- Technical Paper -

Analysis of a School Building Damaged by the 2015 Ranau Earthquake Malaysia

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ABSTRACT

To evaluate the seismic resistance of the existing buildings in Malaysia built by the British Standard, a school building that was damaged at the 2015 Ranau earthquake is analyzed by STERA_3D software which was developed by the author. The earthquake response analysis was conducted using the modified time history acceleration data measured at the Ranau earthquake by the seismograph installed at Kota Kinabalu station. By comparing the results of the analysis and the actual damage of the building, the reason that caused damage to the building is clarified.

Keywords: Ranau earthquake, earthquake damage, earthquake response analysis.

1. INTRODUCTION

1.1 Ranau Earthquake

The 2015 Ranau earthquake with Magnitude 6.0 occurred at the foot of mount Kinabalu near the highland town of Kundasang, Malaysia, at 07:15 on June 5th, 2015. The epicenter is 5.987° north 116.541° east. Fig. 1 shows the locations of the observation station, Kota Kinabalu (KKM), that is 54.7km far from the epicenter and the target school building of this stusy in Ranau that is 14.9km far from the epicenter. The maximum acceleration of the ground motion observed at the KKM station by MMS (Malaysia metrological service) is 121 gal (cm/sec^2) in NS direction, 132gal in EW direction and 51gal in UD direction [2]. More than 120 aftershocks of Magnitude above 2 were recorded during the three months after the main shock [1]. The main shock and aftershock generated landslides and rock falls which caused damage to roads and bridges and substantial damage to several villages located at the foot of the mountain. Also water supply system in the Ranau and Kota Belud districts faced acute shortages. 18 people were killed due to rockfalls [1].

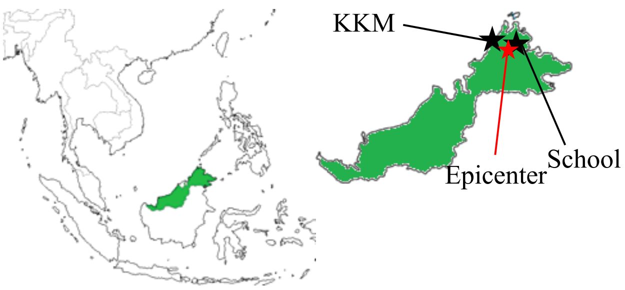

Fig. 1 Location of Ranau, Sabah, Malaysia

1.2 Building Damage

The main shock caused moderate damage to infrastructure and public buildings such as schools, hotels, hospitals etc. and private buildings such as shops and residential houses around Ranau and Kundasang areas [1]. Fig. 2 shows the damage which observed at the target school building due to the earthquake. The structural damage of the building was concentrated on the 1st story masonry walls, columns and column-beam connections.

Since the structure of buildings in Malaysia is designed by the British Standard (BS) without consideration of earthquake loads, the seismic resistance of the Malaysian buildings is unclear. To evaluate the seismic resistance of Malaysian buildings, the target school building is analyzed using the STERA_3D software [4] developed by one of the authors.

(a) Column (b) Infill wall (c) Column

Fig. 2 Actual damage observed at the earthquake

1.3 Evaluation method

Seismic resistance of the building is evaluated by two methods; one is the push-over analysis to verify the lateral resistance of the building another one is the dynamic response analysis using input ground motions.

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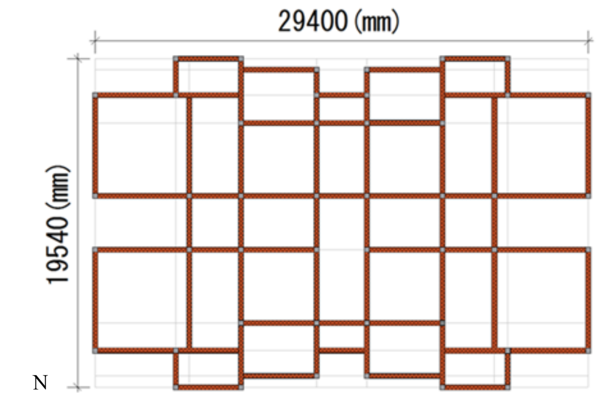
2. OUTLINE OF TARGET BUILDING

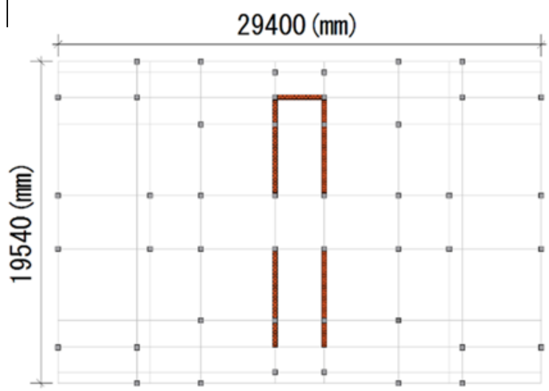
A school building that was damaged at the Ranau earthquake is selected as the target building. The satellite photo of school complex and the exterior of one of the buildings are shown in Fig. 3. The target building is a four-story building with a height of 11.2 m constructed in 2004 and 1st floor is designed to be a parking space without walls except the staircase area (Fig.4). The structural type is infill masonry where the main frame is configured by reinforced concrete.

(a) School complex (b) Target building

Fig.3 The school complex and target building [3]



a) 2nd to 4th floor



(b) 1st floor

Fig.4 Floor plans of the target building

2.1 Building Model

The STERA_3D model of the target building is shown in Fig. 5. This software can introduce the force-deformation relation from structural component directly. The beam is modeled as a line element with the nonlinear flexural springs at the both ends and a nonlinear shear spring at the center. The degrading tri-linear slip model is used for the flexural hysteresis. The column is modeled in a similar manner, while the nonlinear interaction between axial force and moment is expressed using axial springs of concrete and steel arranged in the section at both ends (so called MS-model) and the nonlinear shear characteristics are modeled by the nonlinear shear springs. The hysteresis model of the masonry wall is defined as the poly-linear slip model. The detail is described in the technical manual of STERA_3D [4].

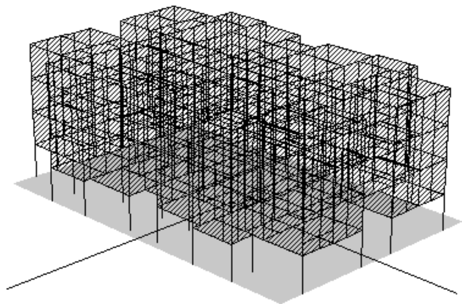


Fig.5 STERA_3D building model

2.2 Structural Information

The list of compressive strength and unit weight of structural material is shown in Table 1. Fig. 6 shows the representative sections of column and beam. Table 2 shows the floor weight of each floor.

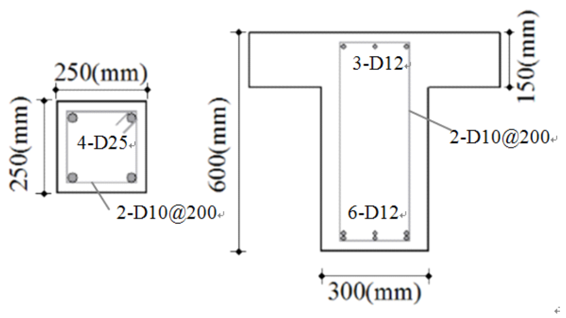


Fig.6 Representative sections of column and beam

Table 1 Material strength and unit weight

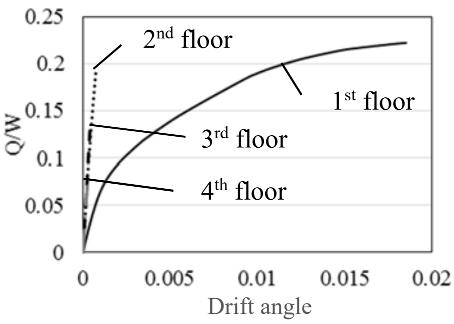
Materials	Compressive and tensile strength (N/mm^2)	Unit weight (kg/m^3)
Concrete	28	2447
Steel Bar	Main Bar: 460 (yield strength)	7852
	Stirrup: 250 (yield strength)	
Brick	21	2192
Mortar	10	2345

Table 2 Weight of the building in each floor

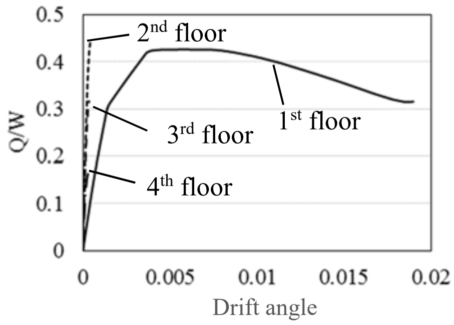
Floor	Live Load(kN)	Dead Load (kN)	Total Weight (kN)
4	380	4335	4715
3	380	6188	6568
2	380	6159	6539
1	380	5595	5975

3. EVALUATION OF LATERAL RESISTANCE BY PUSH OVER ANALYSIS

Fig. 7 shows the relationship between shear force and story drift angle in each story obtained by the push-over analysis in the longitudinal direction and the transversal direction. The deformation is concentrated in the 1st floor and the base shear coefficients are 0.22 in the longitudinal direction and 0.41 in the transversal direction. The building has parking space in the 1st floor and it has no masonry walls expect staircase area and much masonry walls in the upper floors therefore deformation is concentrated in the 1st floor. To compare both directions, shear coefficients on longitudinal direction is smaller than transversal direction. It is because in the longitudinal direction, it has less masonry walls to load shear force than transversal direction.



(a) Longitudinal direction



(b) Transversal direction

Fig. 7 Relationships between shear force and drift angle by push-over analysis

4. EVALUATION OF EARTHQUAKE GROUND MOTIONS AT SCHOOL SITE

To obtain the ground motion at the school site, the observation records at the KKM were amplified by considering the difference of epicenter distance and the ground condition. As shown in Fig. 8 there are two phases to estimate the ground motion at the school site. First, decrease the ground motion measured at KKM to its bedrock due to ground condition differences (① in Fig. 8). Next, increase the ground motion at the bedrock at KKM to estimate the ground motion at the surface of the school site based on the attenuation formula (② in Fig. 8).

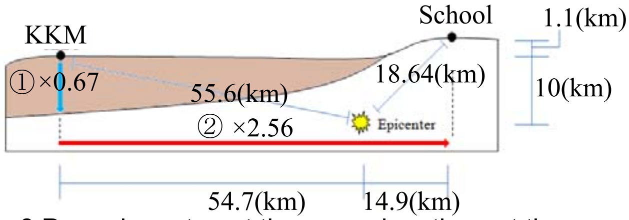


Fig.8 Procedures to get the ground motions at the building site

4.1 Soil Amplification Factor (EURO CODE)

The site classes of KKM and the school site are determined based on the soil classification of Euro Code [5]. As shown in Table 3. The site class of the KKM is classified as site class C (medium dense sand) based on standard penetration test [6]. The site class of the school site is classified as site class A (rock) based on the micro tremor measurement [3]. Bedrock is also classified as the site class A (rock). Based on the hazard and risk assessment in Malaysia [7], Table 4 shows the spectral parameters in each soil class.

Equation (1) and Fig. 9 (a) shows the function and shape of the response spectrum in Euro code. The design response spectrums at those sites are calculated as shown in Fig. 9 (b). From the design spectrum on both sites, the amplification of ground motion due to site class differences is 0.67 (=1.0/1.5) that showed in Fig. 8 procedure ①.

Table 3 Site class properties of Euro Code [5]

Site class	Description	V(m/s)
A	Rock	V>800
B	Dense sand	360<V<800
C	Medium dense sand	180<V<360
D	Medium cohesionless soil	V<180
E	Surface alluvium layer	

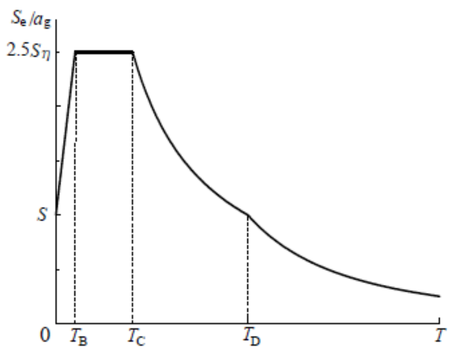
Table 4 Parameters of design spectrum [5]

Site Class	S	T_B (s)	T_C (s)	T_L (s)
A	1.0	0.05	0.25	1.2
C	1.5	0.1	0.25	1.2

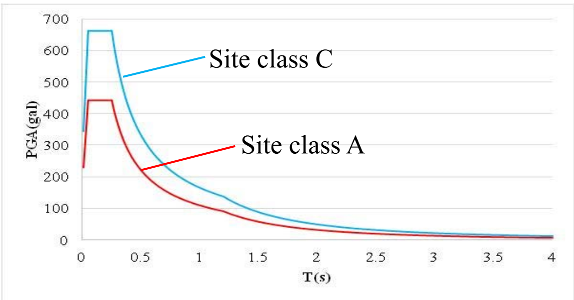
$$S_e(T) = \begin{cases} \alpha_g \cdot S \cdot \left[1 + \frac{1.5T}{T_B}\right] & (0<=T<=T_B) \\ 2.5 \cdot \alpha_g \cdot S & (T_B<=T<=T_C) \\ 2.5 \cdot \alpha_g \cdot S \cdot \left[\frac{T_c}{T}\right] & (T_C<=T<=T_D) \\ 2.5 \cdot \alpha_g \cdot S \cdot \left[\frac{T_c \cdot T_D}{T^2}\right] & (T_D<=T<=4s) \end{cases} \quad (1)$$

where,

$S_e(T)$: elastic response spectrum
α_g : design ground acceleration on type A ground
T : vibration period of a liner single degree of freedom system
T_B : lower limit of the period of the constant spectral acceleration branch
T_C : upper limit of the period of the constant spectral acceleration branch



(a) Euro code design response spectrum



(b) Response spectrum of site classes A, C

Fig.9 Design response spectrum

4.2 Attenuation Formula

Two different attenuation formulas are examined. The Andres, Hilmar and Leif attenuation formula [8] is proposed by collecting inter-plate earthquake data which recorded in North America, Europe, China and Australia. The distance used in this formula is diagonal distance from the epicenter. The formula is shown in Equation (2). Fukushima and Tanaka attenuation formula [9] is proposed by collecting earthquake data which recorded in Japan. The distance used in this formula is horizontal distance from the epicenter. The formula is shown in Equation (3).

Since the building is located 14.9(km) in horizontal distance and 18.64(km) in diagonal distance from the epicenter, the earthquake ground motion at the building is considered larger than the observed records in KKM at 54.7(km) in horizontal distance and 55.6(km) in diagonal distance from the epicenter. Fig. 10 shows the relation of both attenuation formulas. It is clear that reduction of PGA is more severe if distance is going to be far in case of the Fukushima and Tanaka attenuation formula than the Andres, Hilmar and Leif attenuation formula. In addition, the PGA on the KKM using the Andres, Hilmar and Leif attenuation formula is closer to the PGA of the observed records than the PGA using Fukushima and Tanaka attenuation formula as shown in Table 5. Therefore, this study adopts the Andres, Hilmar and Leif attenuation formula and the amplification of the acceleration between KKM and the school side is estimated as 2.56 as shown in Fig. 8 procedure ②.

$$InA=-1.471+0.849M+0.00418R+In(R^{-1}) \quad (2)$$

where,

A : peak acceleration of the horizontal component (gal)
M : magnitude
R : hypocentral distance (km)

$$Iog_{10}A=-0.42M-1og_{10}(R+0.025(10^{0.42Mw})) -0.0033R+1.22 \quad (3)$$

where,

A : peak acceleration of the horizontal component (gal)
Mw : magnitude
R : shortest distance between site and the fault rupture (km)

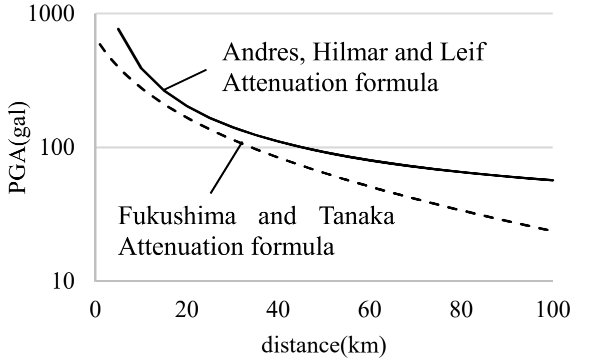


Fig.10 comparison of attenuation formula

Table 5 Distance and PGA differences on both attenuation formulas

Site	KKM		School	
Parameters	Distance (km)	PGA (gal)	Distance (km)	PGA (gal)
Recorded earthquake	/	121(EW) 132(NS) 53 (UD)	/	?
Andres	55.6	84.9	18.64	217.2
Fukushima	54.7	57.6	14.9	211.8

4.3 Ground Motion at the School Site

The procedures to estimate ground motion at school site are described below.

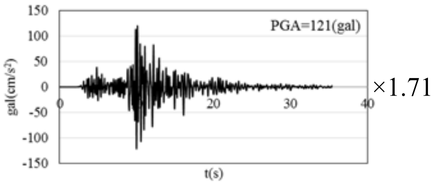
① Estimate bedrock ground motion at KKM
From Fig. 9, the ground motions measured on the surface of KKM (site class C) are reduced 0.67 at the bed rock (site class A)

② Estimate ground motion at school site
From Fig. 10, the ground motions at the bedrock of KKM (55.6km from the epicenter) are amplified 2.56 at the school site (18.64 km from the epicenter)

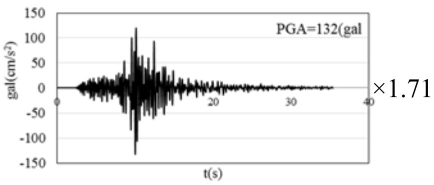
Summarizing procedures ① to ②, the ground motions at school are estimated by multiplying the following factor (1.71 = 0.67×2.56).

$$\text{Ground motion(school)} = 1.71 \cdot \text{Ground motion(KKM)}$$

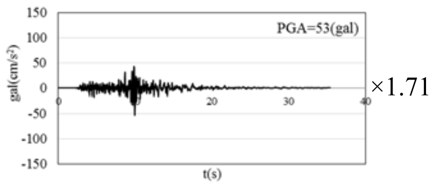
Fig. 11 shows the earthquake acceleration records at KKM. The ground motion at school site should be estimated by multiplying 1.71 to the ground motions at KKM.



(a) EW component



(b)NS component



(c) UD component

Fig.11 Earthquake ground motion at KKM

5. THE TIME HISTORY ANALYSIS OF BUILDING USING MODEIFIED EARTHQUAKE GROUND MOTIONS

The nonlinear time history analysis of the building is conducted by STERA_3D using the estimated earthquake records at the school site.

5.1 Deformation of the Building

The result of the maximum story drift angle is shown in Fig. 12. Maximum drift angle is 0.004 at the 1st floor in the longitudinal direction and 0.003 at the 1st floor in the transversal direction. It is clear that the drift of the first floor is severe in the both direction.

Fig. 13 shows the location of damaged elements in the 1st floor. Fig. 14 shows the actual damaged part of the building. The damage is appeared in the infill wall at the staircase in the STERA_3D analysis. Actual damage of the target building is also appeared at the same infill wall parts. Therefore, the characteristic damage pattern of the analysis is consistent with the actual damage.

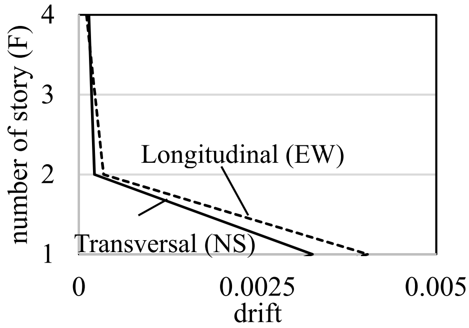


Fig.12 Maximum drift angle of the building in both directions

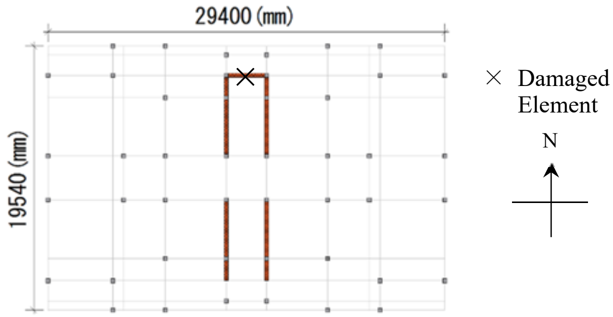


Fig.13 Location of damaged elements of time history analysis

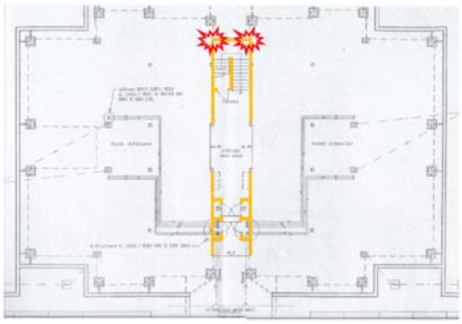


Fig. 14 The damaged parts of the building due to Ranau earthquake [3]

5.2 Natural Period of the Building

Lim [3] conducted the micro tremor measurement in 23rd and 24th January 2016 to estimate the dynamic characteristics of the school building. In his study, there are three micro tremor instruments installed in the target building. Positions of the micro tremor instruments were located at the center of mass of target building.

Fig. 15 show the locations of the measurement points in the target building. Table 6 shows the natural periods obtained by the micro tremor measurement and STERA_3D before and after the earthquake.

In the longitudinal direction, the natural period of the micro tremor measurement exists between the natural period of STERA_3D (before) and that of STERA_3D (after). However, the natural period in transversal direction of the micro tremor measurement is less that the natural period of STERA (before). To solve this problem, more detailed analysis model of masonry wall members is required.

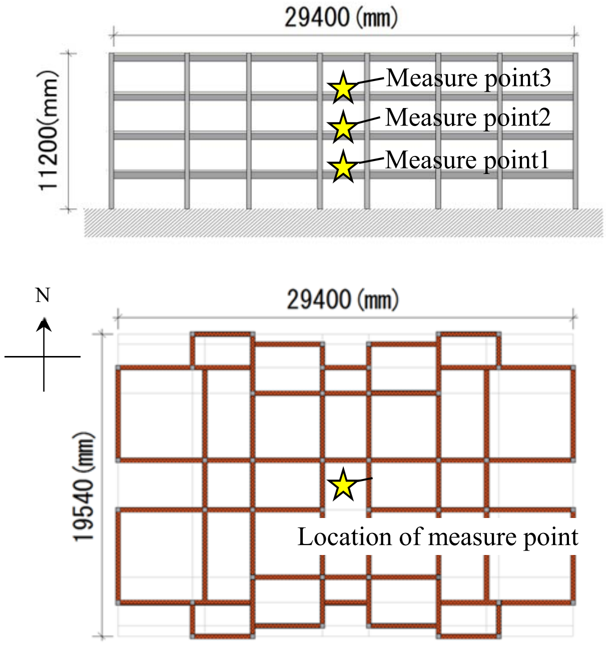


Fig. 15 Location of measurement points at different floor levels

Table 6 Natural period of the building

Direction	Micro tremor	STERA 3D (Before earthquake)	STERA 3D (After earthquake)
Longitudinal	0.571(s)	0.411(s)	0.650(s)
Transversal	0.299(s)	0.311(s)	0.504(s)

6. CONCLUSION

The summary of this study is itemized as follows:

- From the analysis of STERA_3D, the estimated damage is found in the same location of the actual damage of the target building.
- The deformation is concentrated in the 1st story because of the lack of wall elements.
- The earthquake ground motion is relatively strong because of the short distance from the epicenter.
- The building is designed by British Standard without consideration of earthquake load and its seismic resistance was not strong enough to withstand the earthquake.

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