbric	Starken S3	Alamcon
Dimension, Length *x* Height *x* Thickness (mm)	600 *x* 200 *x* 200	600 *x* 200 *x* 300	600 *x* 200 *x* 250
Modulus of Elasticity (MPa)	NA	1500	NA
Mean Compressive Strength (MPa)	4.0	3.5	4.3
Working Density (kg/m^3)	550	700	800
Thermal Conductivity, K (W/mK)	NA	0.16	0.24

NA = Not Applicable

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Based on the results, a number of interesting conclusions can be drawn.

According to EN 1998 Malaysia National Annex draft, Malaysia has two seismic response spectra. The one with lower peak ground acceleration (PGA) was for Peninsular Malaysia and Sarawak, while Sabah's response spectrum has a higher PGA. The PGA adopted for Sabah was 0.18 g on rock with a return period of 2475 years. In this study, only Sabah's response spectrum was used, as the aim of this study was to reduce damage caused by earthquake tremors, particularly in Sabah. However, the results and recommendations proposed from this study are applicable for cases in whole Malaysia for conservative purpose.

Masonry wall failure comprised of both in-plane failure and out-of-plane failure modes. If requirements of Eurocode 6 and UBBL were considered, in-plane failure was not found to be the controlling failure mode. Out-of-plane failure appeared to be more critical, mainly due to the weak bonding of infill panel and frame, and subsequent increase in lateral force after in-plane failure. Out-of-plane failure would cause more harm as compared to in-plane failure in ultimate limit state.

The objective of this study was to look into a few influencing parameters. It could be concluded that for Sabah's response spectrum, under damage limitation, a minimum masonry wall thickness of 200 mm was recommended in Sabah, even though according to EN 1998-1, the minimum wall thickness required for low seismicity areas like Malaysia was only 170 mm. As for Peninsular Malaysia and Sarawak with lower PGA, it would be more economical to recommend a minimum masonry wall thickness of 170 mm. As for the allowable maximum height of wall panels corresponding to this thickness, 2.7 m height was proposed.

In this study, the effect of aspect ratio has been studied by varying the length of the wall panels while keeping the height of wall panel constant. It has been proven in this study that the smaller the aspect ratio of wall panel is, the longer the wall is, the stiffer the wall panel is. However, for walls with too long spans, a U-type wall failure would easily occur if the top portion of the wall were not restrained well. Thus, by taking into consideration these criteria and requirements in design codes, a minimum aspect ratio of 0.6 was proposed.

In short, by taking into consideration the requirements in Eurocode 8, Eurocode 6 and UBBL, a few design requirements were proposed as the base case for the future design works in Sabah, namely the allowable maximum height of wall panels is 2.7 m, corresponding to the minimum masonry wall thickness of 200 mm and a minimum aspect ratio of 0.6.

Aspect ratio is a simple parameter considering only the length and height of infilled wall panel. The graphs in Figure 4.5, Figure 4.6 and Figure 4.7 have included the limits requirement in Eurocode 8, Eurocode 6 and UBBL, which consist of drift limit of 0.5 % and h/t limit of 15. These graphs serve as a preliminary seismic assessment for existing wall panels in Sabah. Since the PGA of Peninsular Malaysia and Sarawak is lower than Sabah, these graphs would be a conservative assessment for both Peninsular Malaysia and Sarawak.

Besides, autoclaved aerated concrete (AAC) is a worldwide construction material used as an alternative to clay masonry units. With its low value of Modulus of Elasticity, high value of strain and deformability and poor thermal conductivity, this serves as an alternative to increasing the wall panel thickness.

5.2 Recommendations

From the conclusion, a few seismic assessment charts were proposed by considering the configuration of wall panels. These charts can be used for preliminary design for new masonry. Further studies on other factors have to be done in order to apply to detailed assessment for existing masonry. For instance,

1. Considering the type of masonry unit
2. Taking into consideration the physical condition of masonry elements and presence of any degradation
3. Studying the properties of constituent materials of masonry elements and quality of connections, and
4. Collecting information on adjacent buildings potentially interacting with the building under consideration, can be done.

Besides, further studies could also be conducted on the retrofitting of buildings. For instance, reinforced concrete jackets or steel profiles can be used to strengthen existing walls. Concrete are applied by shotcrete while jackets are to be reinforced (on one face or both faces of the wall) by welded wire mesh or steel bars. Steel profiles can be used in a similar way as jackets, provided that the steel profiles are appropriately connected to both faces of the wall or on one face only, depending on site condition.

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APPENDICES

APPENDIX A: SCIA Engineer Result for Aspect Ratio of 0.5

APPENDIX B: SCIA Engineer Result for Aspect Ratio of 0.75

APPENDIX C: SCIA Engineer Result for Aspect Ratio of 1.0

APPENDIX D: SCIA Engineer Result for Aspect Ratio of 1.25