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STRUCTURAL EVALUATION OF NURUL HAQ SHELTER BUILDING CONSTRUCTED ON LIQUEFACTION PRONE AREA IN PADANG CITY - INDONESIA

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ABSTRACT: West Sumatra Province, especially Padang City, is an area prone to the earthquake that is not only can trigger a tsunami but also liquefaction disaster. In order to face the disaster, the Padang city government planned to build as many as shelters as a vertical evacuation building. One of them is Nurul Haq shelter located in a coastal area that has high liquefaction potential. A structural evaluation of the shelter was conducted to check the capacity of the existing shelter structure in resisting the working loads. From the result of the soil evaluation, it was found that the soil in the shelter location has high liquefaction potential. Therefore, the shelter structure is analyzed using specific response spectrum of the earthquake loads considering soil liquefaction, which is 1.5 higher than those on the non-soil liquefaction. The tsunami loads were calculated used based on FEMA P-646. The analysis result shows that the shelter building is not capable of resisting the working loading, in which the elements of the beams and foundations don't have enough capacity to resist the working loads, especially earthquake and tsunami loads. Furthermore, the shelter building should be retrofitted before being used as a vertical evacuation building.

Keywords: Earthquake, Tsunami, Shelter, Soil Liquefaction, Structural Evaluation

1. INTRODUCTION

Padang city is located on the west coast Sumatra, which borders on the open sea (Indian Ocean) and the active two-plate collision zone, the Indian and Asian plates, makes Padang city one of the most earthquake-prone and tsunami-wasting cities [1]. Therefore, after the occurrence earthquake on September 30, 2009, the government began to take action by establishing a vertical evacuation building is called a shelter. By using the shelter, people in Padang city can reach a safe place from a tsunami puddle in a shorter period of time when horizontal evacuation cannot run properly. One of the shelters is Nurul Haq shelter, located in Parupuk Tabing sub-district, Padang city, Indonesia.

Shelters are usually built in residential areas close to the coast because shelters can be used by people around the settlement to take shelter when the earthquake and tsunami occur. However, this becomes dangerous because the shelter is likely to be built in areas where the soil has the potential to liquefy. This makes the shelter collapse before it can be used as a shelter after the earthquake. Therefore, it is necessary to evaluate the structure of the Nurul Haq shelter building, whether this shelter has taken into account the potential aspects of liquefaction, earthquake, and tsunami-resistant building standards in its planning design.

2. EVALUATION OF LIQUEFACTION POTENTIAL

One of the causes major destruction of various types of structures during an earthquake is the loss of strength or stiffness of the ground. This process is called soil liquefaction. Liquefaction of the ground results in the settlement of buildings, failure of earth dams, landslides, and other hazards [2-4]. Another way of evaluating the soil liquefaction potential in the field is to prepare correlation charts with the standard penetration resistance, as shown in Fig. 1. The corrected N-SPT value (N') can be obtained using the Eq.(1):

$$N' = C_N\ N \quad (1)$$

The correction factor can be expressed as Eq.(2) (Liao and Whitman, 1986) [2]:

$$C_N = 9.78\sqrt{\frac{1}{p'_o}} \quad (2)$$

Where
N = Field standard penetration test value
C_N = Correction factor to convert to an effective overburden pressure (p'_o)
p'_o = effective overburden pressure in kPa

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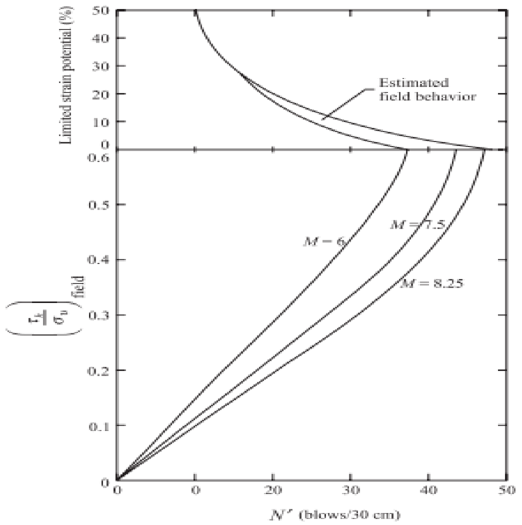

Fig. 1 Variation of $(\tau_h/\sigma_v)_{field}$ with N' and M (Seed, 1979) [2]

From Fig. 1, it can be seen that if N' is more than 30, the liquefaction is unlikely to occur [2].

According to Seed and Idris, the maximum shear stress determine from the shear stress-time history during the earthquake can be converted into an equivalent number of significant stress cycle [1], using Eq.(3) :

$$\tau_{av} = 0.65\, C_D \left[\left(\frac{\gamma\, h}{g} \right) a_{max} \right] \quad (3)$$

Where C_D is a stress reduction factor. The range of C_D for different soil profiles is shown in Fig. 2, along with the average value up to a depth of 12 m.

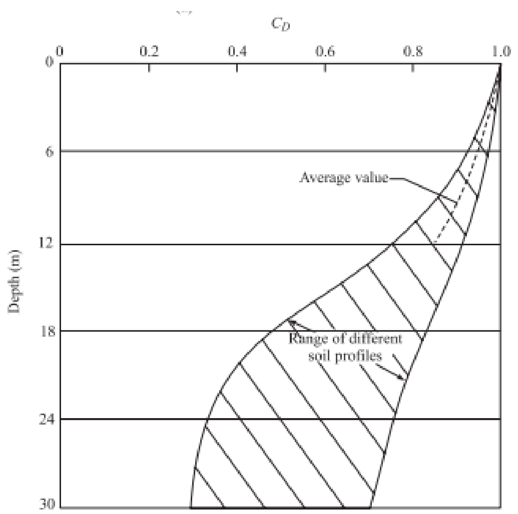

Fig. 2 Range of the shear stress reduction factor C_D for the deformable nature of soil (after Seed and Idriss, 1971) [2]

A geotechnical investigation carried out in a deposit of soil provided the field SPT-N at Nurul Haq shelter as given in Table 1.

Table 1 Field SPT-N value of the soil in the Shelter area

A Range of depth (m)	Depth (m)	SPT-N (blows/30 cm)	SPT-N average (blows/30 cm)
0 - 6	1.55 3.55 5.55	34 54 52	46.7
6 - 11	7.55 9.55	48 50	49
11 - 18	11.55 13.55 15.55 17.55	38 22 18 10	22
18 - 25	19.55 21.55 23.55	8 8 12	9.3
25 – 30	25.55 27.55 29.55	10 24 28	20.7
30 – 40	31.55 33.55 35.55 37.55 39.55	19 14 40 44 49	33.2

From Table 1, it can be seen that depths of soil between 30 m and 40 m is silt soil with SPT-N average of 33.2 (correction not necessary). Table 2 shows an empirical relationship of granular soils physical properties with N' values [5].

Table 2 Empirical values for ϕ, D_r and unit weight of granular soils based on corrected N' (after Bowles, 1977) [5]

Description	Very loose	Loose	Medium	Dense	Very dense
Relative density, D_r	0-0.15	0.15-0.35	0.35-0.65	0.65-0.85	0.85-1
Corrected Standard Penetration N' value	0-4	4-10	10-30	30-50	>50
Approximate angle of internal friction, ϕ (°)	25-30°	27-30 °	30-35 °	35-40 °	38-43 °
A range of approximate unit weight, γ (kN/m^3)	11-15.7	14.1-18.1	17.3-20.4	17.3-22	20.4-23.6

Correlations of cohesive soil physical properties with N values are crude, and therefore, correction of N values in cohesive soils is not necessary [5].

Soil profile at the Nurul Haq shelter is shown in Fig. 3. The groundwater table is encountered at a depth of 1 m measured from the ground surface.

The maximum peak ground acceleration at the site $_{max}$ = 0.6 g, with an earthquake magnitude of 7.6.

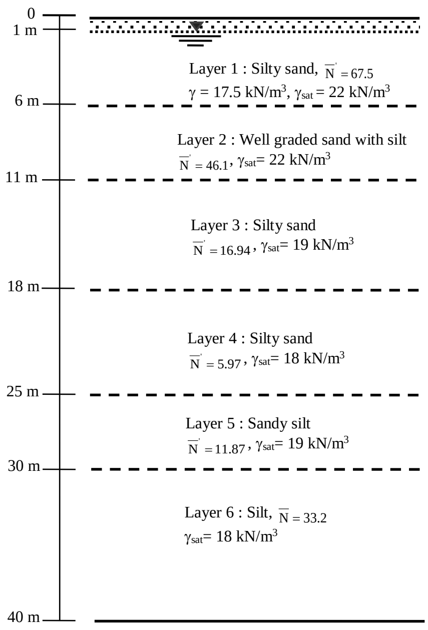

Fig. 3 Soil profile at Nurul Haq shelter

The shear resistance available and the shear stress induced in the sand deposit at different depths can be calculated using the following steps:

Step 1. Table 3 can be used to calculate the shear resistance available (τ_{av}) in the sand deposit at different depths.

Table 3 Calculating the available shear resistance

Depth (m)	N_F (blows/30 cm)	Vertical eff. stress (kPa)	C_N [Eq.(2)]	N' (blows/30 cm)	$(\tau_h/\sigma_v)_{field}$ [Fig. 1]	τ_h (kPa)
1.55	34	24.20	1.99	67.66	*	*
3.55	54	48.58	1.40	75.60	*	*
5.55	52	72.96	1.14	59.28	*	*
7.55	48	97.34	0.99	47.52	*	*
9.55	50	121.72	0.89	44.50	*	*
11.55	38	144.45	0.81	30.78	0.340	49.11
13.55	22	162.83	0.77	16.94	0.170	27.68
15.55	18	181.21	0,73	13.14	0.140	25.37
17.55	10	199.59	0.69	6.90	0.075	14.97
19.55	8	215.97	0.67	5.36	0.054	11.66
21.55	8	232.35	0.64	5.12	0.051	11.85
23.55	12	248.73	0.62	7.44	0.081	20.15
25.55	10	265.66	0.60	6.00	0.070	18.60
27.55	24	284.04	0.58	13.92	0.154	43.74
29.55	28	302.42	0.56	15.68	0.170	51.41
31.55	19	318.80	*	19	0.21	66.94
33.55	14	335.18	*	14	0.15	50.27
35.55	40	351.56	*	40	0.48	168.7
37.55	44	367.94	*	44	0.6	220.7
39.55	49	384.32	*	49	0.6	230.6

Step 2. Table 4 can be used to calculate the shear stress-induced (τ_{av}) in the sand deposit at different depths using Eq. (3)

Table 4 Calculating the induced shear resistance

Depth (m)	Total vertical stress (kN/m^2)	a_{max}/g	C_D Fig. 2	τ_{av}, kPa, Eq. (3)	Liquefaction potential
1.55	29.60	0.6	*	*	N.L
3.55	73.60	0.6	*	*	N.L
5.55	117.60	0.6	*	*	N.L
7.55	161.60	0.6	*	*	N.L
9.55	205.60	0.6	*	*	N.L
11.55	247.95	0.6	0.86	84.10	L
13.55	285.95	0.6	0.69	75.80	L
15.55	323.95	0.6	0.60	75.80	L
17.55	361.95	0.6	0.50	70.60	L
19.55	397.95	0.6	0.43	66.60	L
21.55	433.95	0.6	0.38	67.20	L
23.55	469.95	0.6	0.34	63.70	L
25.55	506.50	0.6	0.32	62.80	L
27.55	544.50	0.6	0.31	63.40	L
29.55	582.50	0.6	0.30	67.80	L
31.55	618.50	0.6	*	*	N.L
33.55	654.50	0.6	*	*	N.L
35.55	690.50	0.6	*	*	N.L
37.55	726.50	0.6	*	*	N.L
39.55	762.50	0.6	*	*	N.L

Note : N.L = Not Liquefaction, L = Liquefaction

From Fig. 2, it can be seen that C_D value at the depths > 30 m is not available so that the shear stress-induced (τ_{av}) cannot be determined.

Step 3. The factor of safety against liquefaction

$$FS_L = \frac{\tau_h}{\tau_{av}} \quad (4)$$

Check to see if $\tau_{av} \geq \tau_h$ at any depth in the soil deposit, $FS_L \leq 1$ so liquefaction occurs between this depths (Eq.(4)). From Tables 3 and 4, it can be seen that τ_{av} is greater than τ_h at the depths of soil between 11.55 m and 29.55 m, so liquefaction occurs between these depths.

3. EVALUATION OF BUILDING STRUCTURE

3.1 Location of Existing Shelter

Nurul Haq shelter is located close to the coastline, with a distance of 0.37 km towards the coastline of Padang. The pattern of land use around the location is a residential area.

3.2 Tsunami Vulnerability Level

Based on the tsunami hazard map of Padang city issued by the Regional Development Planning Board of Padang City, the depth of tsunami inundation in the location of the plan is 4-5 m is based on the contours of the Padang city area and the prediction of tsunami waves in Padang city.

3.3 Data for Building Structure

a. Structural type: Reinforced concrete
b. Concrete strength, FC' : 30 MPa
c. Steel yield strength, f_y : 400 MPa
d. Number of floors: 7 (seven) floor
e. Building height: 23.12 m
f. Building area: 36 m x 18 m
g. The thickness of slab: 15 cm

3.4 Modeling of Existing Structure

Fig. 4 shows the 3D modeling of the shelter structure using ETABS commercial software [10]. The columns and beams of the building structure are modeled as frames while the floor plates are modeled as slab elements.

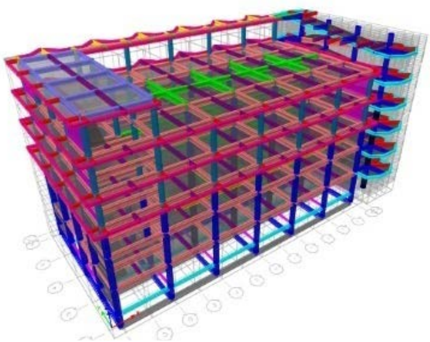

Fig. 4 Modeling of existing structure

3.5 Loading

The dead and live load refers to Minimum Loads for Building Design and Other Structures of SNI 1727-2013 [6]. The analysis of earthquake load using dynamic analysis (earthquake response spectrum) for Padang city based on SNI 1726-2012 [7] and the earthquake hazard map 2017 by making its own design of response spectrum. Refugee Live Load refers to FEMA P-646 [5], which was 250 kg/m^2.

From the N-SPT soil data, soil condition of shelter location is the middle ground (SD), which can be seen from the N-SPT value of 22.35. According to SNI 1726-2012 [7] and SNI 8460-2017 [9], the value of N-SPT of medium soil ranges from 15 to 50.

However, the soil at that location has the potential for liquefaction to be categorized as special soil (SF) according to SNI 1726-2012 [7] or SNI 8460-2017 [9]. Therefore, the design response spectrum should be multiplied by 1.5. The comparison of the response spectrums between the SD and SF soil conditions can be seen in Fig. 5.

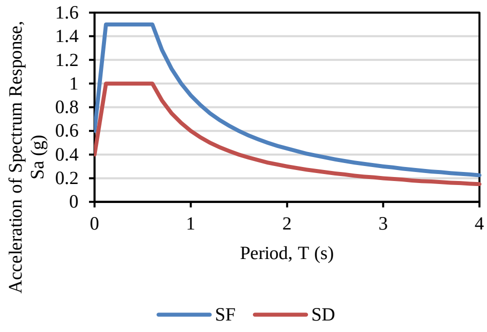

Fig. 5 Comparison of response spectrum between SD and SF soil conditions at the Nurul Haq shelter building

Tsunami loads are calculated according to the FEMA P-646 standard [8]. The magnitude of each load value based on the predicted wave height of the tsunami, the ground elevation of the shelter area, the distance from the shore and other assumptions used, which can be seen in Fig. 6.

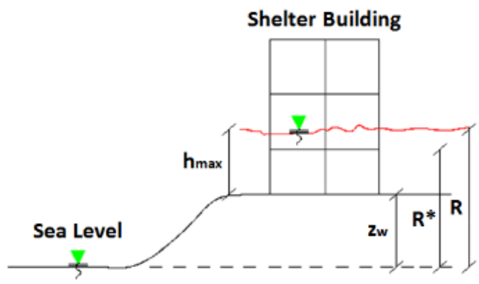

Fig. 6 Inundation plan of the tsunami [8]

The tsunami loads were calculated based on FEMA P-646 [8], as follows:

1. Hydrostatic forces
   This load is given centered on the column as high as 1/3 the maximum height of water in a tsunami-immersed area in the direction of the tsunami.
2. Buoyant forces
   Buoyant force loads are evenly distributed on the tsunami's upper floors.
3. Hydrodynamic forces
   Hydrodynamic force loads are concentrated at the column as high as ½ of tsunami water puddle in all the affected columns of tsunami flows in the direction of the tsunami.
4. Impulsive forces
   The impulse force load is evenly distributed on the structural wall as high as a tsunami puddle in the direction of the tsunami.
5. Floating debris impact forces
   The force of the impact of the debris is evenly distributed on the first part of the structure of the affected part of the stream.

6. Damming of accumulated waterborne debris
   The force due to debris is given evenly on structural elements with a minimum width of 40 ft (12 m).
7. Additional retained water loading on elevated floors. The added gravity load is evenly distributed on the top floor affected by the tsunami.
8. Uplift forces on elevated floors
   The hydrodynamic lift force is evenly distributed on the top floor affected by the tsunami.

The calculation results of applied tsunami loads on the analysis of the Nurul Haq shelter are shown in Table 5.

Table 5 Applied tsunami loads on the shelter

No.	Tsunami load	Load
1	Hydrostatic forces	28,541.7 kg
2	Buoyant forces	5,830.0 kg
3	Hydrodynamic forces	7,595.9 kg
4	Impulsive forces	11,393.7 kg
5	Floating Debris Impact forces	63,593.2 kg
6	Daming of accumulated waterborne debris	151,916.0 kg
7	Additional retained water loading on elevated floors	6,116.0 kg/m^2
8	Uplift forces on elevated floors	6.5 kg/m^2

3.6 Capacity of Structure

3.6.1 Column Capacity

P-M interaction diagram is a diagram illustrating the ability or capacity of the column based on the relationship between the moment and axial load of the column. Figs. 7-10 show P-M interaction diagrams of columns obtained from the results of the structural analysis.

Based on these P-M diagrams, it can be seen that the flexural capacity of the 1st and mezzanine floor columns can withstand the working loads including earthquake and tsunami loads because all axial and bending moment data are within the P-M column interaction diagram. For columns on the 2nd, 3rd, 4th, and roof floor also have enough capacity of resisting the working loads. The shear capacity of the columns has been calculated with the results shows that the shear capacity of the columns is able to resist the loads acting on the shelter structure.

3.6.2 Beam Capacity

Table 6 shows the flexural capacity of the beam elements. From Table 6, it can be seen that beam elements on the 1st floor, the mezzanine floor, and the 2nd floor are unable to withstand the working loads. Meanwhile, the beams on other floors are able to resist the loads.

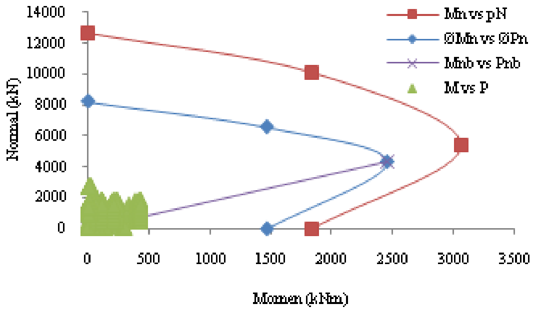

Fig. 7 P-M interaction diagram of column K1 (Ø70 cm) on1stfloor

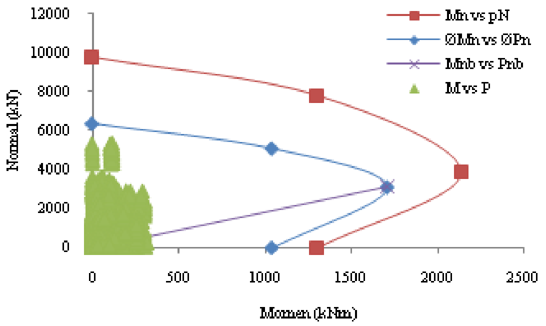

Fig. 8 P-M interaction diagram of column K2 (Ø 60 cm) on 1st floor

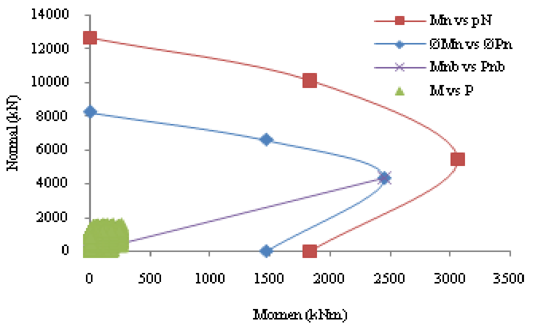

Fig. 9 P-M interaction diagram of column K1 (Ø70 cm) on the mezzanine floor

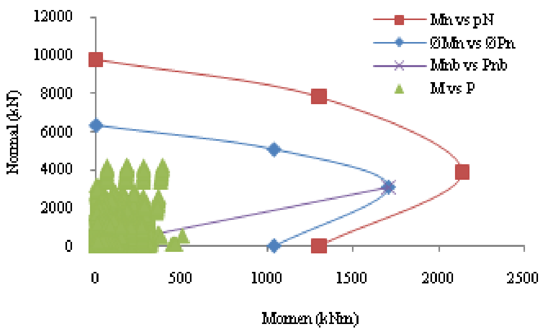

Fig. 10 P-M interaction diagram of column K2 (Ø60 cm) on the mezzanine floor

Table 6 Beam flexural capacities of the shelter

Floor	Type of beam	Num. of steel bar		ØMn	Mu	Note
		Tens.	Comp.			
First	B1	6 D 22	3 D 22	399	279	OK
	30 x 60	3 D 22	6 D 22	203	232	NOT OK
	B1	6 D 22	3 D 22	399	489	NOT OK
	30 x 60	3 D 22	6 D 22	203	483	NOT OK
	B2	8 D 22	4 D 22	677	1381	NOT OK
	35 x 75	4 D 22	8 D 22	343	1320	NOT OK
	BA	3 D 19	3 D 19	97	162	NOT OK
	25 x 40	3 D 19	3 D 19	97	212	NOT OK
Mezza-nine	B1	6 D 22	3 D 22	399	261	OK
	30 x 60	3 D 22	6 D 22	203	235	NOT OK
	B1	6 D 22	3 D 22	399	462	NOT OK
	30 x 60	3 D 22	6 D 22	203	475	NOT OK
	BA	3 D 19	3 D 19	97	137	NOT OK
	25 x 40	3 D 19	3 D 19	97	212	NOT OK
Second	B1	6 D 22	3 D 22	399	238	OK
	30 x 60	3 D 22	6 D 22	203	209	NOT OK
	B1	6 D 22	3 D 22	399	281	OK
	30 x 60	3 D 22	6 D 22	203	234	NOT OK
	B2	8 D 22	4 D 22	677	289	OK
	35 x 75	4 D 22	8 D 22	343	225	OK
	BA	3 D 19	3 D 19	97	30	OK
	25 x 40	3 D 19	3 D 19	97	17	OK

Table 7 Beam shear capacities of the shelter

Floor	Type of beam	Num. of bar	Vr	Vu	Note
First	B1	D13 - 150	420	203	OK
	30 x 60	D13 - 175	377	218	OK
	B1	D13 - 150	420	487	NOT OK
	30 x 60	D13 - 175	377	488	NOT OK
	B2	D13 - 100	747	872	NOT OK
	35 x 75	D13 - 150	554	873	NOT OK
	BA	D13 - 150	262	151	OK
	25 x 40	D13 - 200	212	151	OK
Mezza-nine	B1	D13 - 150	420	218	OK
	30 x 60	D13 - 175	377	211	OK
	B1	D13 - 150	420	487	NOT OK
	30 x 60	D13 - 175	377	513	NOT OK
	BA	D13 - 150	262	158	OK
	25 x 40	D13 - 200	212	152	OK

Table 7 shows the shear capacity of the beam elements. As seen in the table, the beams don't have enough shear capacity, especially on the 1st floor and the mezzanine floor. This indicates that the beams should be strengthened to improve its flexural and shear capacities before using the building as a shelter for the earthquake and tsunami disasters.

4. EVALUATION OF FOUNDATION

The evaluation of foundation refers to Indonesian standard, SNI 8460-2017 of article 12.2.4.3 [9] as follows :

- The shaft resistance at the soil layer with liquefaction potential should be ignored.
- The settlement in soil due to densification of soil under liquefaction must be evaluated.

Specifications of the pile foundation are :

- Type of concrete pile : Pile spun
- Embedded length : 30 m
- Dimension of pile : Ø350 mm
- Concrete strength : K.800
- Concrete comp. strength, fc' : 55 MPa
- Steel strength, fy-400 MPa : BJTD 40

Fig. 11 shows the foundation plan of the Nurul Haq shelter.

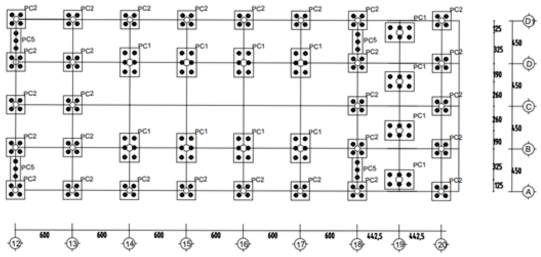

Fig. 11 Foundation plan of the Nurul Haq shelter

4.1 Design of Single Piles

The ultimate bearing capacity of a single pile (Q_u) was calculated using Eqs. (5) and (6):

$$Q_u = R_s + R_t \quad (5)$$

or

$$Q_u = f_s A_s + q_t A_t = f_s C_D \Delta d + q_t A_t \quad (6)$$

The allowable bearing capacity of the pile (Q_a) was calculated using Eq.(7):

$$Q_a = \frac{Q_u}{FS} = \frac{R_s}{FS_1} + \frac{R_t}{FS_2} \quad (7)$$

Where:

R_s = The shaft resistance
R_t = The toe resistance
f_s = The unit shaft resistance
q_t = The unit toe resistance

4.2 Ultimate Capacity of Single Piles by Using Effective Stress Method

One of the static analysis methods is used for calculating the ultimate capacity of a single pile in cohesionless, cohesive and layered soils can also be performed using effective stress based, the unit shaft resistance is calculated using Eqs.(8) and (9):

$$f_s = \beta\, \overline{p}_o' \quad (8)$$

The unit toe resistance, q_t, in kPa is

$q_t = N_t \, p_t$ (9)

Where :
β = Beta coefficient (Table 8)
N_t = Toe bearing capacity coefficient (Table 8)
$\bar{p}'_o$ = Average effective overburden pressure along the pile shaft, in kPa
p_t = Effective overburden pressure at the pile toe in kPa

Table 8 Approximated range of β and N_t coefficients (Fellenius, 1991) [2].

Soil type	ϕ'	β	N_t
Clay	25-30	0.23-0.4	3-30
Silt	28-34	0.27-0.5	20-40
Sand	32-40	0.3-0.6	30-150
Gravel	35-45	0.35-0.8	60-300

From evaluation results of the soil liquefaction potential in the shelter area, the depths of soil between 11.55 m and 29.55 m (layer 3, layer 4 and layer 5) has potential liquefaction, so the shaft resistance at the soil layer should be ignored. The shaft resistances calculations are only carried out in layers 1, 2 and 6. The unit toe resistance calculation is also carried out in layer 6.

Table 9 Summary of pile foundation capacity

Group pile	A calculated ultimate capacity of a single pile (Q_{us})			The ultimate capacity of the pile group, Q_{ug} (kN)	The allowable capacity of the pile group, Q_{ag} (kN), (FS = 3)	Load (kN)
	R_s (kN)	R_t (kN)	Q_{us} (kN)			
PC 1 (6 pile)	1221.5	732.4	1,953.9	11,723.4	3,907.8	5,526
PC 2 (4 pile)	1221.5	732	1,953.5	7,814	2,604.	2,859

The ultimate pile group capacity at the Nurul Haq shelter is being taken as the sum of the ultimate capacities of the individual piles in the group. From calculation results, it is found that the allowable pile group capacity on the PC 1 and PC2 respectively are 3,907.8 kN and 2,604 kN, this means that the PC 1 and the PC2 foundations have not enough capacity to resist the shelter building.

4.3 Settlement of Pile Groups in Layered Soils with Using Equivalent Footing Method

The settlement of pile groups is determined using Eq.(10):

$$s = H\left[\frac{1}{C'}\log\frac{p'_o + \Delta p}{p'_o}\right] \quad (10)$$

Where:
s = Total layer settlement (mm)
H = Original thickness of layer (mm)
C' = Dimensionless bearing capacity index from Fig. 12, determined from average corrected SPT-N' value N.
p'_o = Effective overburden pressure at the midpoint of layer prior to pressure increase (kPa)
Δp = Average change in pressure in layer (kPa)

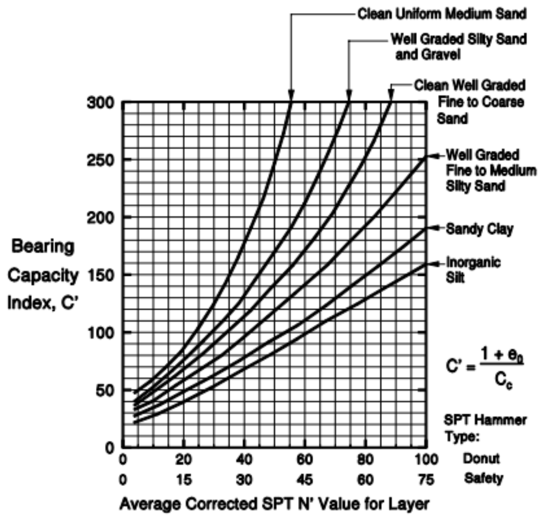

Fig. 12 Values of the bearing capacity index, C', for granular soil (Modified after Cheney and Chassie, 1993) [5]

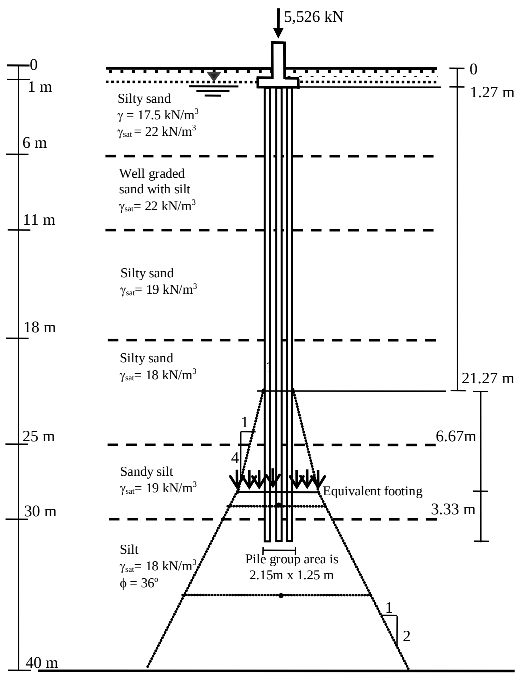

Fig. 13 Pressure distribution below equivalent footing for pile group (PC1).

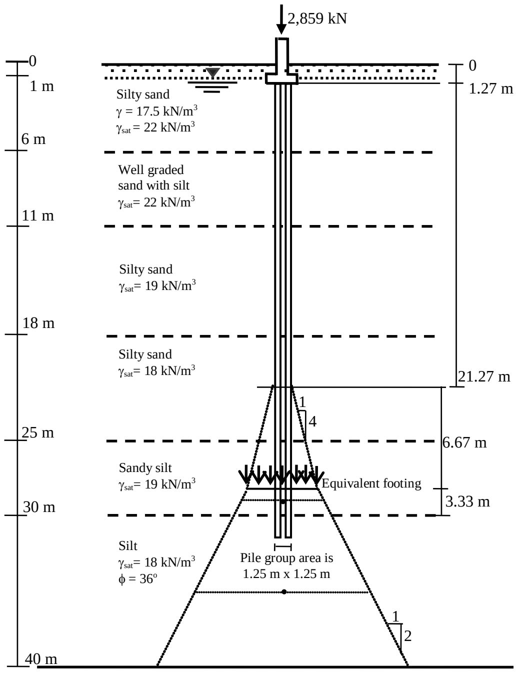

Fig. 14 Pressure distribution below equivalent footing for pile group (PC2).

The location of the equivalent footing is based on the shaft and toe resistance condition and the soil profile. The equivalent footing is placed at a depth of 2/3 from the bottom of the pile cap as shown in Figs. 13 and 14 for PC1 and PC2, respectively.

Table 10 The settlement of pile groups

Group pile	H (m)	N'_{avr}	C'	σ'_v	Load distribution surface		$\Delta\sigma_v$	S (mm)
					B (m)	Z (m)		
PC 1 (6 piles)	2.06	11.87	30	237.5	6.515	5.61	151.2	14.7
	10	33.2	60	348.0	17.55	16.64	18.93	3.8
							Total settlement	18.5
PC 2 (4 piles)	2.06	11.87	30	237.5	5.61	5.61	90.84	9.66
	10	33.2	60	348.0	16.64	16.64	10.32	2.12
							Total settlement	11.78

From Table 10, it can be seen that the total settlement on PC1 and PC2 are 18.5 mm and 11.78 mm, respectively. It can be concluded that the total pile group settlement is less than the maximum allowable pile group settlement of 25 mm [5].

5. CONCLUSIONS

1. The Soil condition at the Nurul Haq shelter between the depths of 11.55 m and 33.55 m has a potential of liquefaction.
2. The result of the structural analysis showed that flexural and shear capacities of columns are able to loads applied on the shelter structure, but the flexural and shear capacities of beams on floor 1, mezzanine and second floor have not strong enough capacities to resist the working loads.
3. Foundations of the shelter building, PC1 and PC2 have not enough capacity to resist the weight of the shelter building, especially when the occurrence of earthquake-induced liquefaction.
4. The total pile group settlement on Nurul Haq shelter is less than the maximum allowable settlement of 25 mm.
5. The shelter building should be retrofitted before being used as a vertical evacuation building for earthquake and tsunami.

6. ACKNOWLEDGMENTS

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