Liquefaction Resistance of Sands in the Northern Part of Thailand

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Abstract

There are a few active faults recently found in the western and northern parts of Thailand. These could possibly induce earthquakes of magnitude (M_L) of 5.5-6.5. Although seismic design code has been enforced in the area since 1980, the fundamental knowledge on dynamic soil behavior has not been extensively attained. Literature reviews of the existing boreholes from the two largest provinces in the north, including Chiang-Mai and Chiang-Rai, revealed that the areas are underlain by loose to medium dense sand layers found at shallow depths. The corrected SPT N-value of those sand layers varies in the range of 5 – 20 blows/ft. These borehole information, together with the result obtained from the logistic regression based on worldwide liquefaction database are used to conduct the effective stress analysis. A simple tool correlating the liquefaction probability, estimated excess pore water pressure and peak ground acceleration is proposed. Preliminary risk zones in these two provinces were identified.

1. Introduction

Thai people were not much concerned about earthquake risk until recent occurrence of several moderate earthquakes as examples shown in Table 1. The centers of a few recent medium earthquakes were in the northern and western parts of the country. Nutalaya et al. [7] formulated a database containing instrumental data of earthquakes from 1910 to 1989 within the regions bounded by latitudes 5°N to 25°N and longitudes 90°E to 110°E. This includes Thailand, Indochina, and parts of Burma and China. Their results are reproduced in Fig. 1 and 2 and Table 2. They indicated that earthquakes of local magnitude (M_L) of 6.5 with maximum ground acceleration of 0.2g may occur in the northern and western parts of Thailand.

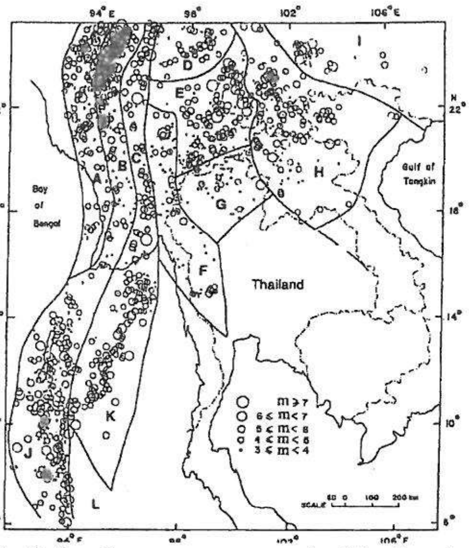

Fig. 1. Seismic source zones in Thailand and vicinity (Nutalaya et al., [7])

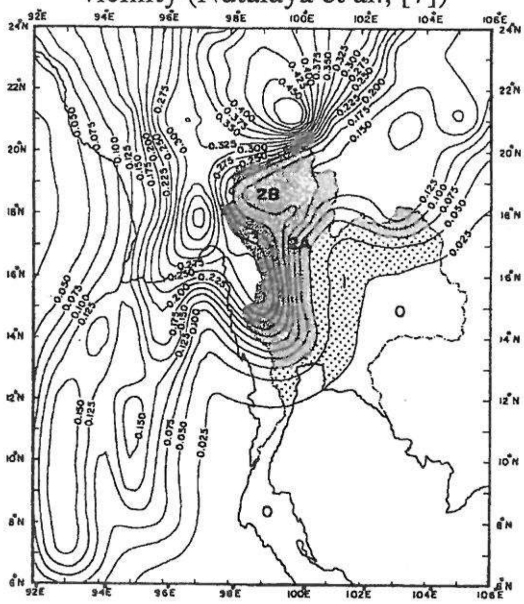

Fig. 2. Map showing contours of peak ground acceleration (in units of acceleration of gravity) with 10% chance of being exceeded in a 50-year exposure time, and seismic zones for earthquake-resistant design (Wanitchai and Lisantono, [14]

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Table 1. Examples of recent earthquakes felt in Thailand

Date	Magnitude	Center	Were Felt at
April 22, 1983	5.9	Kanchanaburi, Thailand	Bangkok, Western and northern parts
November 6, 1988	7.3	Southern of China (1,000 km from Bangkok)	Bangkok, Western and northern parts
September 29 – October 1, 1989	5.3 – 5.4 Several quakes	Western part	Bangkok, Western and northern parts
September 11, 1994	5.5	Phan District (Northern part)	Northern parts
January 22, 2003	7.5	Sumatra Island (1,000 km from Bangkok)	Bangkok
September 22, 2003	6.6	Burma (850 km from Bangkok)	Bangkok and Northern parts

Table 2. Maximum estimated M_L for seismic source zones in Thailand region (after Nutalaya et al., [7] and modified by Warnitchai and Lisantono, [14])

Zone	Name	Maximum M_L
A	Arakan Coastal Area	6.75
B	West-Central Burma Basin	7.40
C	East-Cebtral Burma Basin	7.75
D	Bhamo-Paoshan Area	5.96
E	Burma Eastern Highlands	7.30
F	Tenasserim Range	7.90
G	Northern Thailand	6.50
H	North Indochina	6.75
I	South Yunnan-Kwangsi	8.38
J	Andaman Arc	7.20
K	Andaman Basin	6.50

Among the northern provinces of Thailand, Chiang-Mai and Chiang-Rai were selected as the studied area due to the following reasons:

1. They are located close to some of the recently found active faults.
2. They are the most densely populated areas in the north.
3. They are underlain by layers of soft clay and/or loose to medium dense sand at shallow depths as illustrated in Fig. 3 and 4 (Anantasech and Thanadpipat, [1]). The existence of the loose to medium dense sand layers at shallow depths (2 – 8 m from ground surface) infers certain levels of liquefaction risk of those two provinces.
4. From the metropolitan records of both provinces, more than 80% of housings (1 – 2 stories building) in the center of cities were built on shallow foundation. These are the structures most prone to damages due to liquefaction and/or partially increase in excess pore water pressure.
5. Fig. 5 shows evidences indicating past occurrence of liquefaction in a suburban area of Chiang-Rai. Trace of sand extruding into the upper gravel layer clearly indicated

past liquefaction experience of the lower sand layer.

Although full initialization of liquefaction may not be the case, partial development of excess pore water pressure might cause damages to 1 – 2 stories housing which is usually built on ground or short piles. A preliminary study is therefore needed to survey the liquefaction susceptibility of the areas. Integration among field parameters, probabilistic study, and dynamic analytical results is used as a primary tool for further detail evaluation.

2. Study methodology

Figure 6 shows the general methodology adopted. There are three main information required in the procedure, including:

(a) Subsurface information. Around 50 existing boring logs were collected from each province. Examples of the boring logs are shown in Fig. 7 and 8. The sub soils in both provinces are subject to wide variation. Nevertheless, layers of loose to medium dense sand are found at depths of 2 – 8 m in most of the area. Figure 9 summarizes the gradation of sands found in both provinces. Great variation of grain size distribution is observed. The average diameter, D_{50}, of sands varies in the range of 0.2 to 1.5 mm.

(b) Laboratory determination of liquefaction resistance. Existing cyclic triaxial tests determining the liquefaction resistance of sand were used to obtain some effective stress parameters required in the effective stress analysis (Iai et al., [3]).

(c) Existing liquefaction database (Liao and Whithman, [6]). Since there is no liquefaction database existing for Thailand, the worldwide liquefaction database is used as a reference for determination of other related parameters.

Those three components shall be integrated to obtain a specific tool or guideline for indicating earthquake liquefaction potential in the studied area.

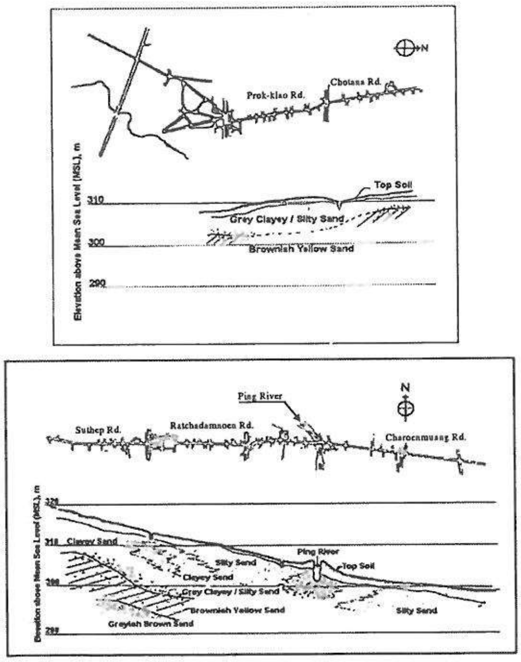

Fig. 3. Typical subsoil section in Chiang-Mai Province (modified from Anantasech and Thanadpipat, [1])

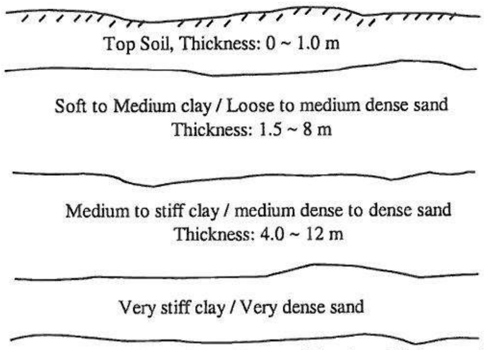

Fig. 4. Typical subsoil profile in Chiang-Rai province

3. Logistic model for evaluation of liquefaction potential

Liquefaction occurs primarily in loose to medium dense saturated sands found at shallow depth. Its occurrence is a function of soil type, relative density, age, amount of clay fraction CF, and intensity and duration of the earthquake motion (Seed and Idriss, [8]). In the past decades, several methods have been proposed to evaluate earthquake liquefaction potential. These methods range from purely empirical to highly analytical and require various degrees of laboratory and/or in situ testing. The most common approach is to use deterministic chart expressing the relationship between the corrected SPT N-value and the cyclic stress ratio (CSR) such as that shown in Fig. 10 (Seed et al., [9]). A deterministic line is subjectively drawn to separate events of liquefaction phenomena.

Juang et al. [5] applied the logistic regression model to create probability function of liquefaction events. The logistic regression is considered more appropriate for events with only two occurrence patterns, i.e., liquefaction or non-liquefaction. In the present study, the liquefaction database complied by Liao and Whithman [6], which composes of 278 case studies is used as reference for conducting the logistic regression analysis. Among the 278 reported cases, 120 events are from Japan, 100 events from California, 20 events from China, and 38 events from other locations around the world.

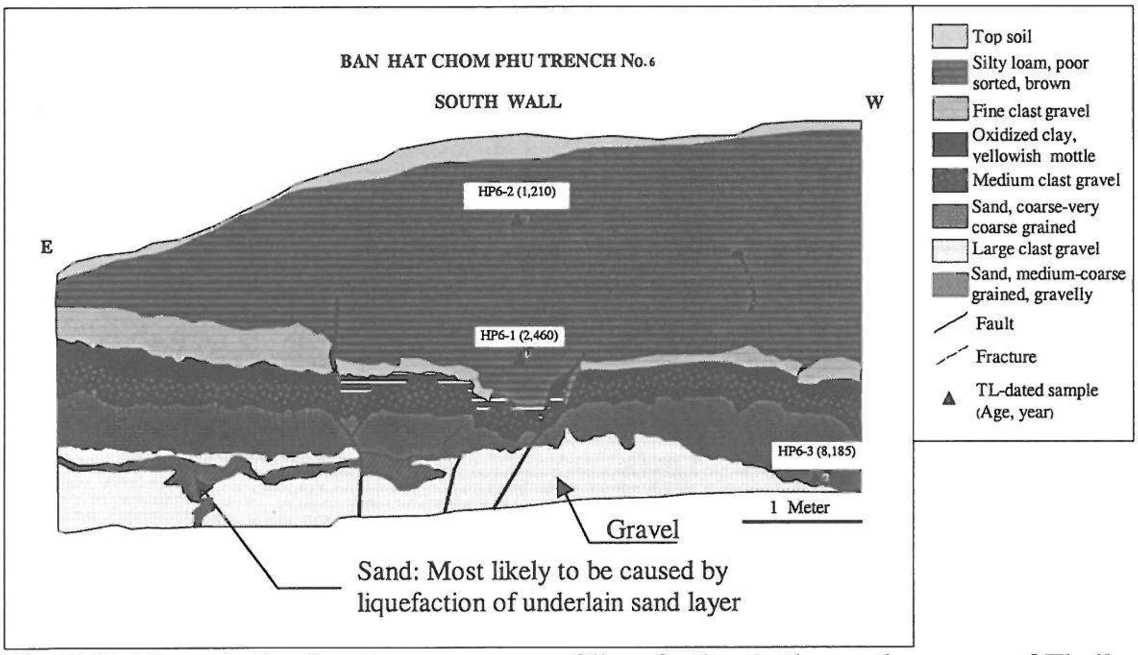

Fig. 5. Evidence indicating the occurrence of liquefaction in the northern area of Thailand

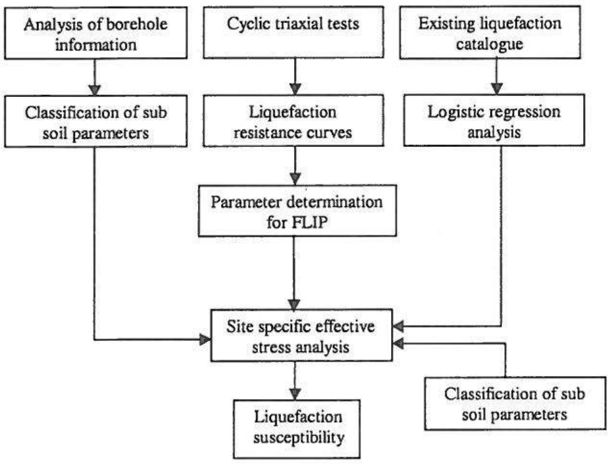

Fig. 6. General study methodology adopted

Each event in the database is represented through a binary variable Y which indicates whether liquefaction did occur ($Y = 1$) or did not occur ($Y = 0$) and a vector expressing the physical variables, $X = [X_1, X_2, ..., X_m]^T$. An observation (event) is then written in a short form as (Y, X). Compilation of n events obtained from the database enables to define the liquefaction probability function (P_L) as;

$$\ln\left(\frac{P_L}{1 - P_L}\right) = \beta_0 + \beta_1 X_1 + ... + \beta_m X_m \quad (1)$$

Where P_L is the probability that liquefaction will occur and β_i is the regression constants.

The most common set of physical variables adopted in several liquefaction studies is the cyclic stress ratio (CSR) and the corrected SPT resistance. The following probabilistic equations provide the best fit to the database.
For earthquake magnitude of 7.5:

$$\ln\left(\frac{P_L}{1-P_L}\right) = 9.119 - 0.243(N_1)_{60} + 3.458\ln(CSR_{7.5}) \qquad (2)$$

For earthquake magnitude of 5.5:

$$\ln\left(\frac{P_L}{1-P_L}\right) = 6.354 - 0.242(N_1)_{60} + 3.450\ln(CSR_{5.5}) \qquad (3)$$

Where;

$(N_1)_{60}$ is the corrected SPT resistance which is normalized to an equivalent overburden pressure of 100 kPa and a hammer energy ratio or hammer efficiency of 60%;

$CSR_{7.5}$ is the cyclic stress ratio generated at the site normalized to a magnitude of 7.5;

$CSR_{5.5}$ is the cyclic stress ratio generated at the site normalized to a magnitude of 5.5.

Figure 11 and 12 show set of probability curves defined by Eqn. (2) and (3), together with the deterministic criteria defined by Seed et al. [11]. The correlation of regression for Eqn. (2) and (3) is 0.637. The success rate in classification of liquefaction from both equations is greater than 80% for both liquefied and non-liquefied cases. Since the probabilistic line at P_L = 30% well traces the deterministic criteria proposed by Seed et al. [11], it is used to determine the success rate.

Since Chiang-Mai and Chiang-Rai are located in the seismic zone G which probable causes earthquake magnitude (M_L) of 5 to 6 with maximum ground acceleration (a_{max}) of 0.2g (Fig. 1 and 2), Eqn. (3) is more appropriate as the probability correlation for further investigation. Factor of safety computed following Seed et al. [9] at various values of P_L is then obtained and summarized in Table 3. At P_L of 5%, there are more than 80% of the sandy sites subject to a certain level of liquefaction susceptibility.

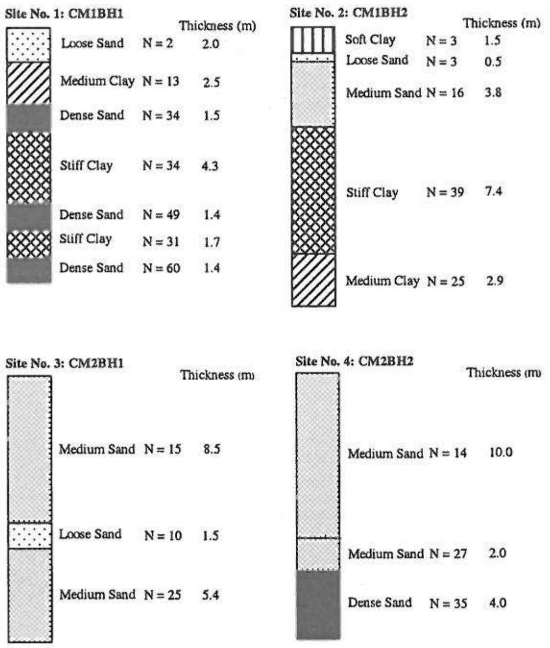

Fig. 7. Examples of the soil profiles and soil properties collected from Chiang-Mai

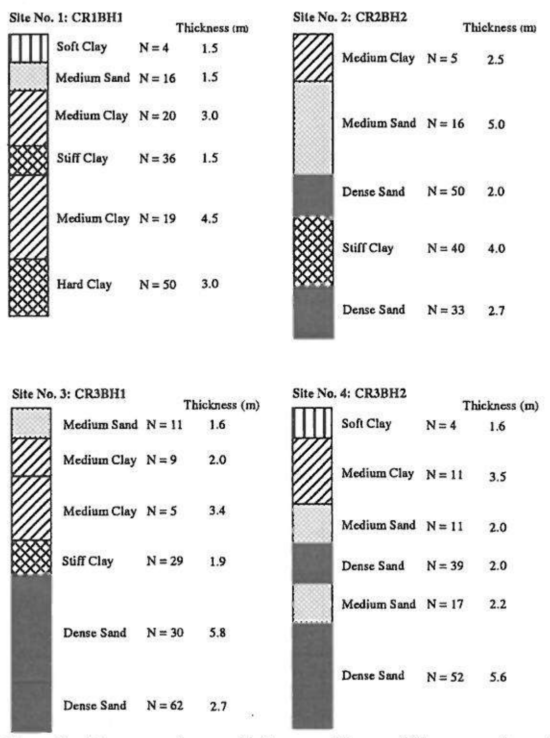

Fig. 8. Examples of the soil profiles and soil properties collected from Chiang-Rai

Table 3. Summary of the estimated values of factor of safety based on the procedure proposed by Seed et al. [9]

Chiang-Mai				Chiang-Rai			
Site no.	Factor of Safety			Site no.	Factor of Safety		
	$P_L = 5\%$	$P_L = 10\%$	$P_L = 30\%$		$P_L = 5\%$	$P_L = 10\%$	$P_L = 30\%$
1	0.51	0.64	0.95	1	2.74	3.40	5.03
2	0.51	0.63	0.94	2	1.39	1.72	2.55
3	0.66	0.82	1.21	3	2.01	2.50	3.70
4	1.26	1.56	2.31	4	0.84	1.03	1.53
5	1.68	2.08	3.08	5	0.36	0.45	0.67
6	0.57	0.70	1.03	6	0.60	0.74	1.10
7	1.23	1.52	2.25	7	1.00	1.24	1.83
8	0.83	1.02	1.51	8	0.99	1.23	1.82
9	0.64	0.79	1.17	9	1.26	1.57	2.32
10	0.89	1.10	1.63	10	0.77	0.96	1.42
11	1.23	1.52	2.25	11	0.49	0.61	0.90
12	1.80	2.23	3.30	12	1.05	1.30	1.92
13	1.66	2.06	3.06	13	0.86	1.06	1.57
14	0.69	0.87	1.27	14	0.50	0.63	0.93
15	0.87	1.08	1.60	15	1.05	1.30	1.93
16	0.50	0.62	0.92	16	10.24	12.72	18.81
17	0.52	0.65	0.96	17	0.78	0.98	1.44
18	0.58	0.71	1.06				
19	1.42	1.77	2.61				
20	0.81	1.00	1.48				
21	1.09	1.35	1.99				
22	1.03	1.29	1.91				
23	1.95	2.43	3.59				
24	0.85	1.05	1.55				
25	0.65	0.82	1.20				
26	2.04	2.53	3.74				
27	1.06	1.31	1.93				
28	0.45	0.56	0.83				
29	0.86	1.07	1.58				

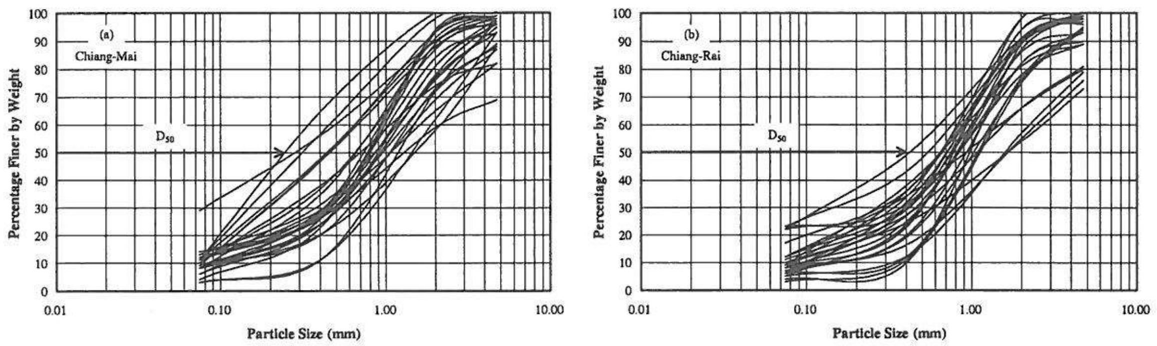

Fig. 9.Grain size distribution of sands: (a) Chiang-Mai; (b) Chiang-Rai

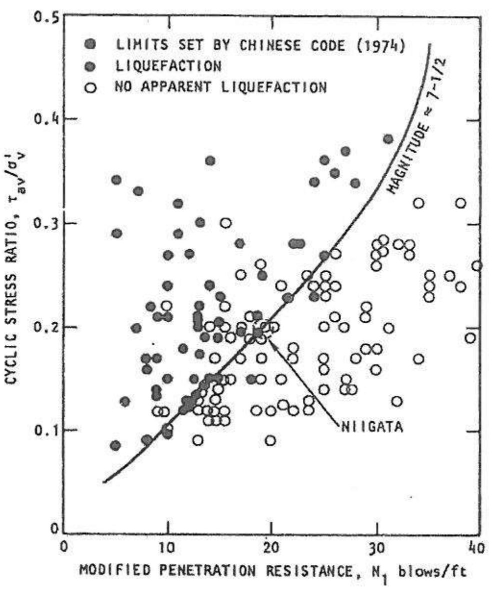

Fig. 10. Example of deterministic method of liquefaction evaluation derived from empirical data for soils with $D_{50} > 0.25$ mm (Seed et al., [9])

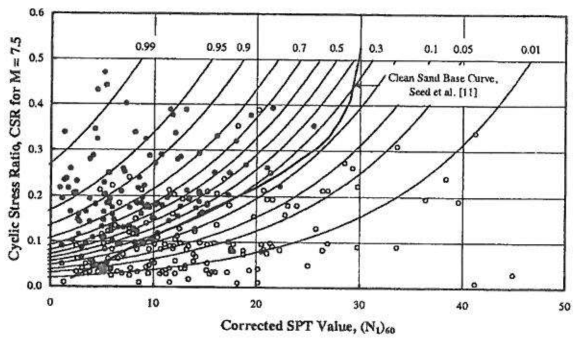

Fig. 11. Contours of equal probability of liquefaction (P_L) for magnitude, M = 7.5 with deterministic line by Seed et al. [11]

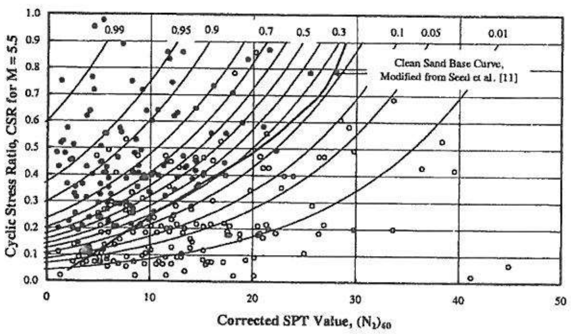

Fig. 12. Contours of equal probability of liquefaction (P_L) for magnitude, M = 5.5 with deterministic line modified from Seed et al. [11]

4. Analysis for estimation of excess pore water pressure

The previous section indicates certain levels of liquefaction risk in the studied area, as a consequence, it is further necessary to somewhat quantify the risk. The most common and direct method is to evaluate the possible amount of excess pore water pressure, which requires the effective stress analysis.

Due to the lack of strong motion record in Thailand, three input motions recorded from elsewhere with different predominant periods were adopted. Their recording station and estimated predominant period are summarized in Table 4. There are quite a few correlations being recognized for estimation of shear wave velocity of soils. Table 5 summarizes those adopted in the present study for various soil types.

The computer program called "*FLIP* (Finite element analysis of liquefaction program)" developed by Iai et al. [3] was used. The effective stress model used in "*FLIP*" requires ten parameters; two of which specify elastic properties of soil, other two specify plastic shear behavior, and the rest specify dilatancy, as summarized in Table 6. The parameters were determined from the SPT N-values and the result of the undrained cyclic triaxial tests as shown in Fig. 13.

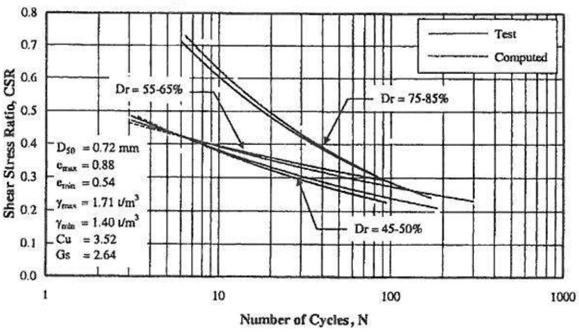

Fig. 13. Test and computed liquefaction resistance curves

The analytical procedure is outlined in Fig. 14. For each site, the minimum SPT N-value of sand was selected in order to determine the value of cyclic stress ratio (*CSR*) from the probability curves (Fig. 12). The maximum ground surface acceleration (a_{max}) for the specific P_L was then computed from the simplified equation proposed by Seed et al. [9] as:

$$\mathrm{CSR} = \frac{\tau_{ave}}{\sigma'_{vo}} = 0.65\,\frac{\sigma_{vo}}{\sigma'_{vo}}\cdot\frac{a_{\max}}{g}\cdot r_d \qquad (4)$$

Where σ_{vo} = total overburden pressure on sand layer under consideration and,

σ'_{vo} = initial effective overburden pressure on sand layer under consideration.

Note that the term r_d was calculated following Liao and Whitman [6] as:

$$r_d = 1.0 - 0.00765\,z \quad \text{for } z \le 9.15\ m \qquad (5a)$$

$$r_d = 1.174 - 0.0267\,z \quad \text{for } 9.15\ m < z \le 23\ m \qquad (5b)$$

In the analysis, the input maximum base acceleration was randomly scaled so that the computed maximum ground acceleration was similar to the prescribed value given by Eqn.(4).

Table 4. Summary of input motions used in analyses

Earthquake	Year	Station	a_{max} (g)	T_p (sec)
Northridge	1994	Topanga	0.33	0.31
El Centro	1940	El Centro	0.34	0.68
Loma Prieta	1989	Yerba Buena Island	0.065	1.41

Table 5. Fundamental soil properties used in analysis

Soil type	Formulation	Reference
Soft clay	$V_s = 68.7 \cdot S_u^{\ 0.475}$ (m/sec) $S_u = t/m^2$	Dickenson [2]
Medium to stiff clay	$V_s = 96.926 \cdot N^{\,0.314}$ (m/sec) N = Uncorrected SPT N-value	Imai and Tonouchi [4]
Silty sand	$V_s = 56.388 \cdot N_c^{\ 0.5}$ (m/sec) N_c = Corrected SPT N-value	Seed et al. [9]
Sandy soil	$V_s = 100.584 \cdot N_c^{\ 0.29}$ (m/sec) N_c = Corrected SPT N-value	Sykora and Stokoe [12]
Strain dependent characteristics of shear modulus and damping ratio	-	Vucetic and Dobry [13]
K_{ma}	$K_{ma} = \dfrac{2G(1+\upsilon)}{3(1-2\upsilon)}$ (kPa) υ = Poisson's ratio = 0.33	-
G_{ma}	$G_{ma} = \rho \cdot V_s^2$ (kPa) ρ = Soil density	-
ϕ_f	-	Peck, Hanson, and Thornburn [17]
ϕ_p	28 degrees	Ishihara et al. [16]
H_m	0.30	Ishihara [15]

Table 6. Model parameters (Iai et al., [3])

Parameter	Value	Type of Mechanism	Kind of the Parameter
K_{ma}	See Table 5	Elastic volumetric	Rebound modulus
G_{ma}	See Table 5	Elastic shear	Shear modulus
ϕ_f	See Table 5	Plastic shear	Shear resistance angle
ϕ_p	See Table 5	Plastic dilatancy	Phase transformation angle
H_m	See Table 5	Plastic shear	Hysteretic damping factor at large shear strain level
p_1	0.6-0.7	Plastic dilatancy	Initial phase of dilatancy
p_2	0.4-0.8	Plastic dilatancy	Final phase of dilatancy
w_1	9.5-38.5	Plastic dilatancy	Overall dilatancy
s_1	0.005	Plastic dilatancy	Ultimate limit of dilatancy
c_1	1.0	Plastic dilatancy	Threshold limit

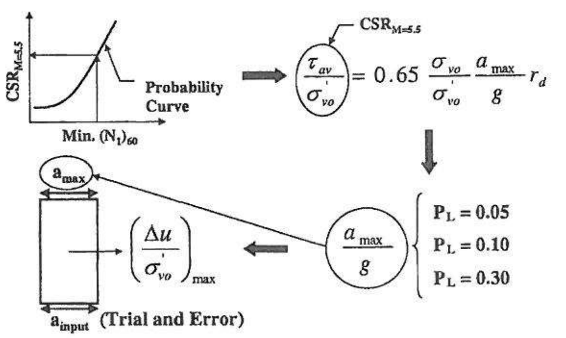

Fig. 14. Flow diagram for evaluation of pore water pressure generation

The total of twenty-nine sandy sites within Chiang-Mai City and seventeen sandy sites within Chiang-Rai City were analysed. Figure 15 and 16 show the typical analytical results by plotting the maximum pore water pressure ratio, $\Delta u / \sigma'_v$, against the maximum ground acceleration. The vertical line crossed at a_{max} = 0.2g is drawn for reference. The points located on the left of this line represent sites where factor of safety is less than 1.0 (corresponding to those shown in Table 3). The pore water pressure ratio for cases when P_L = 5% varies in the range of 0.1 – 0.8. Figure 17 shows the maximum pore water pressure ratio at level of -2.5 m from ground surface. The pore water pressure ratio for cases when P_L = 5% varies in the range of 0.1-0.5. Although near surface sand may not experience liquefaction, factor of safety of shallow foundation can be greatly reduced due to decrease in effective stress.

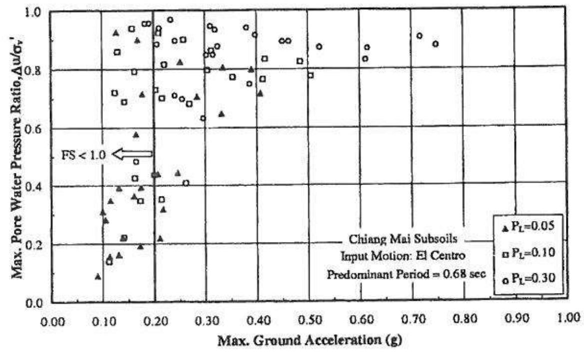

Fig. 15. Relationship between maximum ground acceleration and maximum pore water pressure ratio for Chiang-Mai

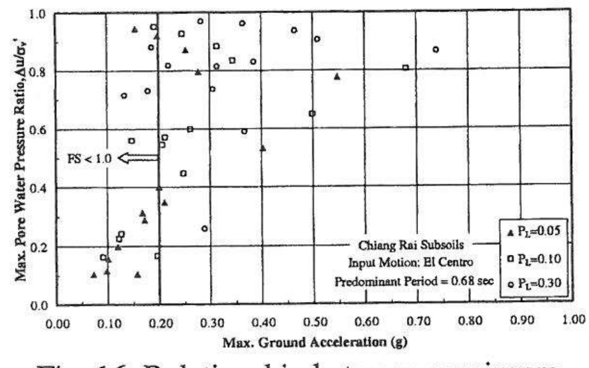

Fig. 16. Relationship between maximum ground acceleration and maximum pore water pressure ratio for Chiang-Rai

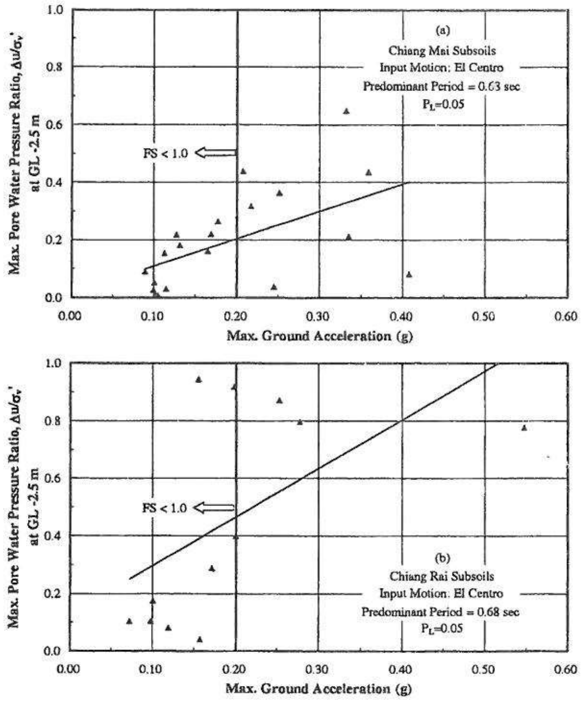

Fig. 17. Maximum pore water pressure at GL –2.5 m: (a) Chiang-Mai; (b) Chiang-Rai

5. Conclusions

The liquefaction probability due to medium earthquakes in the northern parts of Thailand, particularly Chiang-Mai and Chiang-Rai, were studied. Logistic regression model using the worldwide liquefaction database was used to form the probabilistic base correlation between cyclic stress ratio and the SPT resistance. The factor of safety, with certain level of liquefaction probability, was then determined for about 50 specific sites in both provinces. In compilation to the effective stress analysis, it was found that with P_L of 5%, there are more than 80% of the investigated sites prone to liquefaction with the excess pore water pressure ratio varies in the range of 0.1 – 0.8. This may cause discernible damage to the 1 – 2 stories housing which is general rest on shallow foundation or short piles.

6. Acknowledgements

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