A COMPARATIVE STUDY ON LIQUEFACTION PREVENTION APPROACHES

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This paper compares Japanese and European approaches used to evaluate liquefaction potential and effectiveness of different soil improvement (SI) methods including vibro compaction and replacement.

1. INTRODUCTION

Foreign contractors involving in international projects in developing countries may employ different design approaches compared to conventional ones usually used by local engineers due to either their confidence with homeland standards instead of unfamiliar local codes, or shortage or lack of relevant local specifications and guidance. This paper discusses the liquefaction prevention design where mixed use of international codes has been observed. The main objective is to compare Japanese methods and Eurocode combined with Priebe's concept (hereinafter referred to as "Eurocode*" or "EU*") for evaluating liquefaction potential and effectiveness of SI using vibro compaction and replacement.

2. EVALUATION OF LIQUEFACTION POTENTIAL

The simplest and probably most reliable method to assess liquefaction potential seems to be empirical evaluation based on field observations and field and laboratory test data. It was originally developed and published by Seed and Idriss (1971). Basically, two variables are required for assessment of liquefaction resistance of soils: (1) cyclic stress ratio (CSR) or the seismic demand on a soil layer; and (2) cyclic resistance ratio (CRR) or the capacity of the soil to resist liquefaction.

Regarding CSR, Seed and Idriss (1971) formulated the following equation:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

Where a_{max} is peak horizontal acceleration at the ground surface generated by the earthquake; g is acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively; and r_d is stress reduction coefficient. Although r_d is totally ignored in Eurocode, it remains in our study following recommendation by Priebe (1998).

Concerning CRR, several field tests have commonly used, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). Usually, SPTs and CPTs are preferred because of the more extensive databases. Criteria for evaluation of liquefaction resistance based on the SPT have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ where $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of approximately 100 kPa and a hammer energy ratio or hammer efficiency of 60%. According to Youd et al. (2001), CRR of clean sand for earthquake magnitude $M_w = 7.5$ can be obtained using following equation:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10\times(N_1)_{60}+45]^2} - \frac{1}{200} \qquad (2)$$

For earthquake magnitudes different from 7.5, a scaling factor termed MSF needs to be applied to CRR. Thus, factor of safety (FS) is equal to:

$$FS = CRR/CSR = (CRR_{7.5} \times MSF)/CSR \qquad (3)$$

Eurocode adopts Ambraseys (1988) scaling factors (Table 1 below) which is less conservative according to Youd et al. (2001). That would be the reason why Eurocode implies a safety factor of 1.25.

Table 1. Magnitude Scaling Factors by Eurocode

M_w	MSF
5.5	2.86
6.0	2.20
6.5	1.69
7.0	1.30
7.5	1.00
8.0	0.67

In the original development, Seed et al. (1985) noted an apparent increase of CRR with increased fines content F_c(%). Based on the empirical data available, they developed CRR curves for various fines content. In this paper, recommendation by Youd et al. (2001) is used for correction of $(N_1)_{60}$ to an equivalent clean sand value:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \qquad (4)$$

Where α and β are coefficients determined from the following relationships:

F_c(%)	α	β
0 ~ 5	0	1
5 ~ 35	$\exp[1.76-(190/F_c^2)]$	$0.99+(F_c^{1.5}/1000)$
35 ~	5	1.2

Japan Road Association provides similar approach for assessment of soil liquefaction as follows:

$$F_L = R/L \qquad (5)$$

$$R\ (or\ CRR) = c_W R_L \qquad (6)$$

$$L\ (or\ CSR) = (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (7)$$

$$r_d = 1 - 0.015x$$

$$R_L = 0.0882\sqrt{N_a/1.7} \text{ if } N_a < 14 \qquad (8a)$$

$$R_L = 0.0882\sqrt{N_a/1.7} + 1.6\times10^{-6}(N_a-14)^{4.5} \text{ if } N_a \geq 14 \qquad (8b)$$

For sandy soil:

$$N_a = c_1 N_1 + c_2 \qquad (9)$$

$$N_1 = 170N/(\sigma'_{vo}+70) \qquad (10)$$

F_c(%)	c_1	c_2
0 ~ 10	1	0
10 ~ 60	$(F_c+40)/50$	$(F_c-10)/18$
60 ~	$F_c/20-1$	

For gravelly soil: $N_a = \{1-0.36log_{10}(D_{50}/2)\}N_1$ (11)

where D_{50} is mean grain size or diameter through which 50% of the soil passes, in mm.

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3. IMPACT OF SOIL IMPROVEMENT

Many ground improvement techniques have been developed worldwide to mitigate liquefaction risk. Among them vibro compaction and replacement are widely used due to their effectiveness (Yasuda et al. 2012, Priebe 1998). While the former densifies non-cohesive soil by vibrations and improves it thereby directly, the latter uses coarse grained backfill material to build well compacted load bearing columns to enhance non compactible cohesive soil (Priebe 1998).

In term of design, there is no clear guidance in Eurocode regarding vibro replacement to prevent liquefaction even though Priebe's method is commonly used in Europe and in this study for comparison. Priebe (1998) proposed reduction factor α to reduce the CSR created by an earthquake with consideration of vibro replacement:

$$\alpha = \frac{tan^2(45^o - \varphi_C/2)\times(1-A_C/A)}{A_C/A + tan^2(45^o-\varphi_C/2)\times(1-A_C/A)} \quad (12)$$

where,

A is attributable area within the compaction grid;

A_C is cross section of stone columns;

φ_C is friction angle of column material.

In contrary to Priebe, The Japanese Geotechnical Society (2013) placed emphasis on CRR through SPT of improved soil. Normalized SPT N_1 after SI is estimated based on relative density D_{r1} (Eq. 13). The value of D_{r1} can be determined using Eq. 14 to Eq. 21.

$$N_1 = \left\{\left[\left(\frac{D_{r1}}{21}\right)^2 - \frac{\Delta N_f}{1.7}\right] \times (70 + \sigma'_{vo})\right\}/100 \quad (13)$$

$$D_{r1} = 100 \times (e_{max} - e_1)/(e_{max} - e_{min}) \quad (14)$$

$$e_1 = e_0 - a_s R_C (1 + e_0) \quad (15)$$

$$a_s = A_C/A \quad (16)$$

$$e_{max} = 0.02F_c + 1 \quad (17)$$

$$e_{min} = 0.008F_c + 0.6 \quad (18)$$

$$R_c = 1.05 - 0.46 log F_c \quad (19)$$

$$e_0 = e_{max} - (D_{r0}/100)(e_{max} - e_{min}) \quad (20)$$

$$D_{r0} = 21\sqrt{\frac{N}{0.7+\sigma'_{vo}/100} + \frac{\Delta N_f}{1.7}} \quad (21)$$

Where ΔN_f can be obtained using following relationships,

F_c(%)	ΔN_f
0 ~ 5	0
5 ~ 10	$1.2(F_c - 5)$
10 ~ 20	$6 + 0.2(F_c - 10)$
20 ~	$8 + 0.1(F_c - 20)$

and e_0 and D_{r0} are void ratio and relative density of original soil, respectively.

4. EXAMPLE AND COMPARISON

The following case is used for illustration and comparison. A 2m thick liquefiable silty sand layer is overlaid by 2m thick fat clay and 4m thick embankment on top. Underground water level is 0.5m below existing ground. Input soil parameters are shown in Table 2 below.

Table 2. Input Soil Parameters

Top EL	Bottom EL	Soil	SPT-N_0	Bulk weight γ_t [kN/m^3]	Fines Content [%]	σ_v [kN/m^2]	σ'_v [kN/m^2]
4	0	Embankment		19		76	
0	-1	Fat Clay	1	15	99	83.50	83.50
-1	-2	Fat Clay	3	15	99	98.50	88.50
-2	-3	Silty Sand	4	19	15	115.50	95.50
-3	-4	Silty Sand	4	19	15	134.50	104.50

Regarding earthquake, the design is based on a maximum ground acceleration of $a_{max} = 0.35g$ which corresponds to a magnitude of $M_w = 6.5$ according Priebe (1998). Our study shows that the silty sand would be liquefied without SI (Table 3). Sand compaction piles (SCP) and/or stone columns (SC) are thus proposed. For fair comparison, 0.7m diameter SCP/SC arranged in 1.5m x 1.5m square grid are adopted in both cases. Regarding SCP, it is required to achieve $(N_1)_{60} \approx 24$ after SI. In case of SC, $\varphi_C = [20 \times (N_1)_{60}]^{0.5} + 20 = 41.9^0$ is adopted to ensure equivalent $(N_1)_{60}$ of soil with SCP according to Hatanaka and Uchida (1996). Table 3 below summarizes our analysis result for the studied case.

Table 3. Comparison of Analysis Results

Soil	Code	CSR	CRR	FS	Efficiency
Without SI	EU*	0.260	0.143	0.550	2.031
With SI		0.128	0.143	1.117	
Without SI	JP	0.400	0.190	0.475	2.542
With SI		0.400	0.483	1.208	

5. CONCLUSION

First, factor of safety according to Japanese standard and Eurocode* are similar and comparable. Second, in cases without SI, design following Eurocode* results in slightly greater safety factor compared to Japanese code (0.550 vs. 0.475), possibly due to higher MSF being adopted as discussed in Section 2 above. With SI, the result is opposite (1.117 vs. 1.208), and more importantly FS of 1.117 under Eurocode* is lower than recommended value of 1.25. Finally, employing Japanese standard for liquefaction prevention design would be more efficient (2.542 vs. 2.031), thus more beneficial than using Eurocode*.

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