

Structural evaluation of 3-story dormitory reinforced concrete building considering soil liquefaction potential

Fauzan¹, Nadia Milla Hanifah¹, Willy Peratundhika E¹, Mutia Putri Monika¹, and Zev Al Jauhari²

¹Civil Engineering Department, Andalas University, 25163 Limau Manis, Padang, Indonesia

²Civil Engineering Department, Bengkalis State Polytechnic, 28711 Sungai Alam, Bengkalis, Indonesia

Abstract. The liquefaction phenomenon is the increase in water pressure in the soil, which will reduce the soil strength in supporting the load and loss of binding power between its grains. Soil liquefaction usually occurs when there is a seismic movement in the soil layer due to seismic (earthquakes) loads. Therefore, the building constructed in the soil liquefaction prone area should be designed according to the standard code. However, many design consultants do not pay attention to this condition and the building still was designed as usual even the building is located on soil liquefaction prone area. In 2018, a 3-story dormitory building structure of Hamka's boarding school was constructed on soil liquefaction prone area in Padang city. After reviewing the design document, it was found that the consultant did not consider the soil liquefaction in its structural analysis. In this study, an evaluation of the building structure was carried out to investigate the capacity of the building in resisting the loads. From the soil evaluation using the soil Cone Penetration Test (CPT) result, it was found that the location of the dormitory building has a liquefaction potential at a depth of 1.2 - 8 meters. Considering the soil liquefaction potential in the building, the structural analysis results show that the capacity of the dormitory building, especially column, beam and foundation were not strong enough to resist the combination loads acting on the structures. Therefore, the building structure should be strengthened to face the further big earthquake that will cause the soil liquefaction.

1 Introduction

West Sumatra, especially Padang city, has great potential for earthquakes that can cause liquefaction. Liquefaction is a phase of solid change into a liquid phase caused by an increase in water pressure in the soil cavity [1]. The impact of increasing pore water pressure causes the reduction of soil shear strength significantly due to a decrease in pore water effective stress [1].

From previous liquefaction studies, it is known that co-seismic liquefaction and the distribution of damage caused by liquefaction generally only occur in areas formed by layers of granular sediment saturated with low density, and the possibility of surface co-seismic movements exceeding the value certain threshold [2, 3]. Liquefactions in the soil layer are affected by the nature of soil engineering, geological environmental conditions and earthquake characteristics. Several factors that must be considered include grain size, groundwater level and maximum ground vibration acceleration [2].

Liquidation itself is a threat to construction damage in the city of Padang, which can be caused by the speed and acceleration of the earthquake and the displacement of the land surface. The potential of liquefaction is mainly in the layer of sand that is saturated with water in the presence of dynamic cyclic forces [4]. It has been known that many big earthquakes occurred in Padang City, both tectonic and volcanic earthquakes. If this

intensity continues to increase, it can be confirmed that land subsidence due to liquefaction in Padang city will get worse. As a result, most of the building construction in Padang city will get serious damage due to the soil liquefaction.

In order to prevent the building damages due to the earthquake on soil liquefaction potential area, a structural evaluation should be carried out using the national standard code. In this study, the structural evaluation of a 3-story dormitory building of Hamka's boarding school that was constructed on soil liquefaction area in Padang City, Indonesia, was carried out using the current standard code.

2 Evaluation of existing building

2.1 Structural modeling

The dormitory building of Hamka's boarding school consists of three floors with a total building height of 12.5 m. The building length and width are 37.5 m and 13.25 m, respectively. This building was designed using reinforced concrete structures. The concrete compressive strength, f_c' and steel yield strength, f_y were 22.85 MPa and 400 MPa, respectively. Structural modeling and analysis were carried out using ETABS 9.7.1 software.

* Corresponding author: fauzan@eng.unand.ac.id

Fig. 1 shows the 3D structure modeling. The dimensions of the structural components are:

1. Beam: main beam (40/25); secondary beam (30/20)
2. Column: K1 (400 x 500 mm²), K2 (250 x 400 mm²), and K3 (250 x 250 mm²)
3. Slab thickness: roof = 150 mm; floor = 150 mm

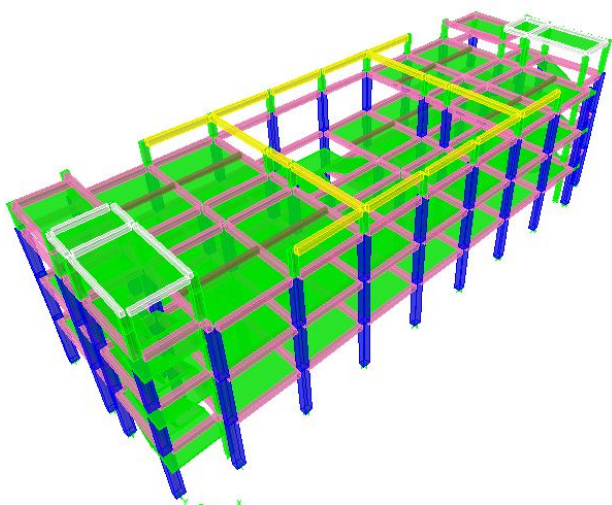


Fig. 1. 3D structural modeling of the Hamka Dormitory Building.

2.2 Loads

The dead and live loads on the building structure based on the Indonesian Building Regulation (PPIUG, 1983) are shown in Tables 1 and 2.

Table 1. Dead loads on the building structure [5].

Load	Load Value
Concrete density	2400 kg/m ³
Mortar (per-cm thick)	21 kg/m ²
Plafond	20 kg/m ²
Brick wall (1/2 brick)	250 kg/m ²
Ceramics	24 kg/m ²
Waterproofing	14 kg/m ²
Mechanical, electrical, plumbing	25 kg/m ²

Table 2. Live loads on the building structure [5].

Load	Load Value
Bedroom	240 kg/m ²
Toilet	200 kg/m ²
Corridor	383 kg/m ²
Lobby	479 kg/m ²

The earthquake loads was calculated using SNI 1726-2012 (Procedures for Planning Earthquake Resilience for Building Structure and Non-Building) [6].

2.2.1 Dynamic earthquake load (spectrum response)

The spectrum response is used as an analysis of dynamic earthquake loads. The spectrum response earthquake load is calculated based on SNI 1726:2012 using the 2017 Indonesian Earthquake Hazard Map. Fig. 2 shows the spectrum response for Padang City with medium and liquefied soil conditions.

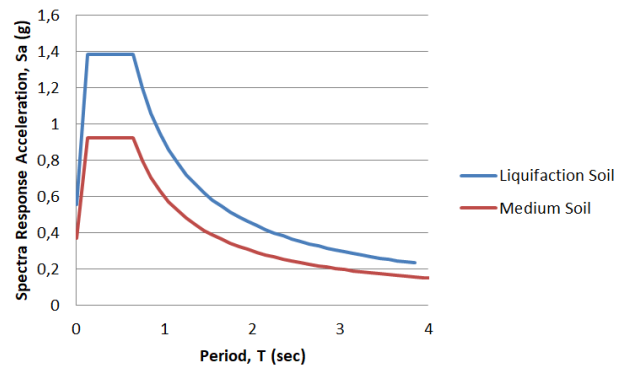


Fig. 2. The comparison of the spectrum response with medium soil and soil liquefaction prone area in Padang City.

The spectrum response data are inputted into the structural modeling, then scale factor calculations are performed on ETABS using equation (1):

$$SF = G.I / R \quad (1)$$

2.2.2 Combination of loads

Based on Article 5.8.2 Indonesian National Standard (SNI) for the Earthquake Load 2012, to simulate the direction effect of the random earthquake plan to the structure of the building, the effect of earthquake loading in the main direction determined according to Article 5.8.1 must be considered to be 100% effective and must be considered to occur together with the effect of deep earthquake loads with a perpendicular direction to the main direction of the load, with an effectiveness of only 30%. The load combination for this analysis is as follows:

1. 1,4 DL
2. 1,2 DL + 1,6 LL + 0,5 R
3. 1,2 DL + 1,0 LL ± 1,0 EQx ± 0,3 EQy
4. 1,2 DL + 1,0 LL ± 0,3 EQx ± 1,0 EQy
5. 0,9 DL ± 1,0 EQx ± 0,3 EQy
6. 0,9 DL ± 0,3 EQx ± 1,0 EQy

2.3 Evaluation of soil liquefaction potential

The behaviour of liquefaction on soil is affected by two main parameters, namely corrected resistance (q_{c1}) and cyclic stress ratio (CSR) [7]. The steps to estimate the depth of the soil which has liquefaction potential are:

- Determine the number of layers and the layer numbering

The number and the layer numbering are determined based on a certain depth range, which aims to simplify the analysis and calculation. The study, calculations were carried out for each layer with a data range of 20 cm of depth.

- Estimating the weight of soil volume

Weight estimate of soil volume is carried out using soil behaviour graphs based on the static cone penetration data, as shown in Fig. 3, then the results of the graph are correlated to Table 3 to obtain the estimated weight of the soil volume based on the zone obtained.

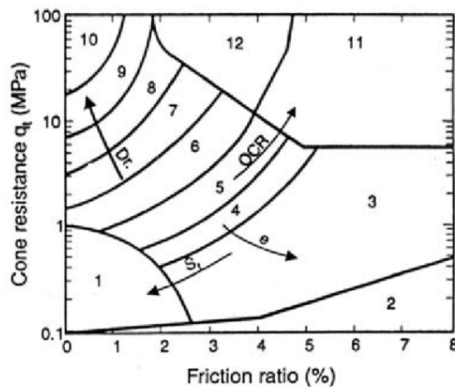


Fig. 3. Soil type classification chart [8].

Table 3. The estimation of unit weight [8].

The estimation of unit weight based soil description	
Zone	Approximate of unit weight (kg/cm ³)
1	0.00175
2	0.00125
3	0.00175
4	0.00180
5	0.00180
6	0.00180
7	0.00185
8	0.00190
9	0.00195
10	0.00200
11	0.00250
12	0.00190

- Determine the overburden ground stress
 Vertical stress on soil was calculated using the following formula:

$$\sigma_o = h \times \gamma \quad (2)$$

where:

- σ_o is vertical stress on soil (kg/m²)
- h is depth (m)
- γ is weigh of soil volume (kg/m³)

- Determine the effective stress for the soil
 The vertical effective soil stress was calculated using the following formula:

$$\sigma_o' = \sigma_o - u = (h \times \gamma) - (h_w \times \gamma_w) \quad (3)$$

where:

- σ_o' is the effective soil stress (kg/m²)
- σ_o is the total soil stress (kg/m²)
- u is pore water pressure (kg/m²)
- h is depth (m)
- γ is weigh of soil volume (kg/m³)
- h_w is groundwater depth (m)
- γ_w is weight of water volume (kg/m³)

- Determine corrected conus resistance (q_{c1})
 The averaged corrected conus resistance according to type per soil depth, corrected conus resistance is calculated with the following equation:

$$q_{c1} = C_1 \times q_c = CN \times q_c \text{ (kg/cm}^2\text{)} \quad (4)$$

where:

- q_{c1} is corrected conus resistance
- q_c is conus resistance
- CN is correction factor (Fig. 4)

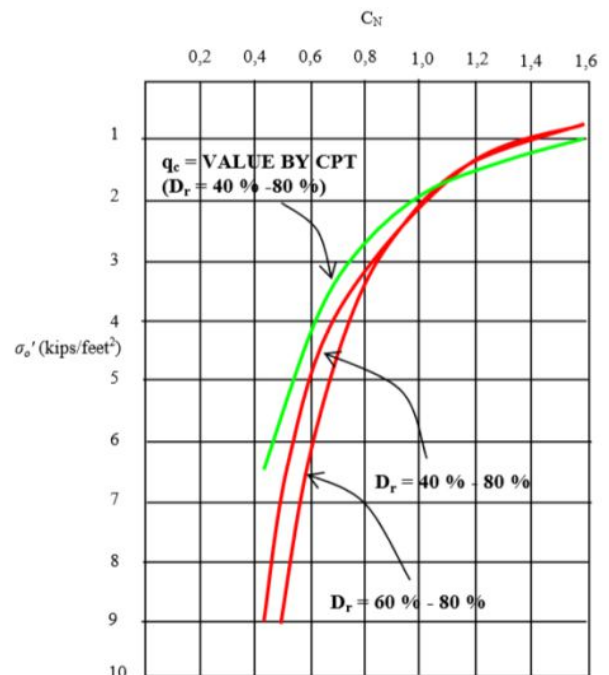


Fig. 4. Ground motion and soil liquefaction during earthquakes [2].

Table 4. The calculation results of soil liquefaction potential for Hamka’s dormitory building.

γ	h	q_c	f_s	f_s / q_c	Soil Type	Total Stress	Eff. Stress	q_c normalization	Reduction of cyclic stress	CSR	CSR average	q_c average	Note
kg/cm ³	cm	kg/cm ²	kg/cm ²			$\sigma_0 = h \cdot \gamma$	σ_0'	CN . q_c	r_d				
0.0014	0	0	0	-	Mounta in Sand Heap	0	0.147	0	1.000	0.000	0.075	1.60	No Liquefaction
0.0014	20	0	0	0.028		0.156	0	0.998	0.033				
0.0014	40	0	0	0.056		0.164	0	0.997	0.062				
0.0014	60	0	0	0.084		0.172	0	0.995	0.088				
0.0018	80	4	0.13	3.250		0.144	0.213	6.4	0.994	0.123			
0.00175	100	2	0.13	6.500		0.175	0.224	3.2	0.992	0.141			
0.00175	120	2	0.13	6.500	Organic Heap	0.21	0.239	3.2	0.991	0.158	0.223	11.5 2	Liquefaction
0.00185	140	6	0.13	2.167		0.259	0.269	9.6	0.989	0.174			
0.00185	160	8	0.13	1.625		0.296	0.286	12.8	0.988	0.186			
0.0018	180	4	0.13	3.250		0.324	0.295	6.4	0.986	0.198			
0.0018	200	4	0.13	3.250		0.36	0.311	6.4	0.985	0.208			
0.00195	220	10	0.13	1.300		0.429	0.360	16	0.983	0.213			
0.00195	240	12	0.13	1.083		0.468	0.380	19.2	0.982	0.220			
0.00195	260	10	0.13	1.300		0.507	0.399	16	0.980	0.227			
0.00185	280	8	0.13	1.625		0.518	0.390	12.8	0.979	0.237			
0.00185	300	8	0.13	1.625		0.555	0.408	12.8	0.977	0.242			
0.00185	320	8	0.13	1.625		0.592	0.425	12.8	0.976	0.247			
0.00185	340	6	0.13	2.167		0.629	0.433	9.6	0.974	0.252			
0.00185	360	6	0.13	2.167		0.666	0.460	9.6	0.972	0.257			
0.00185	380	10	0.13	1.300		0.703	0.477	16	0.971	0.260			
0.00185	400	6	0.13	2.167	0.74	0.495	9.6	0.969	0.264				
0.00195	420	8	0.13	1.625	Clay	0.819	0.554	12.8	0.968	0.261	0.276	58.5 6	Liquefaction
0.00195	440	10	0.13	1.300		0.858	0.574	16	0.966	0.263			
0.00195	460	8	0.13	1.625		0.897	0.593	12.8	0.965	0.266			
0.0019	480	6	0.13	2.167		0.912	0.588	9.6	0.963	0.272			
0.00195	500	8	0.13	1.625		0.975	0.632	12.8	0.962	0.270			
0.00195	520	10	0.13	1.300		1.014	0.651	16	0.960	0.272			
0.00195	540	12	0.13	1.083		1.053	0.670	19.2	0.959	0.274			
0.00195	560	28	0.13	0.464		1.092	0.690	44.8	0.957	0.276			
0.00195	580	30	0.13	0.433		1.131	0.709	48	0.956	0.278			
0,002	600	20	0.13	0.650		1.2	0.759	32	0.954	0.275			
0,002	620	24	0.13	0.542		1.24	0.779	38.4	0.953	0.276			
0,002	640	28	0.13	0.464		1.28	0.799	44.8	0.951	0.277			
0,002	660	55	0.33	0.600		1.32	0.820	88	0.950	0.279			
0,002	680	50	0.33	0.660		1.36	0.840	80	0.948	0.280			
0,002	700	55	0.33	0.600		1.4	0.860	88	0.946	0.281			
0,002	720	60	0.33	0.550		1.44	0.881	96	0.945	0.281			
0,002	740	70	0.33	0.471		1.48	0.901	112	0.943	0.282			
0,002	760	100	0.33	0.330		1.52	0.922	160	0.942	0.283			
0,002	780	80	0.33	0.413		1.56	0.942	128	0.940	0.284			
0,002	800	70	0.33	0.471		1.6	0.962	112	0.939	0.284			
0,002	820	100	0.33	0.330	Rocks	1.64	0.983	160	0.937	0.285	0.286	195. 43	No Liquefaction
0,002	840	110	0.33	0.300		1.68	1.003	176	0.936	0.286			
0,002	860	120	0.33	0.275		1.72	1.023	192	0.934	0.286			
0,002	880	125	0.33	0.264		1.76	1.044	200	0.933	0.286			
0,002	900	110	0.33	0.300		1.8	1.064	176	0.931	0.287			
0,002	920	140	0.33	0.236		1.84	1.085	224	0.928	0.287			
0,002	940	150	0.33	0.220		1.88	1.105	240	0.923	0.286			

• Determine the maximum magnitude and the ground acceleration (a_{max})
 The earthquake magnitude and the maximum ground acceleration are used in the calculation of cyclic stress ratio. This parameter was obtained from Padang Pariaman earthquake data on September 30, 2009, that had magnitude 7.6 SR with a_{max} of 0.28 g.

• Determine the stress reduction factor (r_d)
 The stress reduction factor is calculated based on the Liao-Whitman equation (1986)

$$r_d = 1,0 - 0,00765z \quad (\text{for } z < 9, 15 \text{ m})$$

$$r_d = 1,174 - 0,0267z \quad (\text{for } 9,15 \text{ m} < z < 23 \text{ m})$$

- Calculating the value of cyclic stress ratio (CSR)
 The calculation of CSR is averaged according to the type per depth of soil. The amount of the cyclic stress ratio is determined by:

$$CSR = 0.65 \times (a_{max} \times \sigma_o / (g \times \sigma_o')) \times r_d \quad (5)$$

- Analyze potential liquefaction by plot CSR values (Fig. 5)

From the above calculations, it is obtained the CSR value which is then plotted into the CSR chart to determine the potential for liquefaction. Table 4 shows the results of calculations and plotting results of CSR charts.

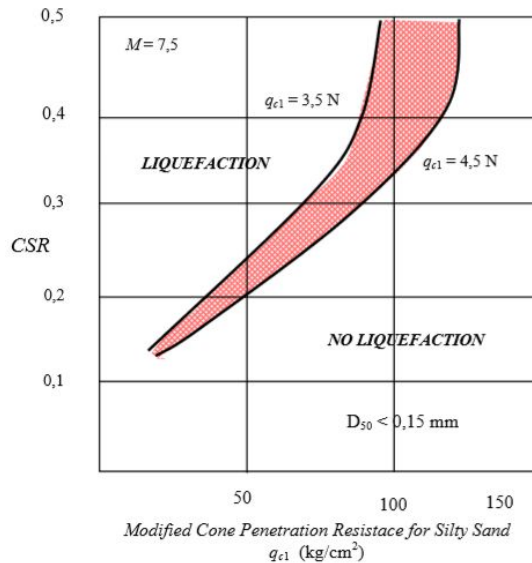


Fig. 5. CSR versus modified cone penetration resistance for silty sand.

From Table 4, it can be concluded that the soil of Hamka’s Dormitory building has liquefaction potential at a depth of 1.2 - 8 meters.

2.4 Foundation capacity

Liquidation could reduce the soil strength in supporting loads because it can make a loss of soil side resistance to axial loads. The Cone Penetration Test (CPT) data were used to analyse the foundation capacity, with a loss of side resistance at a depth that has the potential for liquefaction [9, 10].

2.4.1 Axial bearing capacity of the foundation using the CPT data

The analysis of axial bearing capacity was carried out based on pile group calculations with CPT data along with potential liquefaction.

$$Q_u \text{ pile group} = Q_p \text{ pile group} + Q_s \text{ pile group}$$

$$Q_p \text{ pile group} = E_{qp} \times Q_p$$

$$Q_s \text{ pile group} = E_{qs} \times Q_s$$

- Determine the efficiency of the pile group

$$E_{qp} = (B_q \times L_q) / (m \times n \times A_p) = (0.65 \times 0.65) / (2 \times 2 \times 0.625) = 1.69$$

$$E_{qs} = (2 \times (B_q + L_q)) / (m \times n \times \emptyset) = (2 \times (0.65 + 0.65)) / (2 \times 2 \times 0.25) = 0.845$$

- Determine the capacity of the pile group

a. The end bearing capacity of pile foundation
 $Q_p = A_p \times q_c = 0.625 \times 14709.98 = 919.3734 \text{ kN}$

b. The side resistance of pile foundation
 Due to the potential for liquefaction at a depth of 1.2 - 8 meters, the side resistance at that depth is considered to be 0 (Table 5).
 $Q_s = \emptyset \times JHL$

Table 5. The side resistance of of pile foundation.

Depth	Q _s
0-1	0.010833
1-4	0
4-8	0
8-11	0.0825
11-15	0.0825

Therefore :

$$Q_p \text{ pile group} = E_{qp} \times Q_p (m \times n) = 1.69 \times 919.3734 \times 4 = 6214.96 \text{ kN}$$

$$Q_s \text{ pile group} = \sum E_{qs} \times Q_s (m \times n) = 0.1188 \text{ kN}$$

$$Q_u \text{ pile group} = 919.3734 + 0.196 = 6215.083 \text{ kN}$$

- Determine the allowable bearing capacity (Q_a) of pile group with a safety factor of 5.

$$Q_a = Q_u \text{ pile group} / SF = 6215.16 / 5 = 1243.017 \text{ kN}$$

The ultimate bearing capacity of the foundation based on the analysis of loading is 1000.52 kN.

$$Q_a > Q_u$$

$$1243.017 \text{ kN} > 1000.52 \text{ kN}$$

From the above results, the allowable bearing capacity (Q_a) is greater than the ultimate bearing capacity (Q_u), it indicates that the foundation has enough capacity to resist the working load.

2.4.2 Analysis of lateral capacity on the pile group foundation

Other than axial load and uplift, a pile also experiences lateral loads. Potential sources of lateral loads such as wind loads, lateral soil pressure, water wave loads, ship and vehicle collisions, earthquakes, etc. Foundation deformation due to lateral loads must be within the performance criteria specified for the structure. The calculation of the lateral capacity was conducted using the Broms method with the following procedures:

- Calculation parameters:

$$n_1 : 0.4 \qquad q_u : 13.72 \text{ MPa}$$

$$n_2 : 1.15 \qquad b : 0.25 \text{ m}$$

- Determine the horizontal subgrade reaction coefficient (K_h) at critical depth for cohesive soil and non-cohesive soil.

$$K_h = n_1 \times n_2 \times 80 \times q_u / b = 0.4 \times 1.15 \times 13.72 = 2019.584 \text{ kN/m}^3$$

- Adjust the K_h with loading and soil conditions (static load on cohesive soils):

$$K_h = (1/3 - 1/6) \times K_h = 1/6 \times 2019.564 = 336.59 \text{ kN/m}^3$$

- Determination of pile parameters

- a. Modulus of elasticity (E) = 29510.98 MPa
- b. Moment of inertia (I) = $3.25 \times 10^{-4} \text{ m}^4$
- c. Cross section modulus (S) = 0.002604 m^3
- d. Compressive strength (f_c') = 39.425 MPa
- e. Embedded pile length (D) = 15 m
- f. Diameter or width of the pile (b) = 0.25 m
- g. Eccentricity of applied load (e_c) = 0
- h. The resistance moment of the pile (M_y) = $S \times f_c'$ = 102.6627 kNm

- Determine β_h for cohesive soil

$$\beta_h = ((K_h \times b) / (E \times I))^{1/4} = ((336.59 \times 0.25) / (29510 \times 3.25 \times 10^{-4}))^{1/4} = 0.3059$$

- Determine the dimensionless length factors

$$\beta_h D = \beta_h \times D = 0.3059 \times 15 = 4.588$$

- Determine whether the pile is a long pile or a short pile $\beta_h D > 2.25 \rightarrow$ then, it is a long pile

- Determine other soil parameters that are located along the embedded pile

$$\begin{aligned} \gamma &= 13.72 \text{ kN/m}^3 \\ c_u &= 9.1 \text{ kN/m}^2 \\ \text{sliding angle} &= 1.04 \end{aligned}$$

- Determine ultimate lateral load for a single pile (Q_u) (Fig. 6)

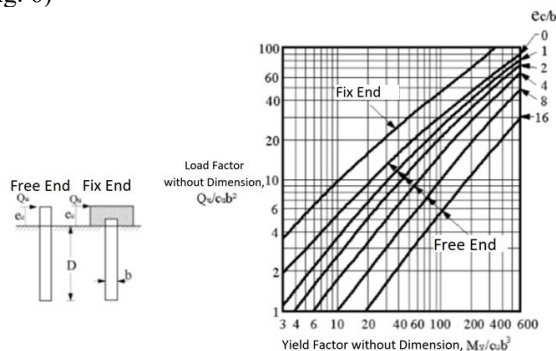


Fig. 6. The ultimate lateral capacity from long pile on cohesive soil [8].

$$\begin{aligned} M_y / c_u b^3 &= 722.023 \\ Q_u / c_u b^2 \text{ based on chart} &= 175.1 \\ Q_u &= 99.58 \text{ kN} \end{aligned}$$

- Determine the maximum allowable bearing capacity of a single-pile (Q_m)

$$Q_m = Q_u / 2.5 = 99.58 / 2.5 = 39.83 \text{ kN}$$

- Allowable bearing capacity for single pile (Q_a) in kN

$$y K_h b D / Q_a = 4.5$$

With a maximum deflection requirement is 10 mm (Fig. 7), a Q_a value obtained is 2.804 KN

- Compare Q_a to Q_m .

Because of $Q_a < Q_m$, then it used Q_m .

$$Q_m = 39.83 \text{ KN}$$

- Working load Reduction for Pile Group in kN

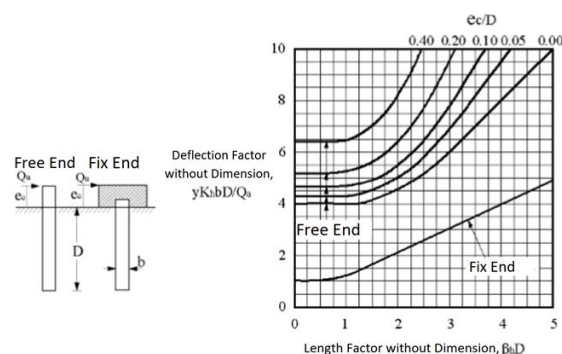


Fig. 7. Lateral deflection on the ground surface from the pile in cohesive soil [8].

Table 6. The reduction factor of working load in the pile.

z	Reduction factor
8b	1
6b	0.8
4b	0.5
3b	0.4

$$\text{where: } z / b = 0.65 / 25 = 2.6$$

From Table 6, the reduction factor is 0.36

$$\begin{aligned} Q_m &= 0.36 \times 39.83 \\ &= 14.338 \text{ kN} \end{aligned}$$

- Allowable bearing capacity of pile group (Q_{group}) in kN

$$\begin{aligned} Q_a &= Q_m \times \text{total pile} \\ &= 39.83 \times 4 \\ &= 159.32 \text{ kN} \end{aligned}$$

- Control of pile group allowable bearing capacity with Ultimate lateral load.

For lateral loads, it was obtained from ETABS modeling that is 187.99 kN.

$$\begin{aligned} Q_a &< Q_m \\ 159.32 \text{ kN} &< 187.99 \text{ kN} \end{aligned}$$

Considering the liquefaction potential of the soil, the foundation is not strong enough to resist the lateral loads.

2.4.3 Settlement of pile group foundation

$$S_g = (2 \times q \times (B_{q1})^{1/2}) / N60 < 25 \text{ mm}$$

$$q = Q_g / (L_g \times B_g) = 126753.98 / (150 \times 150) = 6753.98 \text{ kg/cm}^2$$

$$I = 1 - (L / (8 \times B_g)) \geq 0.50$$

$$= 1 - (150 / (8 \times 150)) \geq 0.50$$

$$= 0.875 \geq 0.50 \dots \text{OK}$$

$$S_g = (2 \times q \times (B_{q1})^{1/2}) / N60 < 25 \text{ mm}$$

$$= (2 \times 6753.99 \times (100 \times 0.875)^{1/2}) / 60 < 25 \text{ mm}$$

$$= 16.6 \text{ mm} < 25 \text{ mm}$$

Therefore, the settlement of pile group foundation is still in the safe category.

2.5 The column capacity

2.5.1 Capacity of the columns

The capacity of column was analyzed using the P-M interaction diagram method. The P-M diagram is a graph of the boundary region that shows the variety of combinations of axial loads and moments that describe the capacity of the columns.

Table 7 shows the result of the bending moment capacity of the column. The checks for the column bending capacity are summarized in Table 8 and Fig. 8. Since the bending moment demand is larger than the capacity, the columns at 1st and 2nd floors were found to be deficient in bending under gravity and seismic loads.

Table 7. The calculation of the column bending moment.

Column	M _n (kN)	P _n (kN)	M _{nb} (kN)	P _{nb} (kN)
1 st Floor	324,46	4312,70	570,86	1820,21
2 nd Floor	174,53	3716,39	408,38	1820,21
3 rd Floor	174,53	3716,39	408,38	1820,21
Slab Floor 1	105,11	2041,85	198,01	890,52
Slab Floor 2	105,11	2041,85	198,01	890,52

Table 8. The checking on column capacity.

Column	Mu (kN)	φMn (kN)	Capacity Check
1 st Floor	260,12	259,57	NOT OK
2 nd Floor	167,70	139,62	NOT OK
3 rd Floor	118,50	139,62	OK
Slab Floor 1	49,15	98,99	OK
Slab Floor 2	37,79	98,99	OK

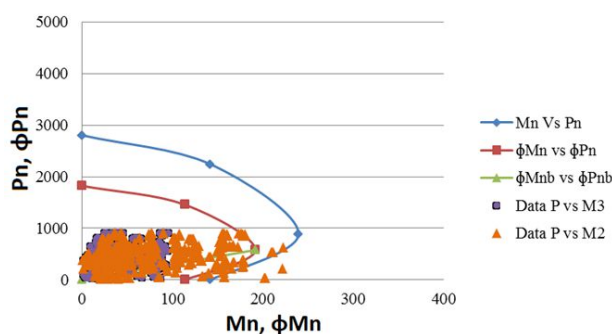


Fig. 8. P vs M interaction diagram of column.

2.5.2 Shear capacities of the columns

According to SNI-2847-2013, the shear strength of concrete structures is a combination of concrete (V_c) and steel (V_s) contributions [11]. The calculation results of the column shear capacity are shown in Table 9. The table shows that the preliminary evaluation results (strength-related checks) indicate adequate in the shear stress carrying capacity of the columns.

Table 9. The calculation of the column shear force.

Column	Vu (kN)	Vc	Vs	φVn (kN)	Capacity Check
1 st Floor	54,75	146,51	70,77	167,69	OK
2 nd Floor	52,38	146,51	70,77	167,69	OK
3 rd Floor	38,04	71,66	60,32	98,99	OK
Slab Fl. 1	44,10	71,66	60,32	98,99	OK
Slab Fl. 2	29,00	71,66	60,32	98,99	OK

2.6 The beam capacity

2.6.1 Bending moment capacities of the beams

The detailed evaluation of the bending capacity of beam elements is summarized in Table 10.

Table 10. The calculation of bending moment in the beams.

Beam	Mr (Design) kNm	Mu (Ultimate) kNm	Check
Sprandel Beam			
Span : 5.6 m (2 nd floor)	68.043	42.57	OK
Span : 5.6 m (3 rd floor)	68.043	30.594	OK
Span : 5.6 m (roof floor)	68.043	33.502	OK
Span : 4.8 m (2 nd floor)	68.043	187.388	NOT
Span : 4.8 m (3 rd floor)	68.043	161.769	NOT
Span : 4.8 m (roof floor)	68.043	131.645	NOT
Span : 4.5 m (2 nd floor)	68.043	58.32	OK
Span : 4.5 m (3 rd floor)	68.043	52.674	OK
Span : 4.5 m (roof floor)	68.043	15.457	OK
Span : 3.0 m (2 nd floor)	68.043	44.35	OK
Span : 3.0 m (3 rd floor)	68.043	57.708	OK
Span : 3.0 m (roof floor)	68.043	107.5	OK
Span : 2.1 m (2 nd floor)	68.043	91.576	OK
Span : 2.1 m (3 rd floor)	68.043	74.833	OK
Span : 2.1 m (roof floor)	68.043	41.764	OK
Joist			
Span : 4.5 m (2 nd floor)	16.546	57.708	NOT
Span : 4.5 m (3 rd floor)	16.546	74.396	NOT
Span : 4.5 m (roof floor)	16.546	41.356	NOT
Ring Beam (RB)			
Span : 5.6 m (RB 1)	20.678	26.373	NOT
Span : 4.8 m (RB 1)	20.678	73.372	NOT
Span : 4.8 m (RB 2)	20.678	68.43	NOT
Span : 4.5 m (RB 1)	20.678	14.327	OK
Span : 3.0 m (RB 1)	20.678	77.527	NOT
Span : 3.0 m (RB 2)	20.678	73.97	NOT
Span : 2.1 m (RB 1)	5.078	14.124	NOT
Span : 2.1 m (RB 2)	5.078	15.183	NOT

From the table, it can be seen that all the joists, almost all ring beams, and the sprandel beams with a span of 4.8 m were stressed beyond the ultimate limits. While the capacity of beams with a span of 2.1 m; 3.0 m; 4.5 m; and 5.6 m on each floor were found structurally adequate to support the working loads from floor and roof framing.

2.6.2 Shear capacities of the beams

Table 11 shows the calculation results of the beam shear capacities. As seen in the table, the shear capacity is less than the shear demand on some beams, it is indicating the deficiency of beam in shear under vertical and seismic loads.

The evaluation was done initially in order to determine the state and see if it is possible to strengthen the existing structures. Thus, the above evaluation suggests that the frame needs to be strengthened and retrofitted.

Table 11. The calculation of shear force in the beams.

Beam	Vr (Design) kN	Vu near support kN	Check
Sprandel Beam			
Span : 5.6 m (2 nd floor)	166.44	56.67	OK
Span : 5.6 m (3 rd floor)	166.44	49.50	OK
Span : 5.6 m (roof floor)	166.44	56.70	OK
Span : 4.8 m (2 nd floor)	166.44	176.83	NOT
Span : 4.8 m (3 rd floor)	166.44	170.46	NOT
Span : 4.8 m (roof floor)	166.44	177.54	NOT
Span : 4.5 m (2 nd floor)	166.44	83.47	OK
Span : 4.5 m (3 rd floor)	166.44	37.08	OK
Span : 4.5 m (roof floor)	166.44	33.37	OK
Span : 3.0 m (2 nd floor)	166.44	51.97	OK
Span : 3.0 m (3 rd floor)	166.44	36.35	OK
Span : 3.0 m (roof floor)	166.44	33.37	OK
Span : 2.1 m (2 nd floor)	166.44	123.91	OK
Span : 2.1 m (3 rd floor)	166.44	103.3	OK
Span : 2.1 m (roof floor)	166.44	65.4	OK
Joist			
Span : 4.5 m (2 nd floor)	159.48	123.91	OK
Span : 4.5 m (3 rd floor)	159.48	103.3	OK
Span : 4.5 m (roof floor)	159.48	65.4	OK
Ring Beam (RB)			
Span : 5.6 m (RB 1)	138.28	42.6	OK
Span : 4.8 m (RB 1)	138.28	139.82	NOT
Span : 4.8 m (RB 2)	138.28	141.83	NOT
Span : 4.5 m (RB 1)	138.28	31.61	OK
Span : 3.0 m (RB 1)	138.28	171.67	NOT
Span : 3.0 m (RB 2)	138.28	152.22	NOT
Span : 2.1 m (RB 1)	138.28	19.06	OK
Span : 2.1 m (RB 2)	138.28	30.10	OK

3 Conclusion

1. The Hamka's Dormitory Building has a liquefaction potential at a depth of 1.2 - 8 meters.
2. Considering the potential liquidation that occurs in the soil of the dormitory building, the pile foundation is unable to resist the required load.
3. Structural elements of the building such as the columns and beams cannot resist the load if the soil liquefaction potential was considered.
4. The building structure should be strengthened to prevent the damage if the big earthquake occurs that cause the soil liquefaction.

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References

1. R.W. Boulanger, I.M. Idriss, *CPT and SPT Based Liquefaction Triggering Procedure*, University of California (2014)
2. H.B. Seed, I.M. Idriss, *Journal of soil mechanics and foundation, Division, ASCE*, **97**, **9**, 1249 – 1273 (1971)
3. S.L. Kramer, *Geotechnical earthquake engineering*, Prentice Hall, Englewood Cliffs, N.J., **653** (1996)
4. A. Egi, *Analysis of Non-Engineered Buildings on Conflicted Land (Case Study: Housing in Padang City)*, Andalas University, Padang (2010)
5. Public Work Department, *PPIUG (Indonesian Standard of Building Loads)*, Jakarta (1983)
6. National Standardization Agency (BSN), *SNI 1726:2012, Design Method of Earthquake Resistance for Buildings and other Structures*, Jakarta (2012)
7. M.D., Braja, G.V. Ramana, *Principles of Foundation Engineering, Second Edition*, PWS-Kent Publishing Company (1990)
8. A. Hakam, *Bridge Manual Design: Chapter 8 Design of Pile Foundation*, Padang (2008)
9. H.G. Putra, A. Hakam, D. Lastaruna, *Liquidity Potential Analysis based on Sondir Test Data (Case Study: Hajj Agus Salim Building and Lapai, Padang City)*, Andalas University, Padang (2009)
10. R. Yuliet, *Heap of Road Agency on Soft Soil in Aie Pacah Area, Padang City*, Andalas University, Padang (2006)
11. National Standardization Agency (BSN), *SNI 2847:2013, Standard of concrete detailing for Building Structure*, Jakarta (2013)