Liquefaction investigation of Balaroa, Central Sulawesi on liquefied and non-liquefied areas

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Abstract. On September 28, 2018, an earthquake occurred in Central Sulawesi with Mw 7.5. This earthquake caused severe damage in Balaroa due to liquefaction that resulted in flow-slide that destroyed 1357 houses. Some fears arose related to the recurrence of the disaster. A study was conducted in the Balaroa liquefied area and the non-liquefied area nearby to examine the ground condition of the site that may not be appropriate for residential purposes. The investigation adopted Liquefaction-Triggering-Based Standard Penetration Test (SPT) Procedure by Idriss and Boulanger, and then was analysed further using Liquefaction Potential Index (LPI) by Iwasaki et al. Data for analysis were obtained from soil investigation at the liquefied area, i.e., at beginning and middle of the flow-slide area, and at outside of the flow-slide end which represented a non-liquefaction area. The results showed that in the liquefied area, the LPI values of 24 and 25 were categorized as very high liquefaction potential, whereas, at the non-liquefied area nearby, the LPI was found about 4 that was in the range of low liquefaction potential. High liquefaction potential conditions occur when thick layer(s) of liquefied soil is found at the upper part of the ground with 4 m or more thickness.

1. Introduction

An earthquake hit Central Sulawesi, Indonesia on September 28, 2018, with a moment magnitude of Mw 7.5. The earthquake was caused by the strike-slip movement of Palu-Koro Fault [1,2]. A super shear rupture occurred with an average rupture rate of 4.1 km/s [3]. The earthquake’s epicenter was at 0.1781°S and 119.8401°E, where the hypocentre was at 10 km depth [4]. This earthquake triggered tsunami and massive liquefaction in Palu City, Donggala Regency, and Sigi Regency. Four locations suffered severe liquefaction, i.e., Balaroa, Petobo, Jono Oge, and Sibalaya [5], in which flow-slides affected large areas. The locations of the flow-slide due to the earthquake are shown in figure 1.

Balaroa is an area located in Palu City with a relatively dense population. This area is the flow-slide area in western Palu Valley. The flow-slide covered about 40 ha with flow distance up to 300 m [6]. The liquefaction in Balaroa had destroyed 1357 buildings [7], most of them were resident's houses. The flow-slide also resulted in 599 death tolls and 136 people missing in Balaroa Village [8], who were believed sinking with their houses into the ground.
The geology of Balaroa consists of alluvium and marine deposits containing relatively loose gravel, sand, mud, and coral limestone in the rivers, deltas, and shallow-sea bed, which took place at the Holocene [9]. Holocene deposited soils are more susceptible to liquefaction than Pleistocene soils [10]. The groundwater table in this area is close to the ground surface that significantly affects the liquefaction susceptibility. The liquefaction susceptibility decreases with the increase in the depth of the groundwater table, where the effects of liquefaction are most commonly observed at sites where groundwater is within a few meters of the ground surface [10]. Balaroa is a high seismicity area due to the presence of the Palu-Koro Fault extending through Palu City. The Palu-Koro Fault is the active zone with a slip rate of 33 mm/yr [11]. These conditions make the Balaroa area having a high liquefaction potential.

Considering the danger, such as the previous liquefaction tragedy, the local government worries about recurrence of a similar disaster in this region. Studies are conducted continuously to obtain various inputs for evaluating the existing buildings and for infrastructure development planning.

In this study, the investigation was conducted to evaluate the liquefaction potential involving liquefied and non-liquefied areas. Analysis was performed using the Liquefaction-Triggering-Based Standard Penetration Test (SPT) Procedure proposed by Idriss and Boulanger [12]. This analysis gave safety factor values that indicated liquefaction potential in each soil layer. The results were further analysed into the Liquefaction Potential Index (LPI) developed by Iwasaki et al. [13]. The LPI showed the level of liquefaction potential at each borehole.

2. Research Method

2.1. Data Collection

Data used in this investigation were taken from the field investigation (boring survey and SPT) and laboratory test results in the report of boring surveys for the basic response for the Central Sulawesi Earthquake and project for the development of regional disaster risk resilience plan in Central Sulawesi conducted by Japan International Cooperation Agency (JICA) in 2019 after the liquefaction disaster. The analysis employed 3 borehole data in the Balaroa area, i.e., B-1, B-2, and B-3 (figure 2). The B-1 was located at the beginning of the flow-slide area, the B-2 was located at the middle of the flow-slide area, and the B-3 was located at the outside of the flow-slide end. The beginning point and the point at the middle of the flow-slide area represent liquefied cases where severe damage occurred. Meanwhile, the point nearby the flow-slide end represents the non-liquefied area.
In the field investigation, the SPTs were conducted at 1.0 m intervals down to the depth of 30 m below the ground surface. The groundwater level (GWL) was recorded during boring work. The N-SPT values greater than 60 were analysed by assuming a value of 60. Soil samples taken during the boring were tested in the laboratory to obtain soil parameters, especially the percentage of fines content (FC) and unit weight.

2.2. Liquefaction Potential Analysis
Analysis of the liquefaction potential in each soil layer was carried out by calculating the factor of safety (FS). The FS value is the ratio of cyclic stress ratio (CSR) and cyclic resistance ratio (CRR). CSR is defined as the shear stress induced by an earthquake, and CRR is defined as the resistance of the soil to liquefaction.

The CSR value was calculated by using equation (1) proposed by Seed and Idriss [14].

$$CSR = 0.65 \left( \frac{\sigma_{ve}}{\sigma_{ve}} \right) \left( \frac{a_{max}}{g} \right) r_d$$  \hspace{1cm} (1)

Where $\sigma_{ve}$ is the total vertical stress, $\sigma_{ve}$ is the effective vertical stress, $a_{max}$ is the maximum ground surface acceleration, and $r_d$ is the stress reduction coefficient. The value of $a_{max}$ used in this study was peak ground acceleration (PGA) on the firm rock at Balaroa based on the report of the U.S. Geological Survey (USGS), which was 3.2 m/s$^2$ (0.33 g) [4]. The $r_d$ was obtained by using equations (2) to (4) developed by Idriss and Boulanger [12].

$$r_d = \exp[\alpha(z) + \beta(z)M]$$  \hspace{1cm} (2)

$$\alpha(z) = -1.012 - 1.126 \sin \left( \frac{z}{11.73} + 5.133 \right)$$  \hspace{1cm} (3)

$$\beta(z) = 0.106 + 0.118 \sin \left( \frac{z}{11.28} + 5.142 \right)$$  \hspace{1cm} (4)

Where $z$ is the depth below ground surface in meters and $M$ is the earthquake moment magnitude. The value of $M$ used in this study was 7.5, which caused the flow-slide in Balaroa. The angle unit in equations (3) and (4) was in radians.

The CRR value was determined based on the SPT results after applying some corrections. Corrections were applied related to the test procedure and overburden pressure. The corrected N-SPT
value against the test procedure in the field and overburden pressure \((N_1)_{60}\) was calculated by equation (5). The maximum value of \((N_1)_{60}\) is 46.

\[
(N_1)_{60} = C_N N_{60}
\]

(5)

Where, \(C_N\) is the overburden pressure factor and \(N_{60}\) is the N-SPT value corrected with test procedure. Idriss and Boulanger [12] formulated \(C_N\) and \(N_{60}\) in equation (6) and equation (7).

\[
C_N = \left(\frac{P_a}{\sigma_{vc}}\right)^{0.784-0.0768\sqrt{\frac{(N_1)_{60}}{10}}} \leq 1.7
\]

(6)

\[
N_{60} = C_E C_P C_R C_S N_m
\]

(7)

Where, \(P_a\) is the atmospheric pressure which is equal to 101 kPa, \(C_E\) is SPT correction factor for energy ratio, \(C_P\) is the correction factor for borehole diameter, \(C_R\) is the correction factor for rod length, \(C_S\) is the correction for samplers, and the \(N_m\) is the field SPT value.

CRR correlation to the \((N_1)_{60}\) depended on the fines content (FC) of the soil. This correlation was expressed in the value of \((N_1)_{60cs}\) which was obtained using equations (8) and (9), developed by Idriss and Boulanger [12].

\[
(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}
\]

(8)

\[
\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right)
\]

(9)

Based on the corrected N-SPT values, the CRR was calculated according to equation (10), proposed by Idriss and Boulanger [12] for earthquake magnitude, \(M = 7.5\), and effective vertical stress of \(\sigma_{vc} = 1\) atm.

\[
CRR_{M=7.5, \sigma_{vc}=1} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \frac{(N_1)_{60cs}^2}{126} - \frac{(N_1)_{60cs}^3}{23.6} + \frac{(N_1)_{60cs}^4}{25.4} - 2.8\right)
\]

(10)

The equation (10) applied only to sandy and saturated soil layers. Silt and clay layers were considered not to liquefy. The soil was classified as silt or clay when the FC ≥ 50% according to the soil classification by Unified Soil Classification System (USCS).

For earthquakes with other magnitude and effective overburden stress values, the CRR value was calculated using equations (11) to (14) by Idriss and Boulanger [12].

\[
CRR_{M, \sigma_{vc}} = CRR_{M=7.5, \sigma_{vc}=1} \cdot MSF \cdot K_{\sigma}
\]

(11)

\[
MSF = 6.9 \exp\left(-\frac{M}{4}\right) - 0.058 \leq 1.8
\]

(12)

\[
K_{\sigma} = 1 - C_{\sigma} \ln\left(\frac{\sigma_{vc}}{P_a}\right) \leq 1.1
\]

(13)

\[
C_{\sigma} = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} \leq 0.3
\]

(14)

The FS value was then calculated using the CSR and the CRR value by employing equation (15).

\[
FS = \frac{CRR_{M, \sigma_{vc}}}{CSR}
\]

(15)

Liquefaction has a potential to occur when FS < 1, while there is no potential for liquefaction to occur when FS ≥ 1.
2.3. Level of Liquefaction Potential Evaluation

The level of the liquefaction potential at a site was determined based on the LPI. The LPI was determined based on the FS values obtained at a depth from $z = 0$ to $z = 20$ m below the ground surface. A study from Mase et al. [15] showed that using LPI, the sites with high soil strength resistance have a relative risk to liquefy, while sites with low soil strength resistance have more risk to liquefy. The LPI was calculated using equations (16) developed by Iwasaki et al [13].

$$LPI = \int_{0}^{20} F(z).w(z).dz$$  \hspace{1cm} (16)

Where $F(z) = 1 - FS$ for $FS < 1$ and $F(z) = 0$ for $FS \geq 1$, and $w(z) = 10 - 0.5z$ with $z$ in meter. Based on the LPI value, the liquefaction potential was classified into the several levels defined by Iwasaki et al [13] as shown in Table 1.

**Table 1. Classification of liquefaction potential based on LPI.** [13]

| Liquefaction Potential Index (LPI) | Level of Liquefaction Potential |
|-----------------------------------|---------------------------------|
| LPI = 0                           | Very Low                       |
| $0 < LPI \leq 5$                  | Low                            |
| $5 < LPI \leq 15$                 | Rather High                    |
| LPI > 15                          | Very High                      |

3. Result and Discussion

3.1. Liquefaction Potential Analysis

Figures 3 to 5 show the N-SPT values and soil fine contents (FC) with depth, and the factor of safety (FS) calculation results in boring points B-1, B-2, and B-3, respectively.

**Figure 3.** Analysis of liquefaction potential in B-1.
Figure 3 shows that the boring point B-1 has a GWL of 1.95 m. The N-SPT values varied. Low-density sand with N-SPT values of 3 - 12 was found at a depth of 0.5 m – 6.5 m. A thin layer of very dense sand and gravel with an N-SPT value of 60 was recorded at a depth of 7.5 m – 8.5 m. At further depths, layers of sand with low density were still found. The N-SPT values in the range of 12 – 33 were found at a depth of 9.5 m – 15.5 m, the N-SPT values in the range of 42 – 51 were found at a depth of 16.5 m – 18.5 m, the N-SPT values in the range of 23 – 26 were found at a depth of 19.5 m – 20.5 m and the values of the N-SPT in the range of 34 – 60 were found at a depth of 21.5 m – 29.5 m. The soil's fine contents (FC) were considered moderate, with a maximum value of about 36.1% in the boring point B-1. These soils were not classified as silt and clay. Therefore, they were considered to have the liquefaction potential, so that the FS calculation for all soil layers was performed. The results showed that at depths of 2.5 m – 6.5 m, 9.5 m – 12.5 m, 14.5 m – 15.5 m, 19.5 m – 20.5 m, and 23.5 m – 26.5 m, the FSs < 1, which means that the liquefaction potential may occur in those layers.

Figure 4 shows that the boring point B-2 has a GWL of 0.15 m. The N-SPT values were found at a depth of 1.5 m – 5.5 m. At further depths, the N-SPT values mostly were 60, where the N-SPT values less than 40 were only found at depths of 15.5 m – 17.5 m and 20.5 m – 21.5 m. Regarding the fine contents, the soil layers in the B-2, which had FC of 50% or more, were only found at depths of 20.5 m and 24.5 m. These soil layers are categorized as silt and clay, so it was considered to have no liquefaction potential. The values of FS < 1 were found at depths of 1.5 – 5.5 m, 15.5 m, and 21.5 m, indicating the presence of the liquefaction potential.
Figure 5 shows that the boring point B-3 has a GWL of 0.55 m. Low N-SPT values were found at a depth of 0.5 – 9.5 m in the range of 3 – 18. At the next depth, the N-SPT values were found that varied between 21 – 50. Referring to the FC, the B-3 was dominated by the soil layer with the FC percentage of 50% or more. These conditions indicated that most of the soil layers in the area consist of silt and clay, which is considered to have no liquefaction potential. The results of the FS showed that no liquefaction potential was predicted in almost all the layers. The liquefaction potentials were only found at a depth of 2.5 m and 18.5 m with thin layers.

Comparing the B-1 and the B-2 results, the B-2 has fewer liquefied layers than the B-1 but occurred massive damage at the B-2 caused by the 2018 Central Sulawesi Earthquake. It showed that severe flow-slide damage had occurred in areas with a thicker liquefied layer near the ground surface which directly supported the buildings on the ground surface.

3.2. Level of Liquefaction Potential Evaluation
The FS values obtained from the previous analysis were then used to evaluate the level of liquefaction potential by using the LPI. The results of the LPI evaluation are shown in table 2.

| No. | Hole | Coordinate | LPI | Level of Potential Liquefaction |
|-----|------|------------|-----|---------------------------------|
| 1   | B-1  | 0.9063°S 119.8402°E | 23.763 | Very High                       |
| 2   | B-2  | 0.9059°S 119.8422°E | 24.863 | Very High                       |
| 3   | B-3  | 0.9043°S 119.8472°E | 4.340  | Low                             |

Table 2 shows that at the B-1 and the B-2, the liquefaction potential levels were found to be very high, and at the B-3, the level of liquefaction potential was found to be low. The B-1, which was at the beginning of the flow-slide area, had an LPI value of 23.763. The B-2, which was at the middle of the flow-slide area, had an LPI of 24.863. Meanwhile, the B-3, which was at the outside of the end of the flow-slide area, had an LPI of 4.340. The results of the evaluation of this study had met the field conditions as shown in figure 2. The very high level of liquefaction potential areas suffered from massive liquefaction resulted in severe damage due to flow-slide. On the other hand, different conditions were found in the area with the low level of liquefaction potential where no severe damage was experienced.

The results in table 2 show that the LPI values at the boring points B-1 and B-2 are very similar. However, the condition of the liquefaction is very different. At the boring point B-1, the liquefaction occurs at almost all of the soils layers down to 20 m deep. In contrast, at the boring point B-2, the liquefaction occurs mainly at the upper layer close to the ground surface. This condition indicates that the contribution of the upper soil layer to liquefy plays a dominant role in the occurrence of severe liquefaction. The boring points B-1 and B-2 could be considered two models of soil layer that cause serious liquefaction occurrence. Other models need to be investigated further to obtain various soil layer conditions which give the LPI value greater than 15. Besides the models, soil stratigraphy in this region needs to be collected. Comparing the collected soil data in this region and the models developed for evaluating the LPI, it is possible to produce a zonation map of the liquefaction risk in this region. From this information, the local government will be able to make regulations on the land use and the building criteria related to the liquefaction risk.

4. Conclusion
The LPI analysis is still appropriate for evaluating the liquefaction potential in this area. It can be seen from the consistency of the results and the fact found on the site. This study produces two models of soil conditions that caused severe liquefaction. Severe liquefaction is most likely to occur in the site where a thick layer of soil susceptible to liquefy is found down to the depth of 20 m. A very high liquefaction potential level also occurs when the soil susceptible to liquefy is found at most upper layers down to a depth of about 4 m only. These two results could be considered as the extreme
condition where more information is necessary to be developed. Using soil data collected from this region for the analysis is possible to make a zonation map of the liquefaction risk of this region.

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References
[1] Mason H B, Gallant A P, Hutabarat D, Montgomery J, Reed A N, Wartman J, Irsyam M, Prakoso W, Djarwadi D, Harnanto D, Alatas I, Rahardjo P, Simatupang P, Kawanda A and Hanifa R 2019 Geotechnical Reconnaissance: The 28 September 2018 M7.5 Palu-Donggala, Indonesia Earthquake
[2] Socquet A, Hollingsworth J, Pathier E and Bouchon M 2019 Evidence of supershear during the 2018 magnitude 7.5 Palu earthquake from space geodesy Nat. Geosci. 12 192–9
[3] Fang J, Xu C, Wen Y, Wang S, Xu G, Zhao Y and Yi L 2019 The 2018 Mw 7.5 Palu earthquake: A supershear rupture event constrained by InSAR and broadband regional seismograms Remote Sens. 11 1–15
[4] U.S. Geological Survey National Earthquake Information Center 2018 M7.5 Palu, Indonesia Earthquake of September 28, 2018
[5] PuGeN (National Research Center for Earthquake Studies) 2018 Study of Palu Central Sulawesi Province Earthquake September 28, 2018 (M7.4) ed M Irsyam, N R Hanifa, D Djarwadi and D A Sarsito (Bandung: Research and Development Agency of Ministry of Public Works and Housing)
[6] Kiyota T, Furuichi H, Hidayat R F, Tada N and Nawir H 2020 Overview of long-distance flow-slide caused by the 2018 Sulawesi earthquake, Indonesia Soils Found. 60 722–35
[7] Central Sulawesi Governor 2019 Central Sulawesi Governor Regulation Number 10 of 2019 Post-Disaster Rehabilitation and Reconstruction Plans (Indonesia)
[8] BAPPEDA (Development Planning Agency at Sub-National Level) Palu City 2019 Data on Victims of The Earthquake, Tsunami and Liquefaction in Palu City in 2018 Phase I, Balaroa Village BAPPEDA (Development Plan. Agency Sub-National Level) Palu City
[9] Sukamto R 1973 Reconnaissance Geological Map of The Palu Quadrangle, Sulawesi (Bandung: Pusat Penelitian dan Pengembangan Geologi)
[10] Kramer S L 1996 Geotechnical Earthquake Engineering (Upper Saddle River, New Jersey: Prentice Hall)
[11] PuGeN (National Research Center for Earthquake Studies) 2017 Map of Sources and Hazards of Indonesian Earthquake 2017 ed M Irsyam, S Widiyantoro, D H Natawidjaja, I Meilano, A Rudyanto, S Hidayati, W Triyoso, N R Hanifa, D Djarwadi, F Lutfi and Sunarjito (Bandung: Research and Development Agency of Ministry of Public Works and Housing)
[12] Idriss I M and Boulanger R W 2008 Soil Liquefaction during Earthquakes Earthquake Engineering Research Institute MNO-12 (United States of America: EERI Publication)
[13] Iwasaki T, Tokida K and Tatsuoka F 1981 Soil Liquefaction Potential Evaluation with Use of the Simplified Procedure First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics (St. Louis, Missouri) pp 209–14
[14] Seed H B and Idriss I M 1970 A Simplified Procedure for Evaluating Soil Liquefaction Potential (Berkeley, California)
[15] Mase L Z, Farid M, Sugianto N and Agustina S 2020 The Implementation of Ground Response Analysis to Quantify Liquefaction Potential Index (LPI) in Bengkulu City, Indonesia J. Civ. Eng. Forum 6 319