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TGS Geotechnical Lecture:

PARTIAL IMPROVEMENT SCHEME FOR LIQUEFACTION MITIGATION

Hsii-Sheng Hsieh [1*], Tin-Mei Lin [2], and Ting-Ling Tseng [2]

ABSTRACT

Liquefaction under seismic loading is an important design issue that has to be carefully dealt with when foundation is sitting on loose or medium dense sand. Loss of bearing capacity or excessive settlement as a result of liquefaction can be detrimental to the integrity of structure even for high rise buildings with relatively rigid raft foundation. Though local seismic design code allows the use of reduction factors on bearing capacity and modulus of subgrade reaction to incorporate the effect of liquefaction in foundation design, it is sometime required to implement certain improvement schemes to eliminate the liquefaction potential if the subgrade is in a very loose or loose state. In urban area where noise and vibration are of main construction concerns, jet grouting and deep mixing that create circular improvement piles are perhaps the only viable schemes for liquefaction mitigation. The liquefiable subgrade is often partially improved as a result of budget constraint, and the improvement ratio generally falls in the range of 10% to 20%. MASW technique is used to measure the shear wave velocity of the improved ground, and it is required that the shear wave velocity of the improved ground exceed a threshold value of 215 m/sec to eliminate the liquefaction potential. Design example is presented in this paper to delineate the logic in implementing the partial improvement scheme for liquefaction mitigation.

Key words: Liquefaction, foundation design, ground improvement, MASW.

1. INTRODUCTION

Taiwan is situated in a seismic active zone, structures often experienced earthquakes of significant magnitude. One foundation issue as a result of earthquake is liquefaction, as cyclic seismic loading may weaken the shear strength of loose material to a state that it offers little or no support to the foundation sitting on top of it.

In urban area, relatively rigid mat or raft foundation is often used as the foundation system for high rise buildings, but these rigid foundation systems are not immune to the detrimental effects of liquefaction if the subgrade has the potential to liquefy under seismic loading. As stipulated in local seismic design code (MOI 2022), the peak ground acceleration (A) adopted for liquefaction analysis is in the vicinity of 240 gals, which is a typical design number for cities in Taiwan with loose or medium dense sandy deposits. No matter which liquefaction analysis model is used, either it is NCEER (Youd and Idriss 2001) or HBF method (Hwang *et al*. 2012), the loose or medium dense sandy materials encountered in urban area is very likely to have a factor of safety against liquefaction (F_L) lower or much lower than 1.0. In current seismic or foundation design code (MOI 2022; MOI 2023), the liquefied soil is allowed to have a residual strength which is characterized by applying a reduction factor (D_E) on the pre-liquefied modulus of subgrade reaction and the pre-liquefied ultimate bearing capacity. However, the reduction factor is in general very low, resulting in low allowable bearing capacity that the raft foundation may not have adequate capacity to support the structure in the event of liquefaction. Other than resorting to expensive pile foundation, the designers have the alternative of using a certain ground improvement scheme to strengthen the subgrade and eliminate the detrimental effects of liquefaction on raft foundation.

However, in urban area, less expensive improvement schemes, such as dynamic compaction or sand compaction piles, are not allowed as these schemes generate noise and vibration that generally exceed acceptable limits. Jet grouting or deep mixing (Xanthakos *et al.* 1994) are the obvious alternative scheme, though at a much higher price tag. Since construction cost, especially the cost for foundation under ground level, is always a concern, the client's expectation is to keep the cost of jet grouting or deep mixing under a tolerable limit. As a result, the liquefiable subgrade under raft foundation is only partially improved, and the effect of partial improvement has to be carefully assessed. A balance has to be reached between the cost of improvement and the effect of liquefaction mitigation.

This paper presents a systematic approach in designing and verifying the effect of partial improvement. The effect of partial improvement on liquefaction mitigation is to be verified by performing on-site shear wave velocity measurement of the improved ground. In theory, the shear wave velocity of the improved ground is required to exceed a threshold value of about 215 m/sec to ensure that there is no liquefaction potential for subgrade under seismic loadings.

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[1*] Senior Engineer, Trinity Foundation Engineering Consultants, Co., Ltd., 3rd floor, 28, Lane 102, Section 1, An-Ho Road, Taipei, Taiwan (e-mail: drhsieh@tfec.com.tw).
[2] Engineer, Trinity Foundation Engineering Consultants, Co., Ltd., 3rd floor, 28, Lane 102, Section 1, An-Ho Road, Taipei, Taiwan.

2. PARTIAL IMPROVEMENT BY DEEP MIXING OR JET GROUTING

Using either jet grouting or deep mixing technique to create circular shape improvement piles, partial improvement design is characterized by three parameters, which are the improvement ratio (I_r), the unconfined compressive strength (q_u) and the depth or length of the improvement piles. As shown in Fig. 1, the improvement ratio I_r is defined as the area ratio occupied by the improvement piles. For example, if 0.8 m diameter improvement piles (d) in a square layout (Fig. 1(a)) with a center to center spacing (s) of 2 m are used, the improvement ratio I_r is 12.5%. The unconfined compressive strength of the improved pile is obtained by laboratory testing on core samples from the completed piles. As a general guide, a q_u value of 2 MPa is often specified in the design, though in-situ q_u may exceed 10 MPa when cement grout is well mixed with sandy materials. The third design parameter is the improvement depth. It is straightforward that the improvement piles should cover the liquefiable soil strata underneath the foundation level, or be extended from the foundation level to a suitable depth where the reduced bearing capacity is adequate to support the structural loading.

Partial improvement of the ground is often used in deep excavation project to strengthen the ground on the passive side, that leads to better overall excavation stability and less deformation of the retaining wall. Partial improvement underneath the foundation level also increases the bearing capacity and reduce the differential settlement of the raft foundation. These beneficial effects on excavation and foundation design can be maximized if the ground is fully improved with an improvement ratio of 100%. However, with a limited construction budget, the optimal improvement ratio generally falls within the range of 12.5% to 19.6% (1 m diameter piles at 2 m center to center spacing). Table 1 shows the relationship between pile diameter, spacing and improvement ratio for general application of partial improvement. As for now, the same partial improvement scheme for deep excavation and foundation can also be used for liquefaction mitigation, that makes the partial improvement more cost effective and more financially appealing to developers of construction projects.

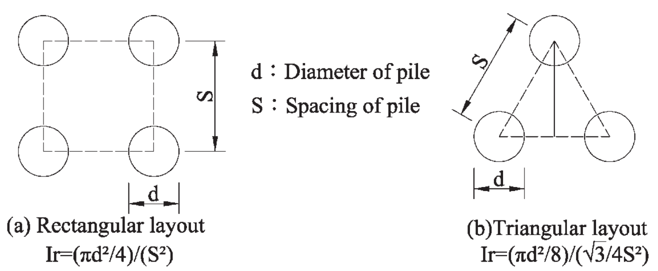

Fig. 1 Typical layout of improvement piles

Table 1 Improvement ratio of various layouts

Dimeter of improvement pile (m)	Center to center spacing of pile (m)	Rectangular layout Improvement ratio	Triangular layout Improvement ratio
0.6	2.2	5.8 %	6.7 %
	2.0	7.1 %	8.2 %
	1.5	12.6 %	14.5 %
0.8	2.2	10.4 %	12.0 %
	2.0	12.6 %	14.5 %
	1.5	22.3 %	25.8 %
1.0	2.2	16.2 %	18.7 %
	2.0	19.6 %	22.7 %
	1.5	34.9 %	40.3 %

3. ASSESSMENT OF LIQUEFACTION POTENTIAL

Liquefaction potential of in-situ ground can be evaluated by well-developed methods using Standard Penetration Test N value (SPT N), cone penetration resistance (q_c), or shear wave velocity (V_s) as key parameters (Youd and Idriss 2001). Almost all site investigations carried out on-site SPTs, and therefore N values are readily available as the basic parameter for foundation, excavation and liquefaction analyses. As a result, geotechnical engineers mainly focus on using SPT N values in conjunction with HBF (Hwang *et al.* 2012), NCEER (Youd and Idriss 2001) or Architectural Institute of Japan (AIJ 2001) methods to assess the liquefaction potential of a project site. The current seismic and foundation design codes require that at least two seismic levels, the small/medium earthquake and design earthquake, be considered in evaluating the liquefaction potential of a specific site. The small/medium earthquake corresponds to a return period of 30-year, and the associated peak ground acceleration (A) for a typical site in urban area is generally in the vicinity of 70 gals. On the other hand, the design earthquake corresponds to a return period of 475-year, and the associated peak ground acceleration (A) for a typical site in urban area is generally in the vicinity of 240 gals. The exact value of peak ground acceleration for seismic design is to follow equations outlined in seismic design code, the 70 gals and 240 gals mentioned above are just typical values.

HBF, NCEER and AIJ methods provides slightly different factor of safety against liquefaction (F_L), but with a typical peak ground acceleration of 240 gals, all these methods predict high possibility of liquefaction (F_L < 1.0) for loose to medium dense sandy materials in urban area. That is to say, under current design code, almost all building construction projects have to deal with liquefaction problem if loose or medium dense sandy materials are found underneath foundation level.

If partial improvement is adopted as a counter measure against the possible liquefaction, the shear wave velocity approach (Andrus and Stokoe 2000) has to be used instead to verify the effectiveness of partial improvement. This approach uses shear wave velocity as the main parameter in evaluating the liquefaction potential of the ground. Based upon field data (Andrus and Stokoe 2000), if the normalized shear wave velocity (V_{s1}) of the sandy material exceeds a threshold value of about 215 m/sec, the sandy material is unlikely to liquefy under almost any seismic events. Another study that incorporated Taiwan local seismic events (Hwang *et al.* 2005) suggested that the threshold shear wave velocity with extremely low possibility of liquefaction is around 230 m/sec, which is a slightly conservative value than the 215 m/sec suggested by Andrus and Stokoe. The goal of partial improvement design is to create a composite material with a combined shear wave velocity higher than the threshold value, either it is 215 m/sec or 230 m/sec, to eliminate altogether the possibility of liquefaction underneath the foundation level.

To be precise, the threshold shear wave velocities mentioned above were normalized or corrected against effective overburden pressure to 1 atm (100 KPa) via the following equation:

$$V_{s1} = V_s \times (P_a / \sigma'_{v0})^{0.25} \tag{1}$$

where P_a is one atmospheric pressure (100 KPa); and σ'_{v0} is the effective overburden pressure.

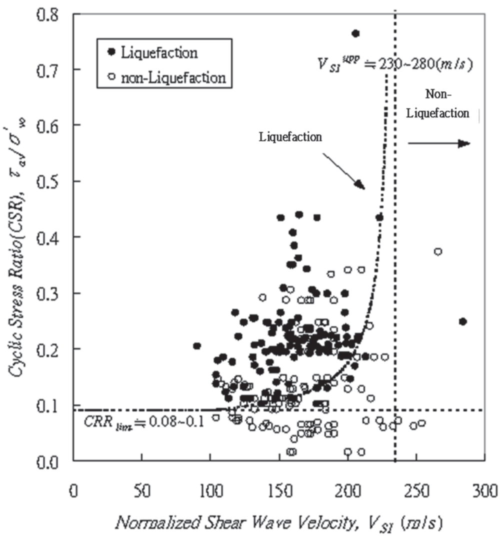

Fig. 2 Relationship between V_s and liquefaction events (Hwang *et al.* 2005)

It has to be pointed out that often-used SPT N or cone penetration resistance (q_c) approach is not suitable for evaluating the liquefaction potential of a partially improved ground as these more conventional approaches are unable to assess the combined liquefaction resistance of a composite ground.

4. SHEAR WAVE VELOCITY OF THE IN-SITU GROUND

The shear wave velocity of the in-situ untreated ground is now an important parameter for liquefaction resistance design if a partial improvement scheme is to be adopted. For in-situ measurement of the shear wave velocity, various methods such as MASW (Multichannel Analysis of Surface Waves) technique (Park *et al.* 1999), seismic cross-hole survey, seismic down-hole survey, or suspension logging are available. However, it has to be emphasized that MASW is perhaps the only suitable method for shear wave velocity measurement of the improved composite ground, as cross-hole/down-hole prospecting or suspension logging only provides V_s value at specific points or at sections with limited intervals. On the other hand, if shear wave velocity measurement is appropriately conducted by using MASW technique, the measured V_s represents the combined dynamic response of the improvement piles and the untreated soil sandwiched in between the improvement piles.

For most site investigation projects, in-situ shear wave velocity measurement is rarely carried out for various reasons such as budget constraints or other technical issues. Without the field measurement data, the geotechnical engineers are still required to estimate the average shear wave velocity of the site ground within a depth of 30 m (V_{s30}) for all building construction projects. This V_{s30} value is important for seismic design as it is the parameter used to determine if the project site is classified as a soft site (V_{s30} < 180 m/sec), a medium site (180 m/sec ≤ V_{s30} < 270 m/sec), or a hard site (V_{s30} ≥ 270 m/sec). On most occasions when field data are not available, the shear wave velocity of clayey or sandy materials can be estimated via the use of empirical equations as stipulated in the seismic design code (MOI 2022). For sandy material, an empirical equation in the following form is used:

$$V_s = 80 \times N^{1/3};\ 1 \le N \le 50 \tag{2}$$

where N is the SPT blow count of the sandy material, which is not to exceed 50 even if for very dense materials.

For a few site investigations with field measurement of shear wave velocity, it appears that there are obvious discrepancies between the field data and the empirical values (Table 2), but the empirical equation nonetheless serves as an adequate approximation from a design point of view. The shear wave velocity of sandy material is supposed to increase with its denseness or SPT N value, and Eq. (2) does provide a V_s prediction that increases with N value.

Table 2 Comparison of field measurement and empirical value of V_s for silty sands

Case No	Location	N value	Measured V_s (m/sec)	$V_s = 80\ N^{1/3}$ (m/sec)
1	Yunlin	7 ~ 40 (18)	206.6	217.5
2	Tainan	5 ~ 38 (19)	201.9	207.5
3	Tainan	3 ~ 31 (9)	206.5	166.4
4	Kaohsiung	4 ~ 17 (11)	197.8	177.0
5	Kaohsiung	8 ~ 22 (13)	190.8	188.1

5. LIQUEFACTION MITIGATION DESIGN

The design concept of using partial improvement for liquefaction mitigation is to increase the shear wave velocity of the improved ground to a level above the threshold value of 215 m/sec or 230 m/sec. A simple design approach based upon fundamental soil dynamics in conjunction with field experience is outlined in this section. Though it is theoretical possible to design a partial improvement scheme to eliminate liquefaction potential, the effect of partial improvement has to be verified on site by the application of MASW technique.

As depicted in soil dynamics textbook (Das 1983), shear wave velocity of soil (V_s) is function of shear modulus (G) and soil density (ρ), and it is written in the following form:

$$V_s = \sqrt{G/\rho} \tag{3}$$

Another equation relates the Young's modulus of soil (E) to shear modulus and Poisson's ratio (μ) is as follow:

$$G = E / \left(2(1+\mu)\right) \tag{4}$$

Combining Eqs. (3) and (4), shear wave velocity of the ground is proportional to the square root of Young's modulus, *i.e.,*

$$V_s \propto \sqrt{E} \tag{5}$$

The next step for the partial improvement design is to estimate the equivalent Young's modulus (E^*) of the improved ground. A major assumption was made here that the equivalent Young's modulus can be reasonably calculated via the weighted approach in the following form:

$$E^* = E \times (1 - I_r) + E_P \times I_r \tag{6}$$

where E_P is the Young's modulus of the improvement pile, and E_P can be obtained by conducting unconfined compression test on core samples taken from the improvement piles. On the design stage, there is yet to have core samples for testing, and empirical equation is used instead as a preliminary estimate of the E_P value. One often used empirical equation (Fang *et al.* 2004) is in the following form:

$$E_P = 100 \sim 300 \times q_u \tag{7}$$

where q_u is the unconfined compressive strength of the improvement pile, which is acquired by conducting unconfined compression test on core samples from the improvement pile.

With a typical q_u value of 2 MPa, the Young's modulus of the improvement pile (E_P) falls in the range of 200 ~ 600 MPa. These E_P values are compatible with the numbers listed in current foundation design code (MOI 2023), that allows the design engineers to use Eq. (7) with confidence. It has to be noted that the field q_u is perhaps much higher than the typical design value of 2 MPa when cement slurry is well-mixed with the in-situ sandy soil under good workmanship. However, using a design strength of 2 MPa is regarded as a conservative and prudent approach at this early theory development stage.

As for the Young's modulus of in-situ untreated sandy soil (E), one can resort to another empirical equation that relates E to the blow count N of that specific sandy layer. Though there are several empirical equations listed in one of the foundation textbooks (Bowles 1996) for the designer to choose from, a more straightforward relationship between E (MPa) and N is in the following form:

$$E = 2 \sim 3 \times N \tag{8}$$

Admittedly, Eq. (8) is just a rough estimate on the Young's modulus of typical sandy materials, but Eq. (8) at least captures the essence of Young's modulus as E is intuitively proportional to the denseness of a sandy material as shown in Eq. (8).

With the above relationships established, the designer can now devise a partial improvement scheme that increases the shear wave velocity of the improved ground to a level of no liquefaction. For example, if a loose sandy ground with N value equals to 10 is to be partially improved, the designer can first estimate the shear wave velocity and Young's modulus of the original ground via Eqs. (2) and (8), which are $V_s = 80 \times 10^{1/3} = 172$ m/sec and $E = 2.5 \times 10 = 25$ MPa, respectively. If the designer selects a typical improvement ratio of 12.5% together with an unconfined compressive strength of 2 MPa as the improvement scheme, then the Young's modulus of the improvement pile is $E_P = 200 \times 2 = 400$ MPa, and the weighted average Young's modulus of the improved ground is $E^* = 25 \times (1 - 0.125) + 400 \times 0.125 = 53.125$ MPa based upon Eq. (6), which is an increment 2.125 times (53.125/25 = 2.125) of the original ground. Further referring to Eq. (5), shear wave velocity of the improved ground is now increased by a factor of 1.46 ($\sqrt{2.125} = 1.46$), and that in turn leads to a composite shear wave velocity of about 250 m/sec for the improved ground. Assuming the effective overburden correction factor is about 1.0, the corrected shear wave velocity of the improved ground is higher than the threshold value of either 215 m/sec or 230 m/sec, and it is very likely that the improved ground is not going to liquefy under the design earthquake loading.

As a final design step, on-site MASW prospecting has to be performed following the completion of improvement piles to ascertain that shear wave velocity of the improved ground indeed exceeds the threshold values. If not, additional improvement piles can be installed as a contingency plan to further increase the composite shear wave velocity.

6. A PARTIAL IMPROVEMENT CASE

This section presents a design/construction/verification case that adopted partial improvement scheme to eliminate liquefaction potential altogether of loose materials encountered at shallow depth of a project site.

A 40,000-seat stadium that occupies a site area of 189,000 m^2 was built in Kaohsiung City (Jou *et al*. 2010) in 2009. Site investigation revealed that the subsurface comprises mainly of loose to medium dense silty sand layers to a depth of 16 m (Table 3). Main structure of the stadium was sitting on a pile/raft foundation system (Fig. 3) that uses piles together with raft foundation to share the structural weight. Piles adopted in this project are pre-drilled PC piles 0.8 m in diameter and 28 m in depth. Pile loading test results showed that the ultimate capacity of a single pile can reach up to 6,000 KN, and side friction contributes about 90% of the ultimate capacity.

Since this stadium is for public use, both design and maximum credible earthquakes had to be considered for liquefaction potential analysis. As required by the seismic design code, the peak ground acceleration for design and maximum credible earthquakes are 330 gals and 400 gals, respectively. With seismic loadings of these magnitudes, the loose to medium dense silty sand layers within a depth of 16 m are very likely to liquefy, resulting in significant loss of bearing capacity of the raft foundation. The capacity of pre-drilled PC pile relies heavily on the side friction as revealed by the pile loading test, and liquefaction of the loose sandy materials would seriously reduce the capacity of the pre-drilled PC pile. In short, though the stadium structure is sitting on

Table 3 Ground condition of Kaohsiung site

Layer	Depth (m)	Soil Type	Average SPT-N	γ_t (kN/m^3)	s_u (kN/m^2)	$\bar{c}$ (kN/m^2)	$\bar{\phi}$ (deg.)	c_c	c_r
I	0.0 ~ 8.0	SM	10	20.0	–	0	30	–	–
II	8.0 ~ 16.0	SM	14	20.0	–	0	32	–	–
III	16.0 ~ 20.0	ML	12	19.5	–	0	31	–	–
IV	20.0 ~2 4.0	ML/CL	9	19.5	40	0	30	0.238	0.026
V	24.0 ~ 30.0	CL/ML	9	19.8	50	0	30	0.214	0.027
VI	30.0 ~ 35.0				60	0	30		
VII	35.0 ~ 40.0				70	0	30		
VIII	40.0 ~ 46.0	CL/ML	13	20.1	80	0	31	0.185	0.021
IX	> 46.0	ROCK	> 50	22.8	–	0	–	–	–

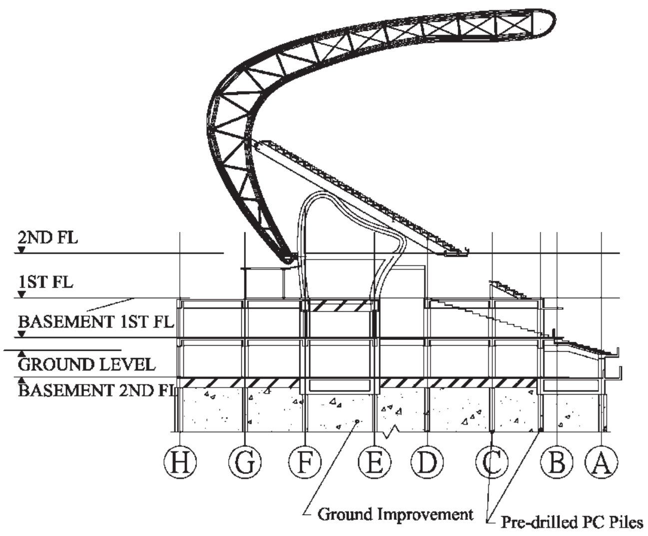

Fig. 3 Typical foundation profile of the stadium

a relatively rigid pile/raft foundation, the stadium is not immune from the effect of liquefaction. This conclusion is perhaps against common sense that foundation with piles should be able to counter detrimental effects from seismic loadings, but in the context of seismic and foundation design codes, liquefaction remains an issue that has to be carefully dealt with.

The final decision from the client was to eliminate altogether the liquefaction potential of the loose to medium dense sandy layer (GL.0 m ~ GL.–16 m). As a result, partial improvement scheme was implemented. Improvement piles 0.8 in diameter with center to center spacing of 2.25 m were installed from foundation level to a depth of 16 m (GL.–5 m ~ GL.–16 m), covering the entire loose sandy materials encountered within a depth of 16 m. The typical plan layout of improvement piles shown in Fig. 4 is equivalent to an improvement ratio of about 10%. A compressive strength of 2 MPa for the improvement pile was specified in the design. Based upon the equations listed in the previous sections, the followings are obtained: Young's modulus of the in-situ loose sand (Layer 1) $E = 2.5 \times N = 25$ MPa; Young's modulus of the improvement pile $E_p = 200 \times q_u = 400$ MPa; equivalent Young's modulus of the improved ground $E^* = 25 \times (1 - 0.1) + 400 \times 0.1 = 62.5$ MPa; shear wave velocity of the in-situ ground $V_s = 80 \times 10^{1/3} = 172$ m/sec; equivalent shear wave velocity of the improved ground $V_s^* = 172 \times \sqrt{62.5/25} = 272$ m/sec. If corrected with the effective overburden pressure at mid-level of the improved loose sandy layer (GL.–6.5 m), the corrected shear wave velocity of the improved ground is about $V^*_{s1} = 272 \times (100/115)^{0.25} = 263$ m/sec. The V^*_{s1} value is higher than the threshold value of 230 m/sec,

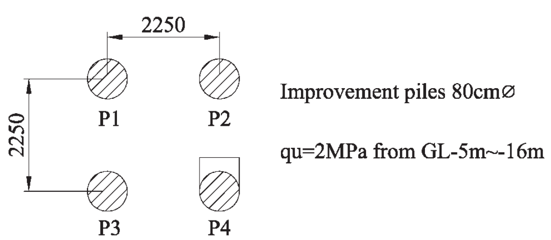

Fig. 4 Plan layout of improvement piles for the stadium project

implying a 10% improvement ratio for this project may suffice in eliminating liquefaction potential of the loose silty sand underneath the raft foundation to a depth of 8 m. As for Layer 2 materials (GL.–8 m ~ GL.–16 m) with an SPT N value of 14, the same calculations can be carried out to conclude that the V^*_{s1} value is also higher than 230 m/sec.

In the field, deep mixing technique was adopted to construct the improvement piles. In addition to proper site supervision on improvement pile construction, 14-day core samples were taken from designated piles for laboratory unconfined compression tests. It was found the unconfined compressive strength (q_u) of the core samples vary from 4 MPa to 32 MPa with an average value of 17 MPa. Though there was a discrepancy on q_u values, test results nevertheless showed that the improvement piles did have strength much higher than the design requirement of 2 MPa.

The effect of partial improvement was further verified by conducting on-site MASW prospecting. The MASW prospecting result was shown in Fig. 5, indicating that the shear wave velocity of the improved ground is in general higher than 300 m/sec with an average value of 350 m/sec. The field measured shear wave velocity is about 30% higher than the design value. The discrepancy between measured and design shear wave velocity can be attributed to several factors. An apparent reason is the strength of the improvement pile is much higher than the value used in the design, however, a ceiling value of 2 MPa is considered a conservative design parameter as the design approach of partial improvement to counter liquefaction is in an early stage of development. There is also uncertainty in estimating the shear wave velocity of the original ground, as Eq. (2) is only an empirical equation acquired by best-fitting of limited laboratory data. All considered, there is no surprise that the design shear wave velocity is lower than the measured value, and the authors consider it is a good design practice to have the key parameters fall on the conservative side.

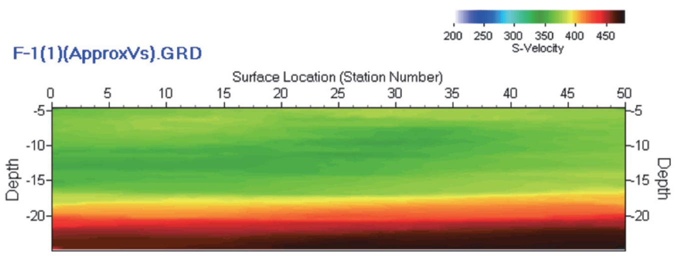

Fig. 5 MASW prospecting result of the stadium project

7. DISCUSSIONS

There is no denial that the partial improvement design scheme is in an early stage of development. The design framework was devised based on limited field observation (Martin *et. al*. 2004) in conjunction with basic soil dynamic and foundation engineering concepts. In theory, the partially improved ground is supposed to have a combined shear modulus much higher than that of the original liquefiable ground, which in turn reducing the shear strain of the improved ground during a seismic event and suppressing the possibility of liquefaction to a minimal. Young's moduli of soil and improvement piles were used in the design process, which were both estimated by empirical equations. However, these empirical equations were aimed to estimate the secant moduli for foundation design at finite strain level (E_{50} and E_{P50}), not exactly the moduli required for shear wave velocity calculation at very small strain or initial level (E_0 and E_{P0}). There could be a threefold difference between secant and initial Young's moduli, that may add further inaccuracy to the design scheme. On the other hand, this shortfall in estimating Young's moduli may not be serious as ratio of Young's moduli E^*/E was used in estimating the increase in shear wave velocity, the ratio E^*/E is less sensitive to how Young's moduli were calculated as long as they were compared at the same strain level.

Another issue regarding the overall effectiveness of partial improvement is about the liquefaction potential of untreated soils between the improvement piles. In theory, the denseness or SPT N value of untreated soil in between improvement piles remains about the same after partial improvement is carried out, therefore it is possible that these untreated soils could still liquefy under seismic loading. If SPT N approach is adopted, there is virtually no difference on the liquefaction potential of these untreated soil prior and after improvement, unless the ground is fully treated with a financially impossible improvement ratio of 100%. At this stage, information is rare regarding the behavior of the untreated soil under seismic loading. It is hypothesized that overall shear strain of the improved ground is reduced to a level that does not allow the development of liquefaction, and since the untreated soils are moving together with the improvement piles as a whole, there is no liquefaction in these materials as a result. A less technical precise description on this issue was once addressed by an expert on MASW prospecting (Xia 2010), who said" the shear wave has a low resolution to distinguish between improvement piles and untreated soil as it passes through the improved ground, as the wavelength of shear wave is long compared to the distance between improvement piles". Further verification on this issue is needed, either by numerical studies or laboratory simulations on shaking table or centrifuge device.

What is the minimal improvement ratio required to eliminate liquefaction potential? This is the main concern of the clients, as it is tied to the cost of improvement. In the field case (Martin *et al.* 2004), an improvement ratio of 7% is reported, which is equivalent to a layout of 0.6 m diameter improvement piles at a center to center spacing of 2 m. The partial improvement scheme for deep excavation projects often uses an improvement ratio of 12.5%, which is probably the optimal improvement ratio after years of field experiments, and 7% improvement ratio is below the confidence level of most design engineers even if it is just for the purpose of liquefaction mitigation. At this stage, it is suggested that an improvement ratio of at least 10% be adopted for liquefaction mitigation design, merely for peace of mind. Asides from improvement ratio, the clients also argue about the maximum strength of improvement pile that can be used in the design. Obviously, a high q_u value will lead to a smaller improvement ratio, meaning the number of improvement piles can be reduced, so as the total cost of improvement. If a higher q_u value is used instead of the often used 2 MPa, the improvement ratio may be dropped to a low value of 3% or an unrealistic value of 1%. For a 3% improvement ratio, the improvement piles are sparsely distributed (0.6 m diameter improvement piles 3 m apart). Though a 3% improvement may perhaps be still effective in mitigating liquefaction, it does exceed the mentally acceptable ceiling of most design engineers. As a result, the maximum q_u should have a ceiling value of only 2 MPa, no matter what number can actually be achieved in the field or claimed by the improvement contractor.

Though partial improvement is considered much less expensive than pile foundation to counter the detrimental effect of liquefaction, there is an alternative that may be even more cost effective. The authors observed in quite a few urban renewal projects that footings of demolished buildings were often strengthened with timber piles about 0.1 m (4-inch) in diameter and less than 3 m in length (Fig. 6). These timber piles were installed at least 5 decades ago when deep mixing or jet grouting techniques were rarely used. Another interesting field observation following the 1999 Chi-Chi earthquake is that the low-rise buildings supported by these short timber piles showed no signs of liquefaction induced damages. Once again, these observations were neither well documented or fully verified by numerical or full-scale model studies, but timber piles do have the potential of replacing partial improvement piles as an inexpensive alternative to eliminate liquefaction potential underneath the footings.

Fig. 6 Timber piles underneath old footings in loose sand

8. CONCLUSIONS

This paper serves as a succinct outline on the use of partial improvement scheme to minimize liquefaction potential. The essence of this scheme is to increase the shear wave velocity of the loose ground to exceed a threshold value that the liquefaction potential is a very unlikely event under severe seismic loadings. Threshold shear wave velocity, if corrected with respect to the effective overburden pressure, is about 215 m/sec or 230 m/sec. For typical loose to medium dense sandy materials, the shear wave velocity may have to be increased by 20% to 40% in order to reach the threshold value.

Three parameters are imperative to the design of partial improvement scheme, namely, the improvement ratio, improvement depth, and unconfined compressive strength of improvement pile. The authors' suggestion at this moment is to use an improvement ratio of at least 10% together with an unconfined compressive strength of 2 MPa as a basis for liquefaction mitigation design. The design shear wave velocity as a result may barely exceed the threshold value, while field measured shear wave velocity of the improved ground could be a lot higher than the design value, this is a favorable scenario as the primitive partial improvement scheme is on the conservative side that guarantees a non-liquefaction situation. Finally, the improvement depth is to cover the layers with liquefaction potential under seismic loading, but can be adjusted based upon engineering judgment of the design engineer.

On-site MASW prospecting is required to ensure that the shear wave velocity of the improved ground does exceed the threshold value. In case the measured shear wave velocity fails to meet the design requirement, additional improvement piles can be installed to further increase the shear stiffness of the site ground before the foundation of building is in service.

The partial improvement scheme is conjured up by the authors based upon limited field information. Though its effect is yet to be thoroughly verified by numerical simulations or laboratory model tests, partial improvement scheme is nevertheless a cost-effective plan for liquefaction mitigation. Further refinement on the design approach as well as related researches on issue like the use of timber piles are imperative, especially when liquefaction is now an inevitable design issue that has to be carefully dealt with.

NOTATIONS

A	peak ground acceleration
d	diameter of improvement pile (m)
D_E	reduction factor
E	Young's modulus of soil (MPa)
E_0	initial Young's modulus of soil (MPa)
E_{50}	secant Young's modulus of soil (MPa)
E_P	Young's modulus of improvement pile (MPa)
E_{P0}	initial Young's modulus of improvement pile (MPa)
E_{P50}	secant Young's modulus of improvement pile (MPa)
E^*	equivalent Young's modulus of the improved ground (MPa)
F_L	factor of safety against liquefaction
G	shear modulus of soil (MPa)
I_r	improvement ratio
N	blow counts of standard penetration test
P_a	atmospheric pressure (100 KPa) (MPa)
q_u	unconfined compressive strength of improvement pile (MPa)
s	center to center spacing of improvement piles (m)
V_s	shear wave velocity of soil (m/sec)
V_s^*	equivalent shear wave velocity of the improved ground (m/sec)
V_{s1}	shear wave velocity corrected to the effective overburden pressure (m/sec)
V_{s1}^*	equivalent shear wave velocity corrected to the effective overburden pressure (m/sec)
V_{s30}	average shear wave velocity of the top 30 m (m/sec)
z	depth below ground surface (m)
μ	Poisson's ratio
σ'_{v0}	effective overburden pressure (MPa)

REFERENCES

Andrus, R.D. and Stokoe, K.H. (2000). "Liquefaction resistance of soils from shear-wave velocity." *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, **126**(11), 1015-1025. https://doi.org/10.1061/(ASCE)1090-0241(2000)126:11(1015)

Architectural Institute of Japan (2001). *Design Guidelines for Foundation of Buildings*.

Bowles, J.E. (1996). *Foundation Analysis and Design*, 5th Ed., McGraw-Hill Book Companies, Inc., New York

Das, B.M. (1983). *Fundamentals of Soil Dynamics.* Elsevier Science Publishing, New York.

Fang, Y.S., Chain, K.F., Wang, D.R., Kao, C.C., and Chou, J. (2004). "Ground improvement with Super-Jet-Midi method." *Sino-Geotechnics,* **102**, 79-88 (in Chinese).

Hwang, J.H., Yang, C.W., and Chen, C.H. (2005). "Simplified methods for assessing liquefaction potential of soils by using hyperbolic cyclic resistance curves." *Sino-Geotechnics*, **103**, 53-64 (in Chinese).

Hwang, J.H., Chen, C.H., and Juang, C.H. (2012). "Calibrating the model uncertainty of the HBF simplified method for assessing liquefaction potential of soils." *Sino-Geotechnics,* **133**, 77-86 (in Chinese).

Jou, Y.W., Hsu, J.L., and Hsu, C.H. (2010). "Foundation excavation and ground improvement of the main stadium for the world games 2009 Kaohsiung." *Sino-Geotechnics*, **124**, 7-14 (in Chinese).

Martin, J.R., Olgun, C.G., Mitchell, J.K., and Durgunoglu, H.T. (2004). "High-modulus columns for liquefaction mitigation." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, **130**(6), 561-571. https://doi.org/10.1061/(ASCE)1090-0241(2004)130:6(561)

Ministry of Interior (2022). *Seismic Design Code for Buildings* (in Chinese).

Ministry of Interior (2023). *Design Code for Foundation of Buildings* (in Chinese).

Park, C.B., Miller, R.D., and Xia, J.H. (1999). "Multichannel analysis of surface waves." *Geophysics*, **64**, 800-808. https://doi.org/10.1190/1.1444590

Xanthakos, P.P., Abramson, L.W., and Bruce, D.A. (1994). *Ground Control and Improvement.* John Wiley and Sons, Inc., New York.

Xia, J.H. (2010). personal communication.

Youd, T.L. and Idriss, I.M. (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, **127**(10), 817-833. https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817)