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AN EXPERIENCE OF USING DYNAMIC CONE TEST TO EVALUATE LIQUEFACTION POTENTIAL OF LIQUEFIED SUBSOIL LAYERS

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Abstract

This article presents a considerably successful experience of using Dynamic Cone Penetration Test or well-known in Thailand as Kunzelstab Penetration Test (KPT) to assess liquefaction potential of subsoil layers in place of Standard Penetration Test (SPT). In this study, KPT was performed at eight different sites where liquefaction occurred as well as four sites without liquefaction due to the 2014 Chiang Rai earthquake. The values of N_{KPT} obtained from KPT were converted in to the values of N_{SPT} before, along with the results of soil classification, being used in Idriss and Boulanger's 2014 procedure of calculation for the values of factor of safety against liquefaction which were found reasonable. The smaller size and lighter weight make KPT more favorable in comparison to SPT, which the later has some difficulties when field test must be performed at sites with limited space or sites impossible to be reached by vehicles carrying SPT equipment. KPT tools are simple and can be made with a relatively low cost. Another advantage of KPT was the penetration resistance of the subsoil can be obtained continuously throughout the penetration depth. The only disadvantage of KPT was it may not be performed deeper than 4-6 meters due to its relatively small diameter of the penetrating rods. However, this limited depth should be sufficient for regions where biggest earthquake magnitude is expect to be around M_L 6.0-6.5 such as Thailand because ground damages are not likely to occurs when the thickness of non-liquefied top soil layers is more than 2-3 meters such as found at 4 sites investigated without ground damage in this study.

Keywords: earthquake, soil liquefaction, liquefaction potential, dynamic cone test, Kunzelstab penetration test

1. Introduction

In 2014, a 6.3 M_L earthquake and hundreds of aftershocks struck the southern area of Chiang Rai Province in northern Thailand. The main shock and, probably, a few strong aftershocks induced soil liquefaction at several locations within the area of approximately 30 kilometers around the epicenter. The liquefaction caused quite significant damages including cracks on ground and highway pavement, failure of highway embankment, settlement and collapsing of several houses such as shown in Fig.1 and Fig.2.

Fig.1- Failure of a highway embankment

Fig.2 - Settlement of a shallow foundation

In the past, there have been not many strong earthquakes in this country. After the series of this earthquake, earthquake-induced liquefaction has become more concerned since there are several active faults with sufficient potential to cause earthquakes of 6.0 or larger. Most of them are in mountainous and rural area where houses are normally built on shallow foundations resting approximately 1 meter below ground surface. Those houses can be easily damaged like the one shown in Fig.2 if liquefaction occurs at the soil layers underneath the footings. Therefore, identifying liquefiable sites is important.

There are several methods for evaluation of liquefaction. The stress-based approach initiated by Seed and Idriss in 1967 [1] has been common in practices worldwide as well as several revisions which have been made until nowadays [2, 3]. The concept of this approach is to compare the earthquake-induced cyclic stress ratio (CSR) to the cyclic resistance ratio (CRR) of a given soil layer. The value of CRR divided by CSR is called "factor of safety against liquefaction: *FSL*". The calculation for *FSL* requires soil parameters such as corrected N-values from Standard Penetration Test (SPT) or $(N_1)_{60CS}$, unit weight, and percentage of soil particles passing sieve #200 (percentage of fine content: *%FC*) from field and laboratory tests. Groundwater level is also necessary. The anticipated earthquake magnitude and peak ground acceleration are required as well.

The values of $(N_1)_{60CS}$ normally represent how dense or strong the soil is. Generally, the values can be obtained by SPT. However, investigation of subsoil by SPT is somewhat difficult for those people living in rural areas because it is relatively costly. Working space for SPT also becomes a concern at some places with limited space. In addition, machines and tools used for this test need transportation close to the investigation points as much as possible. Interestingly, there is an in-situ testing method known as Dynamic Cone Penetration or Kunzelstab Penetration Test (KPT). This test utilizes tools with much lighter weight when compared with SPT, and it requires no machine. In Thailand, this test has been used for many years in testing for bearing capacity of soils expected to support the foundations of towers carrying electric power lines running through almost everywhere including rice fields, mountains and woods. Therefore, it is very interesting to try KPT as an alternative to SPT.

2. Kunzelstab Penetration Test

Kunzelstab Penetration Test or KPT is a test for resistance of soil layers to the penetration of a steel rod with a cone-shaped driving head attached to the tip and an anvil attached to the top of the rod as illustrated in Fig.3. They are driven into the ground by dropping a 10-kg hammer on the anvil from a high of 50 centimeters. However, unlike SPT, KPT's driving head cannot collect any soil sample during the penetration. Therefore, another tool such as hand auger is required to collect soil samples. Therefore, groundwater table can be observed in the auger hole. Fig.4 and Fig.5 show KPT operation and using a hand auger to collect soil samples, respectively.

KPT records number of the blows required to drive the rod and the attached head 20 centimeters in the ground. The blow counts are called N_{KPT} and it can be retrieved continuously through the depth of the investigation. Groundwater table can be observed in the hole left by the auger operation.

At this point, KPT seems very simple. However, the calculation of *FSL* needs values of $(N_1)_{60CS}$. Therefore, it is necessary to convert N_{KPT} to $(N_1)_{60CS}$. Another problem is due to the relatively small diameter of the rods. The blow counts might be not reliable when driving KPT rod and the leading head in to the ground deeper than 4 or 5 meters. Therefore, 3 questions listed below was established.

1) How to convert N_{KPT} to $(N_1)_{60CS}$?

2) By using the converted $(N_1)_{60CS}$ with the stress-based method, would it capable to reasonably identify the soil layers with FSL below 1.0 ?

3) With a limited rod length of 4 meters, is KPT capable to detect or reach liquefiable soil layers?

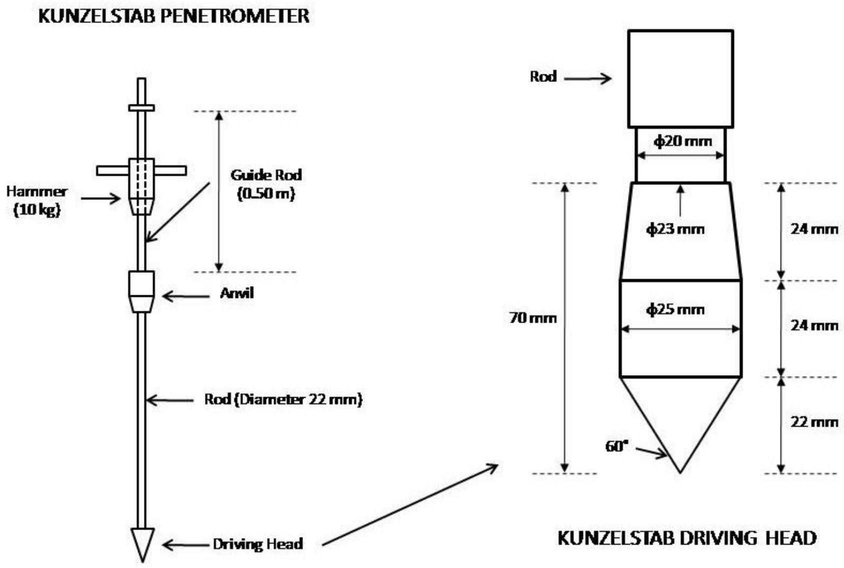


Fig.3 - Kunzelstab penetration kit

Fig.4 - Performing KPT

Fig.5 - Collecting soil samples by a hand auger

3. Converting N_{KPT} to $(N_1)_{60CS}$

As previously indicated, KPT has been used for years in Thailand. The Electricity Generating Authority of Thailand (EGAT) has suggested the following equation they used to convert N_{SPT} from N_{KPT} [4].

$$N_{SPT} = 0.539(N_{KPT} + 0.954) \tag{1}$$

It was not clear that Eq.1 was suitable for which type of soils. Therefore, an experiment was conducted to investigate the relationship between $(N_1)_{60CS}$ and N_{KPT} by running KPT investigation near 6 SPT boreholes with sandy soils. By averaging 3 or 4 of N_{KPT} at the same depth where SPT N-values were measured, the values of $(N_1)_{60CS}$ and N_{KPT} can be plotted as shown in Fig.6.

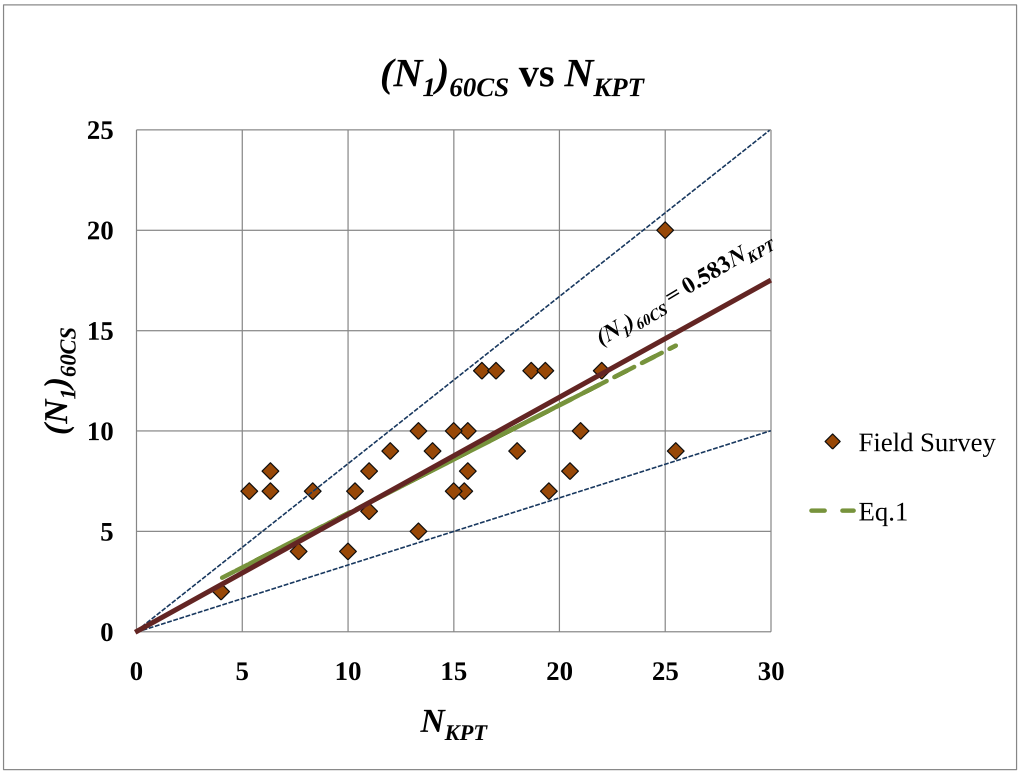


Fig.6 - Observation of $(N_1)_{60CS}$ values and N_{KPT} values in sandy soils

From Fig.5, Eq.1 seems reasonable with the data points. However, this research preferred an equation yielding the origin, therefore another relationship between $(N_1)_{60CS}$ and N_{KPT} may be approximated as shown in the following equation.

$$(N_1)_{60CS} = 0.583\, N_{KPT} \tag{2}$$

4. KPT Investigation and Liquefaction Potential Evaluation

To answer Questions 2 and 3, this study select 9 sites with liquefaction-like evidences such as sand boils, cracks appearing on ground surface, settlement of ground or building, and slope failure. As shown in Fig.7, those sites were located within the radius of 30 kilometers around the epicenter of the main shock occurred in the evening of May5, 2014. For comparison, 3 sites (No.7, 8, and 9) without any previous report of liquefaction were also selected.

The investigation at each site comprised of recording N_{KPT} and collecting soil samples by hand auger to the maximum depth of 4.2 meters except when N_{KPT} values became too high or the driving head and the rod were broken. The soil samples were then tested at the geotechnical engineering laboratory at Chiang Mai University in Chiang Mai. Groundwater table was observed, however, asking local people for possible groundwater table when the earthquake occurred was more important.

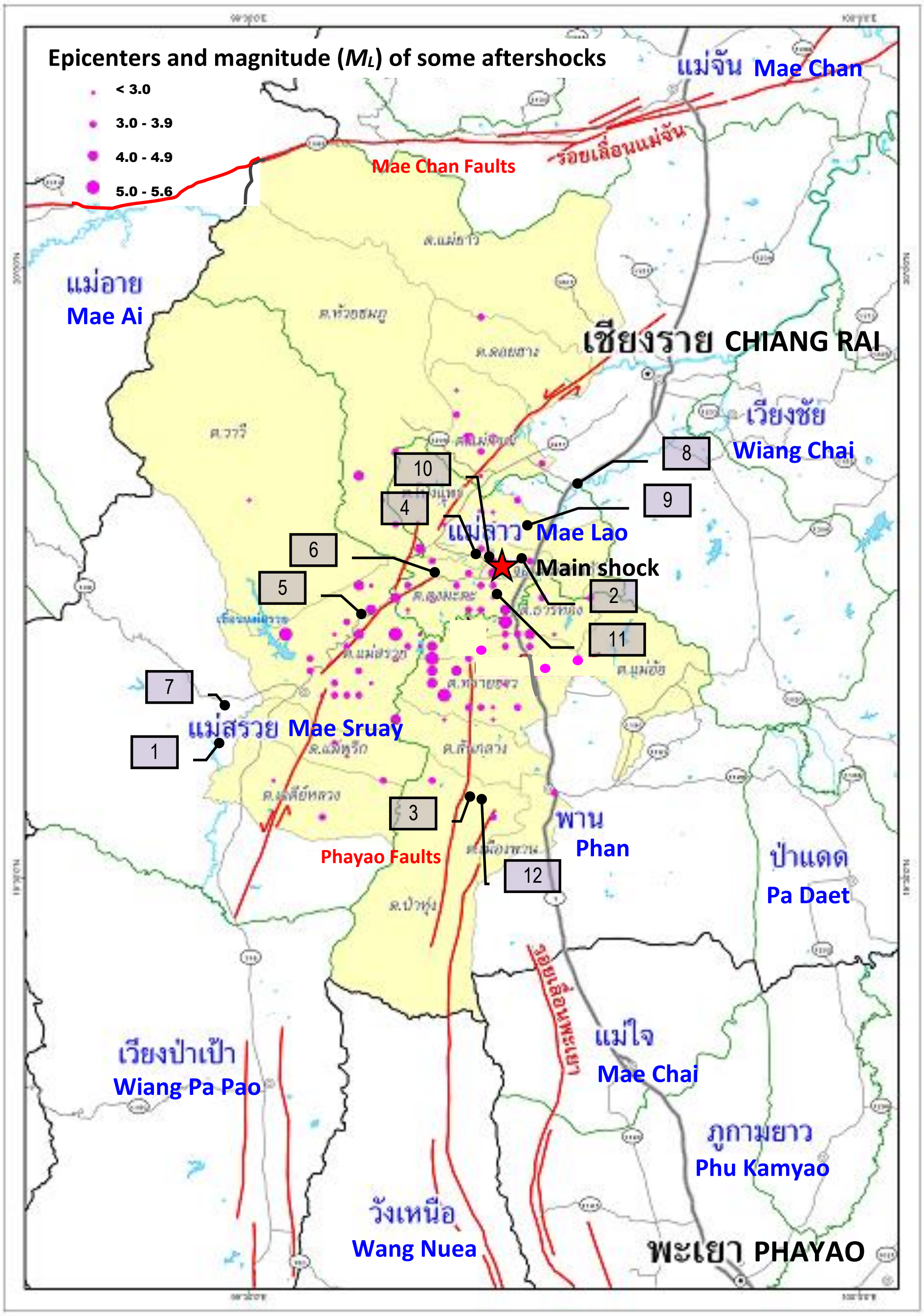


Fig.7 Approximate locations of KPT investigation in this study.

(Modified from the 2014 earthquake map by the Department of Mineral Resources of Thailand [5].)

The values of factor of safety against liquefaction (*FSL*) were then calculated for every thickness of 20 centimeters throughout the depth as demonstrated in Table 1. The calculation was based on the stress-based method revised occasionally until the revision made by Boulanger and Idriss in 2014 [3]. In the calculation, the values of peak ground acceleration (*PGA*) of 0.15g, 0.20g, 0.25g and 0.30g were examined. One important assumption was the liquefied soils can still be re-liquefied again.

4.1 Location 4: an Example of the Investigation and Analysis

The case at Location 4 is presented here as a demonstration of the investigation and analysis. At Tha Ma-O Village, locating just approximately 1.5-2.0 km from the epicenter, some houses were damaged by the earthquake vibration and/or liquefaction. However, the case that gained a lot of interest was collapsing of a house as shown in Fig.8 and 9.

Fig.8 - The house before the earthquake Fig.9 - The house after the earthquake

(Figures from Earthquake Observation Division of Thailand)

As shown in Fig.9, some large cracks can be observed on the ground. This was probably a sign of liquefaction occurring in the subsoil layers. This house was built on an approximately 1-meter fill. Therefore, investigation by KPT and obtaining soil samples by hand auger were made on the original ground off the fence in order to obtain information of the subsoil as much as possible. Fig.10 shows the grain size distribution curves of those soil samples plotted in the liquefiable range suggested by the Japanese Seismic Code for Harbor Structures [6].

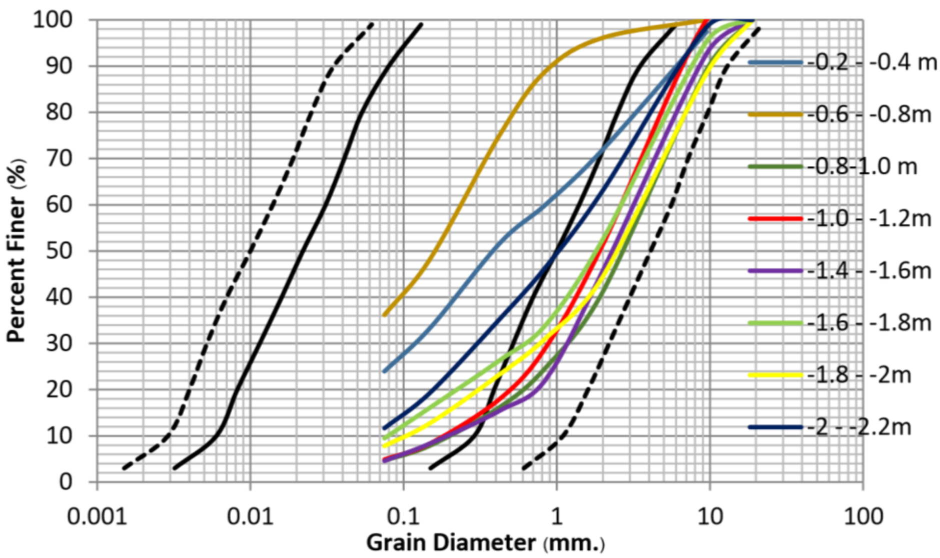
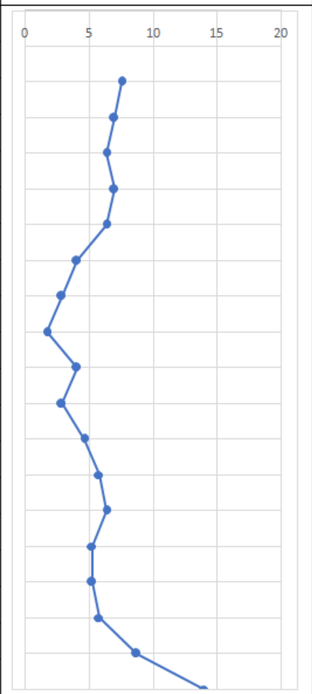


Fig.10 Grain size distribution of the soil samples from Tha Ma-O Village

The soil samples were mostly clayey sand in loose condition. With groundwater at approximately 0.8 m below the original ground surface, *FSL* values were calculated and summarized as shown in Table 1.

Table 1 - Result and analysis of *FSL* of the subsoil layers at Location 4

Depth (m.)	N-KPT	$(N_1)_{60CS}$	$(N_1)_{60CS}$	SOIL	Typical Name	PL	LL	% of grains < 0.075 mm.	*FSL*
-0.2		-		-	-	-	-	-	
-0.4	13	8		SC	Clayey Sand	22.15	29.89	23.96	
-0.6	12	7		SC	Clayey Sand	22.85	31.1	28.220	
-0.8	11	6		SC	Clayey Sand	24.47	54.68	36.170	
-1.0	12	7		SC	Clayey Sand	30.06	37.41	4.650	
-1.2	11	6		SC	Clayey Sand	31.13	38.77	4.890	
-1.4	7	4		-	-	n/a	n/a	n/a	
-1.6	5	3		SC	Clayey Sand	25.36	35.69	4.590	
-1.8	3	2		SP	Poorly Graded Sand	24.2	28.7	9.490	
-2.0	7	4		SC	Clayey Sand	29.2	36.98	7.820	
-2.2	5	3		SC	Clayey Sand	30.76	38.93	11.660	
-2.4	8	5		Soil samples below -2.4 m were not retained in the auger due to groundwater (at approximately -0.8 m). Calculation of the corresponding FSL was made by assuming 10% of fine content.					
-2.6	10	6							
-2.8	11	6							
-3.0	9	5							
-3.2	9	5							
-3.4	10	6							
-3.6	15	9							
-3.8	24	14							



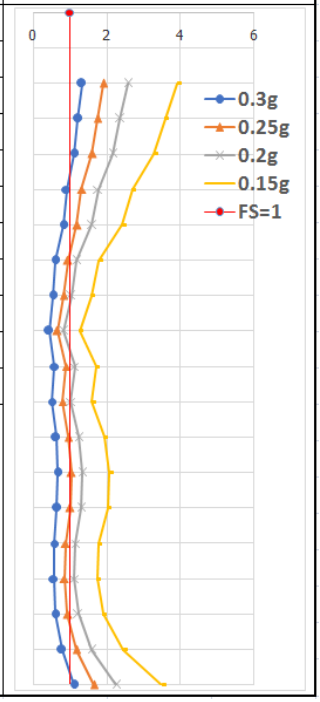


Table 1 show that liquefaction occurred differently when analyze with different values of *PGA*. The table shows that liquefaction might occurred from the depth of approximately 1.0 m to 3.8 m below the ground surface with *PGA* of 0.3*g*. Even when calculated with 0.2g PGA, the ground still liquefiable from the depth of 1.4 m down to 3.4 m.

4.2 Summary of the KPT investigation

The *FSL* values of all 12 locations are listed in Table 2. The values were calculated by varying *PGA* ranged from 0.15*g* to 0.30*g*. However, an actual *PGA* for each site was estimated by relationship between epicenter distance and peak ground velocity measured during the 1995 Kobe Earthquake [7]. Then, the effectiveness of KPT can be evaluated. For instance, at Location 1, the distance from the epicenter was approximately 26 kilometers and the actual *PGA* should be around 0.15*g*. The calculation shows, for this value of *PGA*, the liquefaction should occur from the depth of 1.2 to 3.5 m and damage could occur at the ground surface, which was correct.

Table 2 - Factor of safety against liquefaction at various possible PGA and thickness of liquefied layers

Location	Epicentral Distance (Approx. actual *PGA*)	*PGA* used in *FSL* calculation	Layer with *FSL* < 1.0		Liquefaction or damages
1- Tung Fah Pha Village (KPT depth: 4.8 m) H_1:1.2 m of silty sand above GWT. H_2: 3.2 m of poorly graded sand below GWT. Several cracks and settlement occurred on schoolyard .	≈26 km (0.15*g*)	0.15g	-1.2	-3.5	
		0.20g	-1.2	-4.0	
		0.25g	-1.2	-6.5	
		0.30g	-1.2	-6.5	
	This village was on a river bank with a lot of oxbow lakes and loose sandy soils.				
2- San Kan Haew Village (KPT depth: 4.0 m) H_1: 1.2 m of clayey sand above GWT. H_2: 2.8 m of clayey sand below GWT Houses had some damages, sand boil, an electricity pole sank about 1 m.	≈3.5 km (0.35*g*)	0.15g	-	-	
		0.20g	-1.8	-2.4	
		0.25g	-1.6	-2.6	
		0.30g	-1.2	-3.8	
	This village was on a flood plane with a creek, the ground were mostly loose sandy soils.				
3- Pa Had Village (KPT depth: 3.4 m) H_1: 0.8 m of low plastic clay above GWT. H_2: 1.0 m of low plastic silt and clay below GWT. Footings of a house settled, this could be an example of cyclic-softening soil.	≈24 km (0.15*g*)	0.15g	-	-	
		0.20g	-1.0	-1.2	
		0.25g	-0.8	-1.6	
		0.30g	-0.6	-1.8	
	The stress-based calculation shows no liquefaction if *PGA* = 0.15*g*. Stress-based method cannot be used because the soil are low plastic silts and clay.				
4- Tha Ma-O Village (KPT depth: 4.0 m) H_1: 1.0 m filled soil plus 0.8 m of clayey sand above GWT. H_2: 2.6 m of clayey sand below GWT Cracks occurred on ground and a house near the investigation point collapsed.	≈1.5 km (0.35*g*)	0.15g	-	-	
		0.20g	-1.6	-2.0	
		0.25g	-1.4	-3.4	
		0.30g	-1.0	-3.8	
	Similar to Location 2.				

Location	Epicentral Distance (Approx. actual *PGA*)	*PGA* used in *FSL* calculation	Layer with *FSL* < 1.0		Liquefaction or damages
5- Route 118, km 141-142 (KPT depth: 4.2 m) H_1: 0.4 m of clayey sand / gravels. H_2: 2.0 m of very loose clayey sand. Cracks appeared on the asphaltic pavement surface along Mae Lao River.	≈12 km (0.20*g*)	0.15g	-	-	
		0.20g	-	-	
		0.25g	-0.4	-2.2	
		0.30g	-0.4	-2.4	
		Liquefaction might not occur but the rising of pore water pressure at the sandy soil layers should be softened.			
6- Route 118, km 151-152 (KPT depth: 2.8+3.0 m) H_1: 2.8 m of clayey sand (embankment). H_2: 1.6 m of clayey sand layers alternated with low plastic clay layers. The embankment failed by sliding.	≈3.0 km (0.25*g*)	0.15g	-	-	
		0.20g	-3.6	-3.8	
		0.25g	-3.2	-4.2	
		0.30g	-3.0	-4.4	
		There was a little creek along the left hand side of the road embankment.			
7- Pa Daet Village (KPT depth: 1.8 m) H_1: 1.8+ m of low-plastic clay and too hard for KPT below -1.6m. H_2: not found No sign of liquefaction on the ground surface, some water wells filled with sand.	≈25 km (0.15*g*)	0.15g	-	-	
		0.20g	-	-	
		0.25g	-	-	
		0.30g	-	-	
		Liquefaction was found at deeper level. There was no damage to ground surface. This case shows problem to KPT caused by hard soils.			
8- Rong Khun Village (KPT depth: 3.2 m) H_1: 3.0+ m of high-plastic clay capped by a 0.8-m thick of low-plastic silt. H_2: not found. No sign of liquefaction was found at this village. A famous temple suffered some damages.	≈11 km (0.20*g*)	0.15g	-	-	
		0.20g	-	-	
		0.25g	-	-	
		0.30g	-	-	
		This location had no liquefaction. The temple was likely damaged by structural vibration.			
9- Ton Ngaw Village (KPT depth: 4.2 m) H_1: 2.4 m of soft low-plastic clay and silt. H_2: 0.6 m. No sign of liquefaction on the ground surface, but one-story school building was slightly damaged. Ejected sand found in water well in a nearby village.	≈8 km (0.20*g*)	0.15g	-	-	
		0.20g	-	-	
		0.25g	-2.4	-2.6	
		0.30g	-2.4	-3.0	
		Liquefaction was found at deeper level. The school building was likely damaged by structural vibration.			

Location	Epicentral Distance (Approx. actual *PGA*)	*PGA* used in *FSL* calculation	Layer with *FSL* < 1.0		Liquefaction or damages
10- Huay Wai Village (KPT depth: 3.0 m) H_1:1.0 m of silty sand on top of very soft low-plastic clay. H_2: 1.0 m Several cracks appeared on the asphaltic pavement surface.	≈1.0 km (0.35*g*)	0.15g	-	-	
		0.20g	-0.8	-1.4	
		0.25g	-0.6	-2.0	
		0.30g	-0.6	-2.0	
		Similar to Location 2 and 4.			
11- Phae Village (KPT depth: 3.2 m) H_1: 2.4 m of soft low-plastic clay. H_2: 0.6 m No sign of liquefaction on the ground surface but sand injected into water wells was reported at -3 m or below.	≈3.5 km (0.25*g*)	0.15g	-2.4	-2.8	
		0.20g	-2.4	-2.8	
		0.25g	-2.4	-2.8	
		0.30g	-2.4	-2.8	
		This village was on a foothill. Groundwater was rather low and loose sand deposit was quite deep.			
12- Pa Kham Village (KPT depth: 3.4 m) H_1: 0.6 m of soft low plastic silt above GWT. H_2: 2.8+ m of very soft and near saturated low plastic clay and silt plus silty sand at deeper layers. Ground cracking appeared all over the area, san injected into wells.	≈24 km (0.15*g*)	0.15g	-	-	 Liquefaction?
		0.20g	-	-	
		0.25g	-0.6	-1.0	
		0.30g	-0.6	-3.0	
		Similarly to Location 3, the stress-based calculation shows no liquefaction if *PGA* = 0.15*g*. Stress-based method cannot be used because the soil are low plastic silts and clay.			

The table shows that most of the *FSL* calculation was reasonable. Ten out of twelve locations indicated liquefaction would occur except Locations 3 and 12. This is not a surprise because at both locations, the subsoil layers were low-plastic clay and low-plastic silt. Liquefaction assessment of those types of soils should be evaluated using criteria for cyclic failure of silts and clays [2, 8] instead.

5. Conclusion and Discussion

The use of KPT and hand auger are very simple tools but helpful for the assessment of liquefaction potential in many cases. Some valuable lessons learned and suggestion can be described below.

1) The accuracy of the assessment of liquefaction potential is mainly based on the conversion of N_{KPT} to $(N_1)_{60CS}$ which should be developed more in the future similarly to Standard Penetration Test.

2) KPT cannot be driven through hard soils. It can be damaged easily. Therefore, asking local people about the soil condition should be helpful. In the case of hard soils, SPT should be considered.

3) KPT seems to have a limited working range of about 4-5 meters due to its relatively "skinny" rods and driving head. However, this range should be sufficient for small of lightweight buildings which nornall require only shallow foundations.

4) Classification of soils samples is very important as well. Low-plastic clays and silts might have very high percent of fine content which could overestimate *FSL* values. When these types of soils are found, the criteria for failure of silts and clays or cyclic softening of silts and clays should be considered as indicated in the end of Section 4.

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