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VULNERABILITY STUDY OF EXISTING BUILDINGS DUE TO SEISMIC ACTIVITIES IN SABAH

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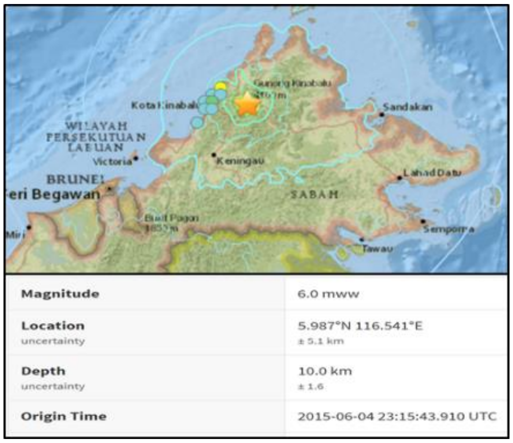
ABSTRACT

The "Vulnerability Studies to Existing Building due to Seismic Activities in Sabah" is the first Malaysia government initiated study for major government buildings and facilities on the safety and structural capacities subjected to local impacts of seismic activities in Sabah. The study was based on the cabinet meeting decision dated 19th June 2015 as the concerns on the building safety was raised after the Ranau earthquake on 5th June 2015. Subsequent from the cabinet meeting decision dated 19th June 2015, Ministry of Works Malaysia had instructed Jabatan Kerja Raya Malaysia (JKR Malaysia) to carry out the vulnerability study in collaboration with Jabatan Kerja Raya Sabah (JKR Sabah). This study is intended to evaluate, to determine structural performance level, identify structural deficiency and providing appropriate retrofitting proposal for those buildings under the risk of damages under predicted intensity of seismic activities. Twelve major districts in Sabah has been identified within this study based on criteria of population, density and levels of potential seismic risk. Total number of fifty-four (54) buildings are evaluate under this exercise under four phases of studies as summarised below as data collection, evaluation by FEMA154 and ASCE41-03, development of demand/capacity analysis and fragility evaluations, and retrofitting proposal.

1. Introduction

On the 5th June 2015, a strong earthquake with recorded moment magnitude of 6.0 had struck Ranau, Sabah (refer Figure 1). The epicenter was located at 10km below the ground, 14km away in northwest of Ranau town. The earthquake caused the biggest casualties and structural damage in the Malaysian history.

Eight-teen (18) lives were taken during this quake. Most of them were killed on the Mount Kinabalu during the climbing activities. The earthquake also affected about 200 families in Ranau and Kota Belud who suffered after subsequent mudslides destroyed their homes, farms and plantations as well as disrupted water supply. Cracks were reported at the residential, commercial buildings, resorts and hotels as well as religious places of worship. Even structures, which are normally used for the emergency such as the hospital, schools and police department were not spared.



Magnitude	6.0 mww
Location uncertainty	5.987°N 116.541°E ± 5.1 km
Depth uncertainty	10.0 km ± 1.6
Origin Time	2015-06-04 23:15:43.910 UTC

Figure 1: Shakemap (USGS, 2015)

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Typical damages reported were brick wall shear failure cracks, cracks on columns and beams, roof failure, failure of supporting columns or tilts, concrete spalling, and shattered windows. There were worries about whether continuous swaying of buildings and vibrations of some parts of the non-structural elements such as doors and windows, will continuously happen and affects the integrity of the whole buildings.

2. Phase I – Data Collection

For the purpose of gathering information of the selected buildings, JKR Malaysia Headquarters had conducted several sessions of meetings and discussion with JKR Sabah whom are regarded as the local authorities and representative for the local communities. With the constant supports from JKR Sabah, most of the required information had successfully acquired without much difficulties.

2.1 As Built Measurement and Soil Information

For those buildings without as-built drawings, several on-site measurements were conducted to verify the plan information and to assess the buildings condition. Total number of 24 buildings were visited in order to carry out dimensional measurement for component structural layout as well as measurement for major structural members.

Since the soil conditions cannot be readily identified by visual methods in the field, existing soil investigation (SI) information for related location was first collected from JKR Sabah during the planning stage and put into a readily information for use during rapid visual screening (RVS). During the screening, or the planning stage, this soil type was documented on the Data Collection Form.

However, for building with no existing SI report available, new SI works was conducted to determine the soil type. There are twenty-six (26) buildings with existing SI reports out of fitty-four (54) buildings. Due to constraint of time and budget, new SI works was limited to eight (8) numbers of SI (bore log) and ten (10) numbers of Mackintosh Probe (M.Probe). Table ***1***1 shows from eight (8) numbers of borehole results.

Table 1: New soil investigation works

No.	Building Name	Average 'N'	Soil Type
1	Dewan Undangan Negeri Sabah	50	D (Stiff)
2	Kompleks Kerajaan Persekutuan	40.2	D (Stiff)
3	Bangunan Utama IPD Kudat	41.1	D (Stiff)
4	Institut Pendidikan Guru Keningau–Flat A	43.1	D (Stiff)
5	Hospital Duchess of Kent, Sandakan – ICU Operation Room	39.5	D (Stiff)
6	Universiti Malaysia Sabah	32.2	D (Stiff)
7	Hospital Tawau	21.5	D (Stiff)
8	Hospital Lahad Datu	36.9	D (Stiff)

3. Phase II – Evaluation by FEMA154 and ASCE 41-03 (Tier 1)

For Phase II evaluation stage, a total of numbers fifty-four (54) identified buildings (see Table) were inspected based on FEMA 154 (2002) Rapid Visual Screening Procedure. These building are mainly public facilities consist of hospitals, police stations, schools, government quarters and offices. Only two types of FEMA 154 (2002) modified data collection forms were used in this exercise which is the Low and Moderate level of seismicity according to local seismicity level as proposed under macrozonation map developed earlier by JKR-UTM (2010). FEMA 154 (2002) was opted for this exercise as it is a simplified format which offers an ease-to-use method to collect information. It also provides a scoring system which enable the users able to determine the potential risk of large numbers of buildings under seismic action within short periods and limited resources.

Table 2: Numbers of buildings involved under each district

No.	District	Nos. of Buildings
1	Kota Kinabalu	3
2	Penampang	3
3	Ranau	4
4	Beaufort	2
5	Kudat	6
6	Kota Marudu	3
7	Keningau	9
8	Tambunan	3
9	Sandakan	6
10	Semporna	6
11	Tawau	3
12	Lahad Datu	6
	Total	54

3.1 FEMA154 Screening Implementation

For steps were involved in the planning and performing RVS of potentially seismically hazardous buildings. First, the seismicity level was determined from the macrozonation map based on two-thirds of the values from a 2,475-year average return period (corresponding to ground motions having a 2% probability of exceedance in 50 years). By interpolation approach, the seismicity intensity for 1.0s and 0.2s period obtained from the map are tabulated in Table 3 and Table 4, respectively. However, the seismicity level for Ranau/Kundasang region were in the low seismicity since this earlier map did not capture effects from the recent Ranau earthquakes. Based on the recorded ground motion intensity during Ranau earthquake, it was decided that Ranau/Kundasang region shall be considered as moderate seismicity.

Table 3: Seismicity level for T = 1.0s

No	District	T=1.0s	2/3 factor	Seismicity
		(Gal)	g	
1	Kota Kinabalu	50	0.034	Low
2	Sandakan	75	0.051	Low
3	Tawau	100	0.068	Moderate
4	Ranau*	50	0.034	Low
5	Kundasang*	50	0.034	Low
6	Penampang	50	0.034	Low
7	Kota Belud	50	0.034	Low
8	Tambunan	50	0.034	Low
9	Keningau	50	0.034	Low
10	Beaufort	50	0.034	Low
12	Lahad Datu	150	0.102	Moderate
13	Semporna	150	0.102	Moderate
14	Kota Marudu	50	0.034	Low

Table 4: Seismicity level for T = 0.2s

No	District	T=0.2s	2/3 factor	Seismicity
		(Gal)	g	
1	Kota Kinabalu	150	0.102	Low
2	Sandakan	200	0.136	Low
3	Tawau	200	0.136	Low
4	Ranau*	150	0.102	Low
5	Kundasang*	150	0.102	Low
6	Penampang	150	0.102	Low
7	Kota Belud	150	0.102	Low
8	Tambunan	150	0.102	Low
9	Keningau	150	0.102	Low
10	Beaufort	150	0.102	Low
12	Lahad Datu	250	0.170	Moderate
13	Semporna	250	0.170	Moderate
14	Kota Marudu	150	0.102	Low

3.2 Summary of Findings for RVS Data Collection Form

The Buildings involved in this screening varies from 3 to 8 stories. The number of stories for these buildings is shown in Figure 2. From RVS Data Collection Form, it is observed that there are 32 buildings were in the category of seismically hazardous and 22 buildings were in the category of acceptable seismic performance with passing score of 2.0. there were twenty-five (25) buildings located in the moderate seismicity area and twenty-nine (29) more in the low seismicity region. However, no high seismicity region was recorded in Sabah. It was also observed that the percentage of soil type D are much higher than soil type E, which is 83% and 17% respectively.

Figure 3 shows building benchmark consisting of pre-code, vertical irregularity and plan irregularity. Vertical irregularity shows the higher percentage with 58%, followed by plan irregularity with 33% and pre-code is 9%. These indicated vertical irregularity is one of the prominent factors that can influence the possibility of building failed.

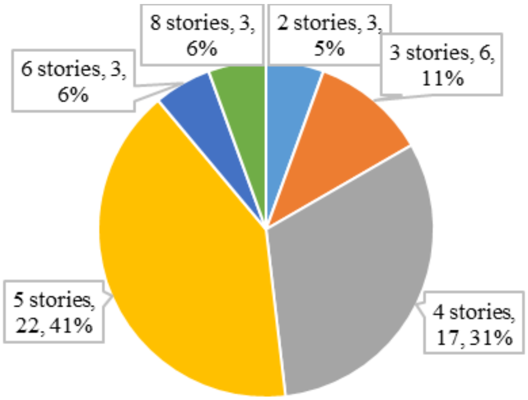


Figure 2: Numbers of stories in selected buildings

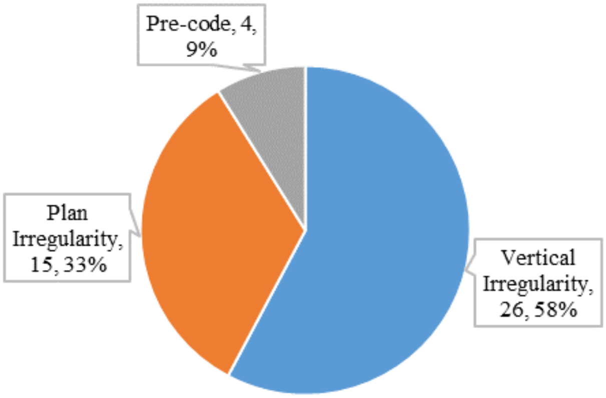


Figure 3: Building benchmark

3.3 ASCE 41-13 (Tier 1)

Tier 1 seismic assessment was performed in accordance with ASCE 41-13 (2014), Seismic Evaluation and Retrofit of Existing Buildings. The evaluation is based on as-built structural drawings provided by JKR Sabah and supplemented by site visits, to perform limited observations of the existing condition of the structures. No material testing or removal of building finishes was performed. All the buildings have been evaluated for the expected structural performance using ASCE 41-13 (2014) procedures.

3.3.1 Scope of Work

Tier 1 screening was required for all buildings to determine the selected building's risk due to seismic activity. It is also to quickly identify buildings that comply with the provision of this standard. If the building meets the benchmark building criteria, it is deemed to meet the structural requirements of this standard for the specified level of performance. In this report, all data collected are sufficient to conduct a Tier 1 evaluation. The evaluation process in Tier 1 screening phase is shown in Figure ***4***4.

However, before we proceed to Tier 1 Screening process, we have to determine Level of Performance and Level of Seismicity for each building. It is important part to identify the Tier 1 checklist. In this study, there are 5 checklists were used and completed according to the targeted building performance expected. For building that does not comply with the criteria outlined in the checklist, further investigation by Linear and Non-Linear structural analysis was done.

3.3.2 Level of Performance

Performance Level shall be defined before conducting a seismic evaluation using this standard. All the buildings were evaluated based on the Life Safety (LS) Performance Level. The Immediate Occupancy (IO) Structural Checklist was only filled up for specific critical buildings with high importance such as Hospitals, Schools, Police Stations and Fire Stations only.

3.3.3 Level of Seismicity

The Level of Seismicity of the building also be categorised as Very Low, Low, Moderate, or High in accordance to this ASCE 41-13 (2014) standard. Building's seismicity level was obtained from local macrozonation map recommended by JKR-UTM (2010). The level of seismicity can be defined based on values shown in Table .

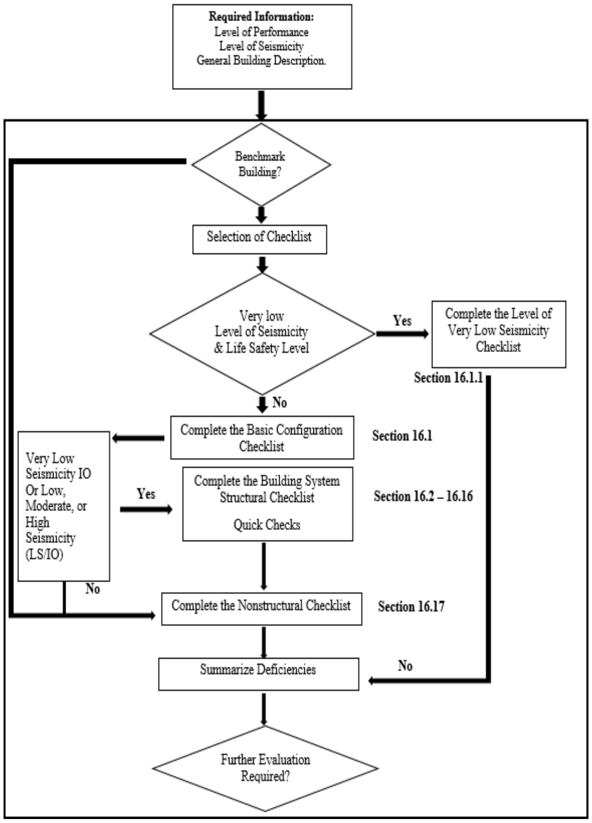


Figure 4: Tier 1 evaluation process (ASCE 41-13, 2014)

Table 5: Level of seismicity, SDS & SD1

Level of Seismicity	S_{DS}	S_{D1}
Very low	< 0.167 g	< 0.067 g
Low	≥ 0.167 g < 0.33 g	≥ 0.067 g < 0.133 g
Moderate	≥ 0.33 g < 0.50 g	≥ 0.133 g < 0.20 g
High	≥ 0.50g	≥ 0.20 g

3.3.4 Selection of Checklists

All these buildings are classified as Building Type C1: Concrete Moment Frames. Thus, the structural checklist associated with building Type C1 are used. The Tier 1 Evaluation of all these buildings involves completing the following checklist:

a. Basic Checklist for Building Type C1.
b. Life Safety Basic Configuration Checklist.
c. Immediate Occupancy Basic Configuration Checklist.
d. Life Safety Structural Checklist for Building Type C1.
e. Immediate Occupancy Structural Checklist for Building Type C1.

Each of the evaluation statements on the checklists shall be marked "compliant" (C), "noncompliant" (NC), or "not applicable" (N/A). Compliant statements identify issues that are acceptable according to the criteria of this ASCE 41-13 (2014), while non-compliant statements identify issues that require further investigation.

3.3.5 Pseudo Seismic Forces

The Pseudo Seismic force, in a horizontal direction of a building, shall be calculated in accordance with equation below:

$$V=CS_aW \tag{1}$$

where;

V = Pseudo seismic force

C = Modification Factor (Refer to table 4.8, ASCE 41-13, 2014)

S_a = Spectral acceleration at the fundamental period of the building,

W = Effective seismic weight of the building

3.3.5.1 Spectral Acceleration, S_a

The calculation of Spectra Acceleration is based on the formula:

$$S_a = S_{x1}/T \tag{2}$$

where $S_a < S_{XS}$

Fundamental period:

$$T = C_t h_n^{\beta} \tag{3}$$

$$S_{xs} = F_V/S_s \tag{4}$$

$$S_{x1} = F_V/S_1 \tag{5}$$

where; S_s = response spectrum ordinates for short period (0.2s) and

S_1 = for long (1.0s) periods, in the direction of maximum horizontal response.

3.3.5.2 Distribution of Story Shear Forces

The pseudo seismic force calculated above shall be distributed vertically accordance with following equation.

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \tag{6}$$

$$V_j = \sum_{x=j}^{n} F_x \tag{7}$$

where, Wx = Portion of the total building weight W located on or assigned to floor level x

Wi = Portion of the total building weight W located on or assigned to floor level x

k = 2.0 for $T \geq 2.5$ seconds

= 1.0 for $T \leq 0.5$ seconds

H_x = Height in ft from the base to floor level x

H_i = Height in ft from the base to floor level i

V = Pseudo lateral load

V_j = Story shear at story level j

3.3.5.3 Shear Stress in Concrete Frame Columns

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building. The average shear stress in the columns of concrete frames shall be computed in accordance with the equation below:

$$V_j^{avg}=1/M_s[n_c/(n_c-n_f)](V_j/A_c) \tag{8}$$

where, n_c = Total numbers of columns

n_f = Total numbers of frames in the direction of loading

A_c = Summation of the cross-sectional area of all columns in the story under consideration

V_j = Story shear computed in section 4.4.2

M_s= System modification factor; Mx shall be taken as equal to 2.0 for building being evaluated to the Life Safety Performance Level and equal to 1.3 for buildings being evaluated to the Immediate Occupancy.

3.3.5.4 Columns Axial Stress Caused by Overturning

Typical columns design only take into account the gravity load hence they may have limited additional capacity to resist seismic forces. Where axial forces caused by seismic overturning moments are added, the columns may crush due to excessive axial compression. Where both demand are large, the combined effect of gravity and seismic forces must be calculated to demonstrate compliance. The axial stress of columns in moment frames at the base subjected to overturning forces, P_{ot}, shall be calculated in accordance with the equation below:

$$p_{ot}=1/M_s(2/3)(Vh_n/Ln_f)(1/A_{col}) \tag{9}$$

where, n_f = Total numbers of frames in the direction of loading
V = Pseudo seismic force
h_n = Height (ft) above the base to the roof level
L = Total length of the frame (ft)
M_s = System modification factor; Mx shall be taken as equal to 2.0 for building being evaluated to the Life Safety Performance Level and equal to 1.3 for buildings being evaluated to the Immediate Occupancy.
A_{col} = Area of the end column of the frame.

3.3.6 Results and Findings

A list of deficiencies identified by evaluation statements for which the building was found to be non-compliant have been compiled upon completion of the Tier 1 Checklists. Further evaluation requirements have been determined once the checklists have been completed. Out of 54 buildings that have been recorded, there are thirty-two (32) buildings (59%) that have been analyzed in Tier 1 Screening Phase.

3.3.6.1 Vertical Irregularity (Soft Story)

Based on Figure 5, the total of buildings having "Soft Story" are twenty-five (25) buildings. These multi-story buildings with weak and/or open front wall lines creating a "soft-story" performed poorly than other buildings and may be subjected to structural failure during or after earthquake. Buildings that are most vulnerable have been identified with the following criteria:

- Column Axial Stress Caused by Overturning is more than $0.3f_c$ psi
- Shear Stress in Concrete Frame Columns is more than 100 lb/in^2

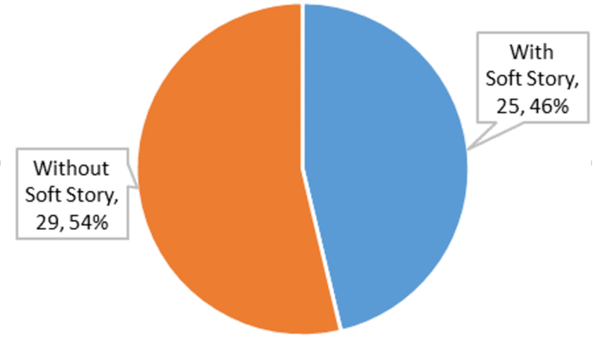
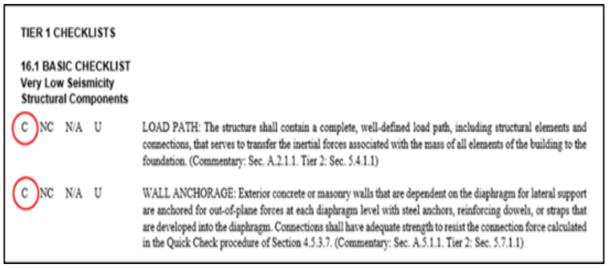


Figure 5: Total building with vertical irregularity

3.3.6.2 Basic Checklist

The following in Figure 6 is an example of completed Basic Checklist. In this report, most of buildings have complied with the basic checklist items such as load path and wall anchorage.


TIER 1 CHECKLISTS

16.1 BASIC CHECKLIST
Very Low Seismicity
Structural Components

[✓ C] NC N/A U LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

[✓ C] NC N/A U WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

Figure 6: Basic checklist (ASCE 41-13, 2014)

The wall anchorage check was based on the assumption of load wall has been transferred through slab diaphragm towards the adjacent beams. The typical top and bottom bars in slabs shall always having 90^0 bent down at the edge connected to adjacent beams. Calculation based on this typical detailing in Malaysia Practice, rebar size of R8 to R10 was widely used in construction shows sufficient capacity for wall anchorage.

3.3.6.3 Column Axial Stress Caused by Overturning

All 32 buildings were found to have adequate column axial stress especially caused by overturning and complied with the requirements of $30f_c$ psi in the standard.

3.3.6.4 Shear Stress in Concrete Frame Columns

Figure 7 shows the results for shear stress in concrete frame columns. Only two (4) buildings are found to be non-compliance to the shear stress requirements. Twenty-eight (28) buildings were found to be in compliance and exceeding the column shear stress requirement. The limiting value of shear stress should be less than the greater of 100 lb/in^2 or $2\sqrt{f'_c}$.

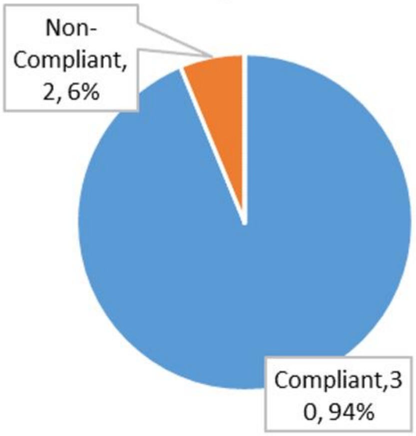


Figure 7: Shear stress in concrete frame columns

3.3.7 Findings from ASCE Tier 1 Screening

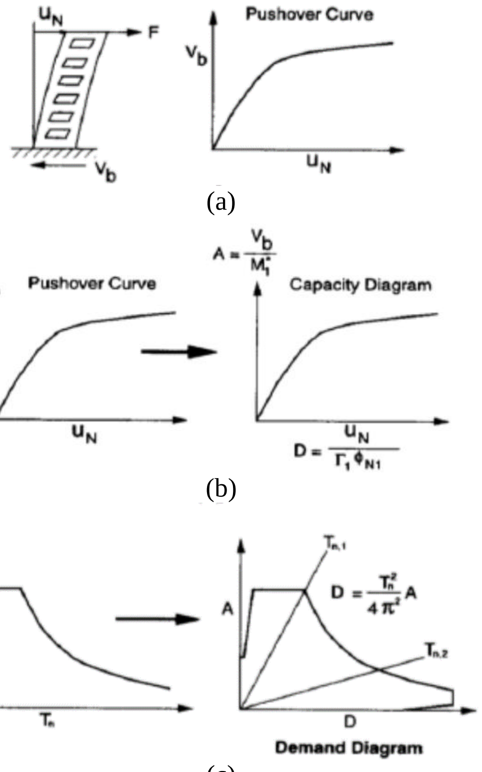
By conducting more evaluation items in the ASCE 41-13 (2014) Checklist, it can be determined that most of the buildings that have ‘Soft Story’ issues did not comply with requirements of Shear Stress Check. These buildings will be further analysed by using Column Axial Stress Check and Shear Stress in Concrete Frame Columns Check.

In the Summary of the ASCE 41-13 (2014) Tier 1 Screening Phase as in this report, there are several items have been deemed “non-compliant” and represent potential structural deficiencies and thus requiring further analysis which is by using Structural Non-Linear analysis.

4. Phase III-Develop Demand/ Capacity Analysis (DCA) and Fragility Evaluations

Most buildings that are designed for seismic resistance using elastic analysis will experience significant inelastic deformations under large scale earthquakes. Modern performance-based design methods require ways to determine the realistic behaviour of structures under such conditions. Enabled by advancements in computing technologies and available test data, nonlinear analyses provide the means for calculating structural response beyond the elastic range. Under both codes ASCE 41-13 (2014) and MS EN 1998 (2015) , Non-linear static analysis (Pushover) is recommend for those buildings which are defined with irregularities. ATC-40 (1996) developed by Applied Technology Council provided the basic guidance of Pushover analysis shall be implemented.

The two key elements of the Pushover procedure are the demand and capacity. Demand is a representation of the seismic response expressed by the response spectrum diagram. While the capacity of a structure in the inelastic state could be defined by the load displacement curve through series of applied incremental load. Both demand and capacity curve are independent in nature. As the demand increases, the structure would yield eventually, as the stiffness decrease and prolonging the period. Since the seismic acceleration depends on period, demand will also change as the structure yields. By capacity spectrum conversion method, the capacity and demand spectrum was converted to matching format as indicated in Figure 8. By overlapping both diagrams, an intersection point of both curve will represent the building’s actual performance point as shown in Figure 9.



(a)

(b)

(c)

Figure 8: Conversion of demand spectrum curves

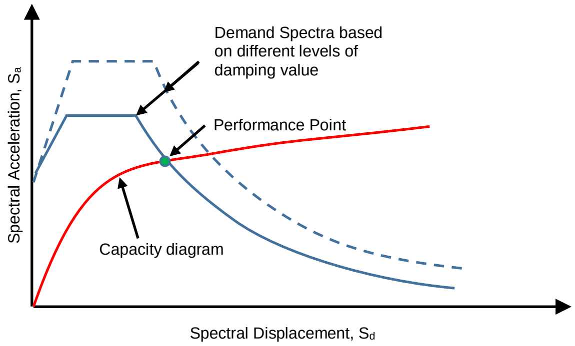


Figure 9: Determination of performance point

4.1 Pushover Analysis

Under the Pushover Analysis exercise, a total of fifteen (15) numbers of building were analysed. The demand capacity curves are shown below in Figure 10 to Figure 12. It is noticed that for some buildings Nos. 2, 3, 5, 12 and 14, the performance point was found to be located within the elastic zone under MS EN 1998 (2015) response spectrum. This behavior may due to the buildings uniform shape and with minimal irregularities. Some of them even remaining within the linear elastic when intersecting with much higher intensity Malaysian National Annex (2015) response spectrum. Whereas the building no. 6 shows the most flexible structural behavior and having most of the performance point at large deformation level.

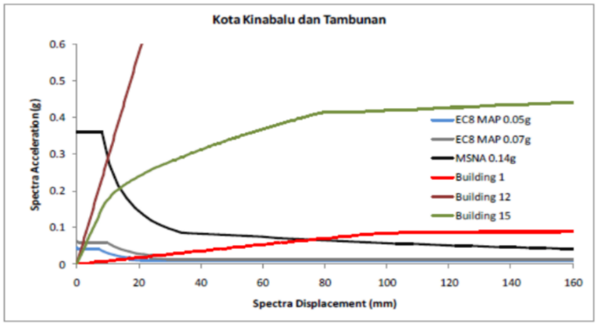


Figure 10: DCA curves- Kota Kinabalu and Tambunan

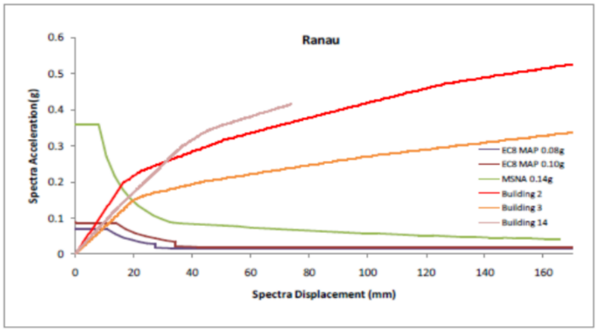


Figure 11: DCA curves- Ranau

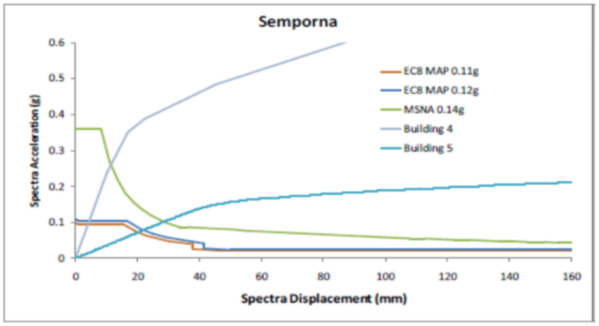


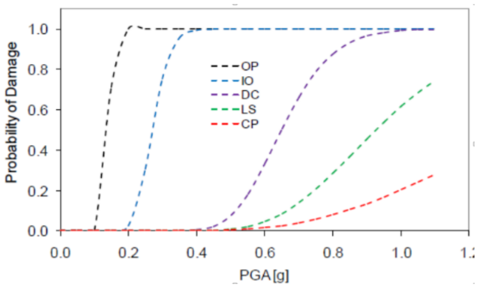
Figure 12: DCA curves- Semporna

4.2 Fragility Curves

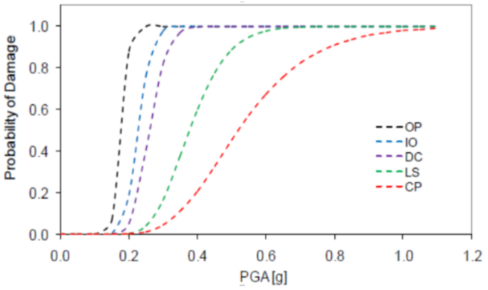
The incremental dynamic analysis (IDA) was carried out by performing series of nonlinear response history analyses based on ground motions that are systematically scaled to increasing earthquake intensities until collapse occurs. Incremental dynamic analysis yields a distribution of results at varying intensities that can be used to generate a collapse fragility. In this study, six (6) types of recorded ground motion as listed in Table 6 were adopted in the development of fragility curve. Among the fifteen (15) analysed buildings, two (2) hospitals (building nos. 8 & 9) shows the highest probability of damages which is 100% and 40% respectively at Operational Performance level under 0.2g PGA. However, both buildings are expectected to be able to perform by less than 3% of fragility under Life Safety performance level as shown in Figure 13. Fragility curve offer the great flexibility of hazard assessment for a typical building subjected to various performance level and seismic demand. It can be utilised as a powerful tool to provide early estimate over vast area of affected region with ease within short period of time.

Table 6: Time History Ground Motion

No	Type	PGA intensity	Location
1	OPACO	1.17g	North America
2	EL-CENTRO	0.31g	
3	POMONA	0.16g	
4	KKMRanau	0.13g	Ranau
5	KDMRanau	0.003g	
6	SPMRanau	0.005g	



(a) Building No. 8



(b) Building No. 9

Figure 13: Fragility curve

4.2.1 Phase IV – Retrofitting Proposal

From total numbers of 54 government buildings selected for this study, 4 buildings were ranked as the most vulnerable building having insufficient capacity to resist earthquake forces and need to be further investigated for their strength requirement and suggested to be retrofitted. Since there was no local document available to be referred, the retrofitting works described in this report were solely based on FEMA 547 (2006). The rehabilitation techniques for building Type C1 will be considered in this report.

4.3 Seismic Deficiencies and Rehabilitation Measures

Failure to meet stipulated criteria in seismic evaluation will identify certain seismic deficiencies. In general, deficiencies were categorized into 7 main categories:

a. Global strength
b. Global stiffness
c. Configuration
d. Load path
e.Component detailing
f. Diaphragms
g. Foundations

In the traditional sense of improving the performance of the existing structure, there are 3 basic classes of measures taken to retrofit a building:

a. Add elements, usually to increase strength or stiffness.
b. Enhance performance of existing elements, increasing strength or deformation capacity.
c. Improve connections between components, assuring that individual elements do not become detached and fall, a complete load path exists, and that the force distributions assumed by the designer can occur.

5. Retrofitting Techniques

For this particular report, all the four (4) buildings that were ranked as the most vulnerable building showing deficiencies specifically in the categories of global strength, global stiffness and configuration. The most common deficiencies found were vertical irregularity (soft story) and plan irregularity. Therefore, this report will emphasize the retrofitting technique for these deficiencies.

5.1 Add Steel Braced Frame (Connected to a Concrete Diaphragm)

Addition of steel diagonal braced frames to an existing concrete frames building is a method of adding strength and/or stiffness to the structural system. The steel braces can be added without significant increase to the building weight. Any variety of the diagonal brace configurations may be used, as well as a variety of brace member section types. Figure 14 shows several common configurations

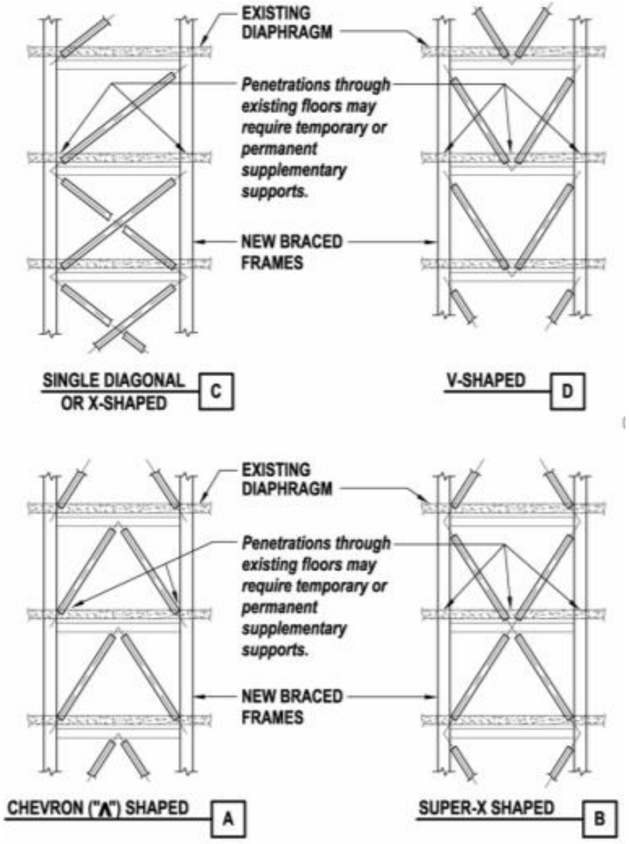


Figure 14: Typical braced frame configurations (FEMA 547, 2006)

5.2 Add Concrete or Masonry Shear Wall (Connected to a Concrete Diaphragm)

Addition of shear walls to an existing concrete frame building is a common method of adding significant strength and/or stiffness to the structure. The new walls may be of cast-in-place concrete, shotcrete or fully grouted concrete masonry unit (MCU) construction. Figure 15 shows typical concrete wall connection to concrete slab.

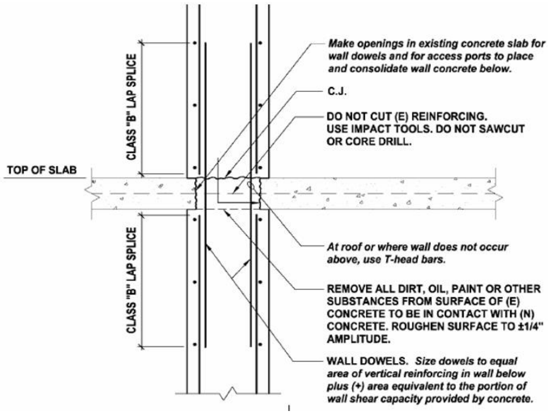


Figure 15: Concrete wall connection to concrete slab (FEMA 547, 2006)

5.3 Enhance Column with Fiber-Reinforced Polymer Composite Overlay

The use of a fiber-reinforced polymer (FRP) overlay with columns has proven to be an efficient rehabilitation technique in the building construction industries. Columns are overlaid with unidirectional fibers in a horizontal orientation, thus providing shear strengthening and confinement similar to that provided by hoops and spirals used with circular columns, and stirrups and ties used with rectangular columns. The confinement enhances the concrete compression characteristics, provides a clamping action to improve lap splice connections, and provides lateral support for column longitudinal bars. Figure 16 shows a typical seismic retrofit of columns using FRP composites.

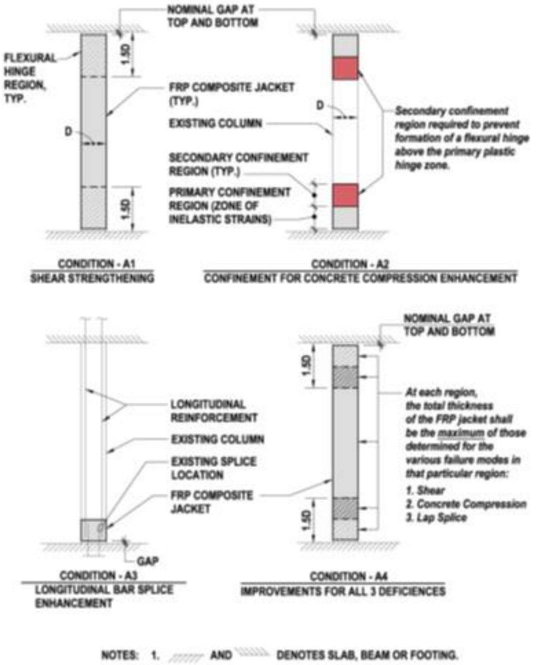


Figure 16: Seismic retrofit of columns using FRP composites (FEMA 547, 2006)

5.4 Enhance Concrete Column with Concrete or Steel Overlay

Adding a fiber reinforced polymer (FRP) composite overlay to a concrete column is a recent approach to addressing seismic deficiencies. Adding a concrete or steel jacket is a more traditional method of enhancing a deficient concrete column. Figure 17 shows an examples of concrete and steel jacketing for a rectangular column.

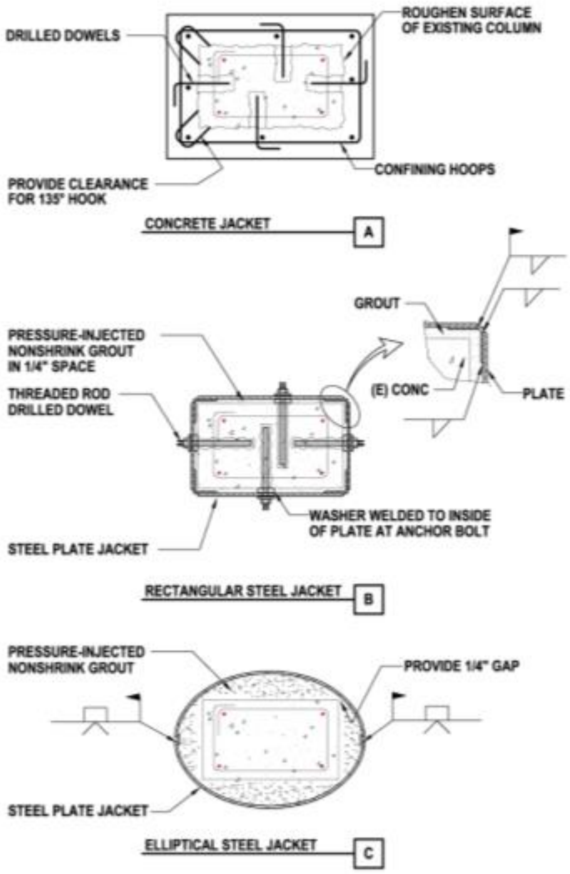


Figure 17: Concrete and steel overlays for concrete columns (FEMA 547, 2006)

5.5 Enhance Concrete Moment Frame

An alternative method to either add strength and stiffness to an existing concrete moment frame or correct non-ductile detailing deficiencies of the frame members is by direct enhancement: increasing the size of the columns and beams of the frame with new reinforced concrete. This method entails adding a jacket of reinforced concrete around the existing columns and beams, an approach similar to jacketing by steel or fiber wrap. The new concrete may be either cast-in-place or shotcrete.

6. Conclusion

Generally, most buildings under the study was found behave linearly elastic under the Pushover analysis. However, the deficiencies such as inadequacies of shear stress discovered from ASCE 41-13 (2014) checklist shall be retrofitted. In order to have the appropriate choice of retrofitting option, multiple techniques and procedures mentioned above could be applied with some deeper thought in term of constructability, cost, targeted seismic performance as well as the aesthetical value aspect.

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