

SEISMIC ANALYSIS OF DAMAGED BUILDINGS BASED ON POST-EARTHQUAKE INVESTIGATION OF THE 2018 PALU EARTHQUAKE

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ABSTRACT: On September 28, 2018, Palu city in the Central Sulawesi area, suffered the M7.5 earthquake. An epicenter of its earthquake located 77 km from the capital of Central Sulawesi Province, Palu city. The main earthquake, followed by a localized 4 to 7 meters high tsunami, which swept shore-lying houses and buildings on its way. A large number of damages on reinforced concrete (RC) buildings, houses, and soil liquefaction spread in Palu city were observed after the earthquake. The post-earthquake observation, as is reported in this paper, was conducted in order to define the typical pattern of the buildings' damages. The field investigation was focused on the damaged RC frame of buildings' structures. Several types of damage were detected on RC structures such as collapse due to the soft story, damage to beam-column joint, failure of short column, shear failure of the column, and collapsed of brick masonry infills. Two damaged RC frame buildings, i.e., one was severely damaged and the other in totally collapsed, were furthermore analyzed to define their seismic capacity based on Japanese standard. According to the analyses of the first story in these buildings, the distinct difference of seismic performance of both buildings was discovered. The severely damaged building had a higher lateral strength index when compared to the collapsed building because of its structure able to maintain the lateral strength within large deformation.

Keywords: post-earthquake investigation, reinforced concrete, seismic performance, the 2018 Palu earthquake

1. INTRODUCTION

Indonesia is one of the countries located in the highest seismic region in the world, which has experienced many strong earthquakes during the last decades[1]. The series of earthquakes have struck several areas in Indonesia, such as Aceh in 2004 and 2016 [2], Bengkulu in 2007 [3], Yogyakarta in 2006, [4], West Sumatera in 2009 [5,6], Lombok 2018, and Central Sulawesi in 2018. The M7.5 earthquake struck Central Sulawesi on Friday, September 28, 2018, at about 6:03 pm local time in Indonesia. According to the U.S. Geological Survey (USGS) [7], the epicenter of the M7.5 earthquake was at 0.256°S, 119.840°E, with a shallow depth of 10 km and it was 70 km away from the provincial capital, Palu city. The earthquake rocked areas around the epicenter is shown in Fig.1. Its earthquake was followed by a tsunami with waves up to 7 m high.

The Palu earthquake of a M7.5 in September 2018 was preceded by a series of small to the moderate magnitude of the earthquakes over the hours leading up to the big one. The first major earthquake was in the magnitude of M6.1 that occurred three hours earlier and located at the southern of the event of M7.5, as was reported by the USGS. There was also followed by the

successive aftershock sequence, with about ten events of M4.7, and it got more significant in the three hours after the earthquake.

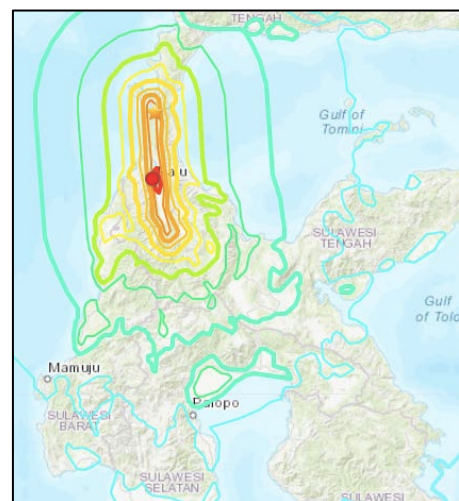


Fig.1 Shock map of M7.5 Palu earthquake (Source: USGS)

The earthquake and tsunami destroyed large residential areas, damaged more than 70,000 houses and nearly 3,000 schools. The earthquake also triggered soil liquefaction and settlements in Palu,

Donggala, and Sigi areas. Many multi-story RC buildings were severely damaged and collapsed. They include the Anuntapura Hospital, the Palu's Tatura Mall, the Roa-Roa Hotel, the Fire Station, and several buildings at Tadulako University. According to the National Disaster Management Authority (BNPB), the earthquake and tsunami caused more than 3,000 death and more than 4,000 injuries in Palu, Donggala, Sigi, and nearby areas. This paper presents the field-investigation results, which were conducted from October 4 to 6, 2018, around Palu city after the Palu earthquake. The observation was focused on evaluation and discussion on the seismic performance of the damaged RC frame buildings caused by the Palu earthquake.

2. OBSERVATION OF DAMAGED BUILDINGS

The locations of observation of the damaged RC buildings were in the areas of Palu, Donggala, and Biromaru. Figures 2(a) and 2(b) show the Sulawesi island and observed area, respectively.



a). Sulawesi island and location of Palu city (Source: Google map)



b). Observed areas (Source: Google map)

Fig.2 Map of surveyed areas

The observation was started in the Donggala area, about 40 km west-north of Palu city. The Palu earthquake destroyed most of the non-engineered houses, which were built in confined brick masonry wall types. The typical damages of these houses are shown in Photo 1.

The next investigation was performed in Palu city that was focused on collapsed and damaged buildings such as the collapsed buildings of Universitas Terbuka, Tadulako University (Forestry Faculty, Law Faculty, and Political Science Faculty buildings), The Sya Regency Hotel, Ramayana shopping center, Anuntapura Hospital, Mecure hotel, Fire Station and Public Apartment and several shophouse buildings. They were RC framed buildings with masonry infills, as shown in Photo 2.

In the district Biromoru of Sigi area, the massive liquefaction caused houses to be swept away and damaged public facilities, as shown in Photo 3.



Photo 1 Damage on confined masonry structures



Photo 2 Collapsed and damaged RC buildings: a) Mercure hotel, b) Politic Science Faculty building of Tadulako University, c) Public apartment, d) Shophouse



Photo 3 Damage due to liquefaction: a) Swept away homes, b) Damaged fuel station facility

3. TYPICAL DAMAGE OF RC STRUCTURES

Based on-site investigation, deficiencies were observed on damaged RC buildings according to Indonesian codes of SNI 03-2847-2002 [8] and SNI 03-2847-2013 [9], such as the use of plain round rebars for longitudinal and shear reinforcements of columns and beams, the absence of stirrups in beam-column joint regions, the use of stirrups with 90-degree anchorage on columns and beams, etc. These deficiencies might cause damage to building structures, as follows;

1. The total collapse of the first story that was classified as soft-story collapse [10], (Photo 4a);
2. Failure of beam-column joint due to lack of transversal reinforcements in the joint region (Photo 4b);
3. Shear failure of column caused by the use of shear reinforcements with 90-degree hooks, inadequate anchorage of hooks (Photo 4c) and the use of small rebars for stirrups;
4. Columns failed in flexure due to buckling the longitudinal reinforcements (Photo 4d);
5. Short columns failed in shear (Photo 4e);



Photo 4 Typical damage of RC structures: a) The first story collapsed due to the soft story, b) Failure of beam-column joint, c) Shear failure of column d) Buckling of longitudinal reinforcements of column e) Shear failure of short columns

4. SEISMIC ANALYSIS OF DAMAGED BUILDINGS

Two damaged RC buildings, i.e., the Forestry Faculty of Tadulako University and the Fire Station buildings, were thoroughly investigated in this study. Specifics of the building, correlated to cross-sectional dimensions of structural components, arrangements of reinforcing bars, and damages to columns, were collected on-site investigation. The Faculty of Forestry building, which was collapsed on the first story, as shown in Photo 5(a), was a three-story RC frame structure building located at Jl. Soekarno-Hatta KM 9 Tondo, Mantikulore, Palu. The Fire Station building was two stories RC frame structure building located at Jl. Balai Kota Timur No. 1, Tanamodindi, Mantikulore, Palu. The Fire Station building survived during the earthquake, but most of its columns were damaged, as shown in Photo 5(b). The calculation to identify the damaged grade of the Fire Station building is described as follows.



a) The Forestry Faculty building of Tadulako University



b) The Fire Station building and its damaged columns

Photo 5 The investigated damage buildings

4.1 Damage Grade Evaluation of Buildings

The evaluation was conducted for the first story to identify the damage grade of the Fire Station building. Based on the reference of Nagano [11], damage to the columns was grouped into five classes classified according to Table 1.

Table 1 Damage class definition of RC columns

Damage class	Descriptions of damage
I	- Visible narrow cracks on a concrete surface (crack width of less than 0.2 mm)
II	- Noticeable evident cracks on a concrete surface (crack width of about 0.2–1.0 mm)
III	- Local crushing of concrete cover - Remarkably wide cracks (crack width of about 1.0–2.0 mm)
IV	- Remarkable crushing of concrete with exposed reinforcing bars - Spalling off concrete cover (crack width of more than 2.0 mm)
V	- Buckling of reinforcing bars - Cracks in core concrete - Observable vertical and/or lateral deformation in columns and/or walls - Visible settlement and/or leaning of building

the first story of the Fire Station building are indicated in Fig. 3. The cross-sectional dimension of column C1 was 350x650 mm with 12 ϕ 19 longitudinal reinforcements and a hoop of ϕ 8-100. The damage degree was decided according to the residual capacity index, R, calculated by Eq. (1) [11].

$$R = \frac{\sum_{j=0}^5 \eta_j \cdot A_j}{A_{org}} \quad (1)$$

where, A_j and A_{org} are the total numbers of columns possessing damage class 0 through V and the total number of the columns, respectively. η_j is the seismic capacity reduction factor from Table 2.

Table 2 Damage class definition of RC columns

Damage class	Brittle column ($h_0/D \leq 3$)	Ductile column ($h_0/D > 3$)
I	0.95	0.95
II	0.60	0.75
III	0.30	0.50
IV	0.00	0.10
V	0.00	0.00

As a result, the damage classes of columns for

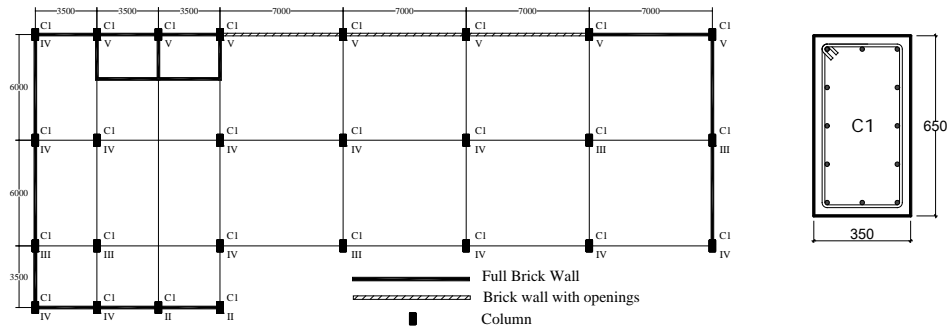


Fig.3 Floor plan first story and damage class of columns of Fire Station building

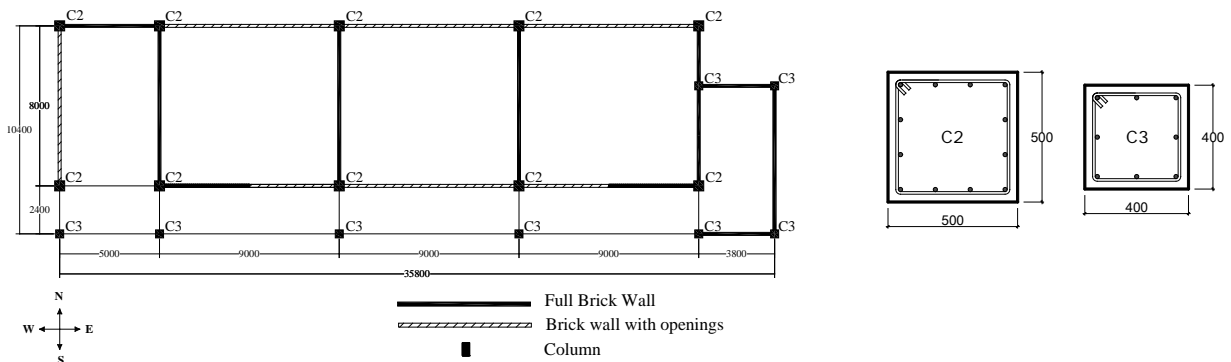


Fig.4 Floor plan first story of Forestry Faculty building of Tadulako University

where, h_0 is a clear height of the column, and D is column depth. Based on R -value, that is slight damage with $95\% \leq R$, light damage with $80\% \leq R < 95\%$, moderate damage with $60\% \leq R < 80\%$, heavy damage with $R < 60\%$, and collapse with $R \approx 0$, the first story of Fire Station building was classified as severe damage. On the other hand, the first story of the Forestry Faculty building totally collapsed.

4.2 Seismic Performance of Investigated Buildings

Seismic performances of the Fire Station building and Forestry Faculty building were evaluated based on the Japanese standard [12]. The seismic performances of both buildings were analyzed in each direction and only for the first story, where the most severe damage was found out to both buildings. The rebars arrangement of the columns of each building is shown in Fig.3 and Fig.4, respectively. For the case of Forestry Faculty building, the cross-sectional dimension of column C2 was 500x500 mm with 12D22 longitudinal reinforcements, and column C3 was 400x400 mm with 8D22, as shown in Fig.4. The stirrups of C2 and C3 were $\phi 10$ -100. The infill walls were neglected in the seismic calculation assuming as non-structure. However, the partial walls were taken into account to calculate the apparent heights of the columns.

Based on the Standard for seismic evaluation of existing building [12], the calculated seismic performance of the building was presented in the relationship between the cumulative strength index, C and ductility index, F . The C index was calculated by Eq. (2) [12]-[14]

$$C = C_i + \sum_j \alpha_j C_j \quad (2)$$

$$C_i = \frac{Q_{ui}}{\sum W} \quad (3)$$

where, C_i : the strength index of the i -th group of columns having the same ductility index, calculated using Eq. (3), C_j : the strength index of the j -th group having the same ductility index larger than that of i -th group, α_j : the effective strength factor of the j -th group for considering differences between yield deformations of i -th and j -th groups as described in [13]. Q_{ui} is the ultimate lateral load-carrying capacity of the i -th group of columns, which is evaluated as the smaller value between the shear force at flexural yielding, Q_{mu} , and the ultimate shear strength, Q_{su} , which are calculated by Eqs. (4) and (6), respectively [12,13]. $\sum W$ is the total weight of the building.

$$Q_{mu} = \frac{2M_u}{h_0} \quad (4)$$

$$M_u = 0.8 a_t \cdot D + 0.5 N \cdot D \left(1 - \frac{N}{b \cdot D \cdot f_c} \right) \quad (5)$$

$$Q_{su} = \left[\frac{0.053 p_t^{0.23} (18 + f_c)}{\frac{M}{(Q \cdot d)} + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy}} + 0.1 \sigma_0 \right] b \cdot j \quad (6)$$

where, p_t is tensile reinforcement ratio calculated ($p_t = a_t / (b \cdot D)$), M/Q is shear span length in which the default value is $h_0/2$, d is effective depth of column, p_w is shear reinforcement ratio calculated by ($p_w = a_w / (b \cdot s)$), σ_{wy} is yield stress of shear reinforcement, σ_0 is axial stress in column ($\sigma_0 = N / (b \cdot D)$), j is the distance between tension and compression forces, default value is $0.8D$, a_w is the cross-sectional area of shear reinforcing bars, s is spacing of hoops. If the value of $M/(Q \cdot d)$ is less than unity or greater than 3, the value of $M/(Q \cdot d)$ to be unity or 3, respectively, and the value of σ_0 is not more than 8 N/mm^2 .

The F index, which represents the deformability of the column, was calculated by Eq. (7) for the shear column [12]. The F index for the flexural column was determined by Eq. (7) in the case $R_{mu} < R_y$ and by Eq. (8) in the case $R_{mu} \geq R_y$ [12]. This index, excepting short ones, ranges between 1.0 and 3.2, which corresponds to a lateral drift ratio of 1/250 and 1/30, respectively.

$$F = 1.0 + 0.27 \frac{R_{su} - R_{250}}{R_y - R_{250}} \quad (7)$$

$$F = \frac{\sqrt{2R_{mu}/R_y - 1}}{0.75 \left(1 + \frac{0.05R_{mu}}{R_y} \right)} \leq 3.2 \quad (8)$$

where, R_{mu} is the drift angle at the ultimate flexural strength of column = $(h_0/H_0) \cdot cR_{mu} \geq R_{250}$, $cR_{mu} = cR_{my} + cR_{mp} \leq cR_{30}$, $cR_{mp} = 10(Q_{su}/Q_{mu} - q) \cdot cR_{my} \geq 0$, $q = 1.0$ for $S \leq 100 \text{ mm}$, $q = 1.1$ for $S > 100 \text{ mm}$. s : spacing of hoops, h_0 = clear height of the column, and H_0 = height of column from bottom to top of the lower floor slab. cR_{150} = standard drift angle of column (measured in the clear height of column), 1/150. cR_{250} = standard drift angle of column (measured in the clear height of column), 1/250, R_{250} = standard inter-story drift angle, 1/250.

Consequently, a distinct difference in seismic performance was obtained in both investigated buildings, as shown in Fig.5. The maximum strength of the Fire Station building was 0.39 of strength index C at 1.27 of ductility index F in South-North direction, as shown in Fig.5(a). The figure shows that the strength of the Fire Station building was maintained in the plastic region up to 2.56 ductility index.

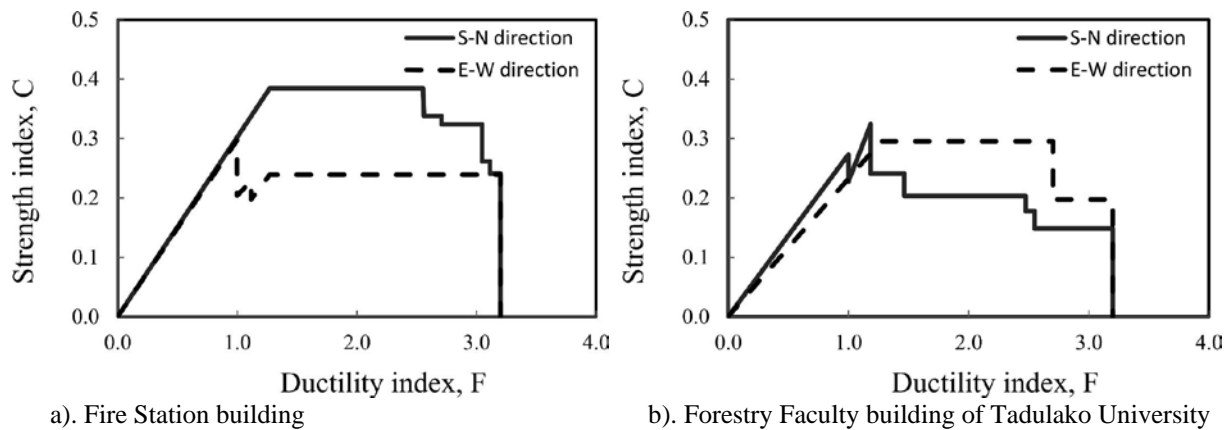


Fig.5 Seismic performance of the investigated building

In the case of the Forestry Faculty building, the maximum strength was identified with a strength index of 0.32 at 1.20 of the ductility index in the South-North direction, as presented in Fig.5(b). The strength capacity of the building gradually dropped to 0.15 in the plastic region. Although its strength index of 0.29 was retained up to 2.70 of the ductility index in E-W direction, this value was relatively low strength capacity for RC building in a high seismic area. According to the calculated results, it is concluded that the RC building can be survived during the earthquake when it has a strength index of more than 0.3 with large plastic deformation.

5. CONCLUSION

According to a post-earthquake investigation conducted after the September 2018 Palu earthquake, several typical damages on building structures and house were observed such as the total collapse of RC buildings, the failure of beam-column joint, the flexural failure of columns, the shear failure of short columns, and the failure of masonry infills. A further analysis was conducted on two damaged buildings, one a survive damage building, and the other collapsed on its first story. As a result, the survived building was able to maintain its lateral strength with large deformation. Therefore, it is concluded that the RC building can survive during the earthquake if such a building has high strength capacity in large plastic deformation.

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