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Shear wave velocity-based liquefaction evaluation in the great Wenchuan earthquake: a preliminary case study

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Abstract: The great Wenchuan earthquake (M_s = 8.0) in 2008 caused severe damage in the western part of the Chengdu Plain. Soil liquefaction was one of the major causes of damage in the plain areas, and proper evaluation of liquefaction potential is important in the definition of the seismic hazard facing a given region and post-earthquake reconstruction. In this paper, a simplified procedure is proposed for liquefaction assessment of sandy deposits using shear wave velocity (V_s), and soil liquefaction from the Banqiao School site was preliminarily investigated after the earthquake. Boreholes were made at the site and shear wave velocities were measured both by SASW and down-hole methods. Based on the in-situ soil information and V_s profiles, the liquefaction potential of this site was evaluated. The results are reasonably consistent with the actual field behavior observed after the earthquake, indicating that the proposed procedure is effective. The possible effects of gravel and fines contents on liquefaction of sandy soils were also briefly discussed.

Keywords: liquefaction; Wenchuan earthquake; shear wave velocity; spectral analysis of surface waves (SASW); hazard definition

1 Introduction

At 14:28 May 12 2008, a devastating earthquake of magnitude 8.0 on the Richter scale occurred in the Sichuan Province of China, causing severe loss both to human lives and the national economy. The epicenter was located at Wenchuan (31.0°N, 103.4°E) (Wang, 2008). One of the causes for the heavy damage was the widespread seismic liquefaction in the Chengdu Plain along the main fault rupture, which covers an area of about 200 km long and 100 km wide (Fig. 1). Mianzhu City is within this affected area and suffered severe damage due to soil liquefaction in several townships including Banqiao, Gongxing and Xinglong. Ground failure in the form of sand boils, lateral spreads and settlement, etc., is the common effect of liquefaction in soil deposits. Many earthquake reconnaissance works indicate that ground shaking is not the only cause of damage to structures/buildings and soil liquefaction could play an important role under certain circumstances (Yuan *et al.*, 2003). Identification and documentation of the observations on liquefied sites in an earthquake is essential to establishing empirical methods for evaluating liquefaction potential, and helps to better understand the seismic hazard and perform appropriate seismic designs for post-earthquake reconstruction (Chen *et al.*, 2008; Yuan and Sun, 2008).

In this paper, a simplified procedure using shear wave velocity measurements for evaluating the liquefaction potential of sandy soils is proposed, and a preliminary case study of Banqiao School is introduced. The authors conducted reconnaissance immediately following the earthquake, and phenomena such as sand boils, settlement and lateral spreading, etc., associated with liquefaction were observed in and around this school. In-situ soil profiles of the Banqiao School were obtained from two borings after the earthquake. Seismic wave measurements by SASW and down-hole methods were conducted to obtain data of shear wave velocities. Then, the simplified procedure was employed to evaluate the liquefaction potential of this site. The estimated results are reasonably consistent with the actual behavior observed after the earthquake, indicating that the proposed method is effective.

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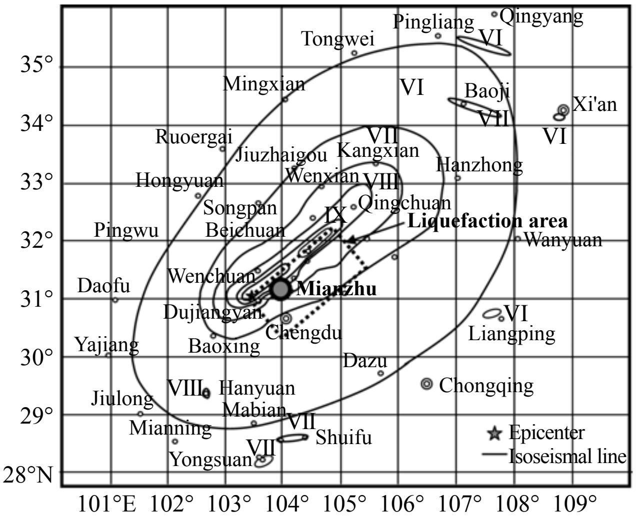


Fig. 1 Liquefaction area in the great Wenchuan earthquake (modified from Yuan, 2008)

2 Field investigation and geologic conditions in Banqiao School

Banqiao is a town in Mianzhu City, which is located approximately 25 km from the Beichuan-Yingxiu Fault rupture that caused the great Wenchuan event. The local water table is 2.5 m to 3.0 m. The ground shaking was severe and the maximum horizontal peak ground acceleration (PGA) is about 0.40 g (the geometric mean value) according to the strong motion observations (Li *et al*., 2008). Banqiao School is located in the central part of this town, along the Sheshui River (see Fig. 2). The main building was a four-story (16.9 m height) frame-structural building with a shallow foundation of 2.05 m below the ground surface. The entire building seemed not seriously damaged directly by the ground shaking as few cracks were observed in walls, columns and beams of the structure. Nevertheless, typical liquefaction evidence such as sand boils, ground fissures, settlement and lateral spreads were found in the schoolyard as well as some parts outside (e.g., the road along the school, the Banqiao Kindergarten at the east side, etc.). The differential settlement that occurred at the main building led to the tilt and knocking together of the main building and building A, and the lateral spreading caused the "pulling damage" on some buildings near the Sheshui River (i.e., building C and D in Fig. 2). Typical phenomena of liquefaction-induced ground failure at different positions in the schoolyard are shown in Fig. 3 (a)-(d).

The school is situated on a relatively flat (slope <1°) alluvial deposit of the Sheshui River. The ground consists of surface backfill soil (Q_4^{ml}) and Holocene alluvium (Q_4^{al+pl}), which overlies older sedimentary deposits. To investigate the local site condition in detail, two boreholes (BH-1 and BH-2) were made close to the building (see Fig. 2). Figure 4 shows the soil samples from BH-2 at different depths. Note that the sandy layer in the depth 5.2 m contains some fines (silt, clay or the mixture of both) and small-size gravel, while the loose gravel layer at 8.5 m contains some amount of medium or coarse sand. According to the boreholes, the soil deposit consists of 6–7 layers from the surface downward, including miscellaneous fill (thickness 0.20–0.70 m),

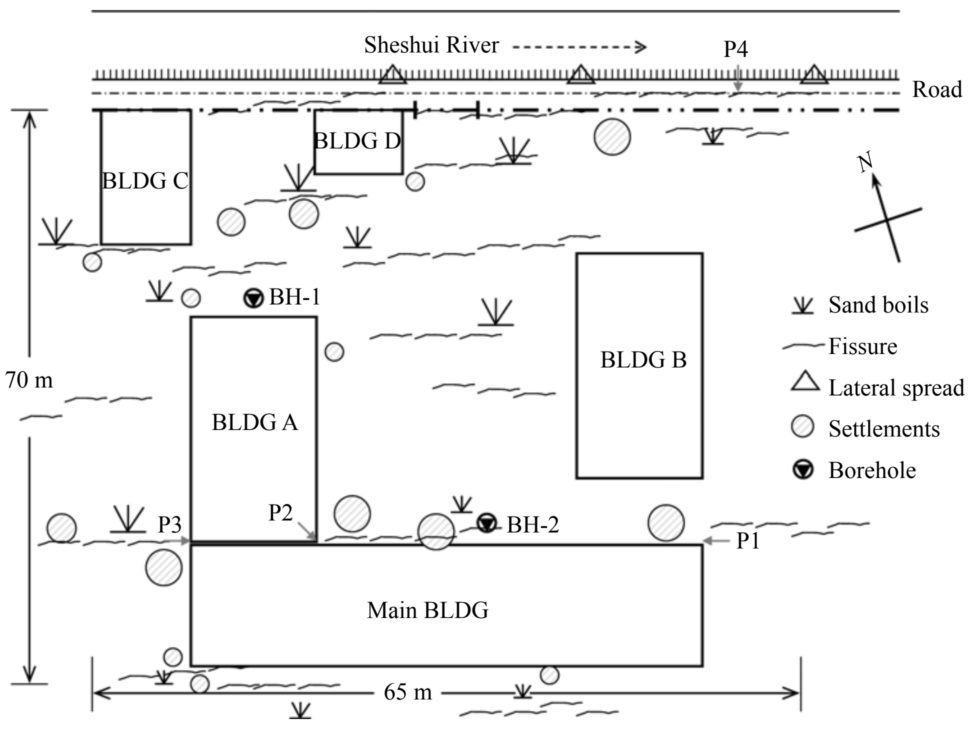
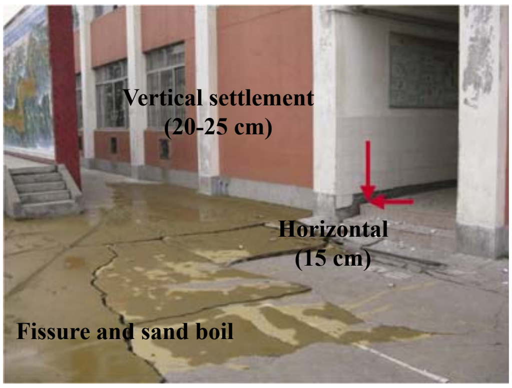


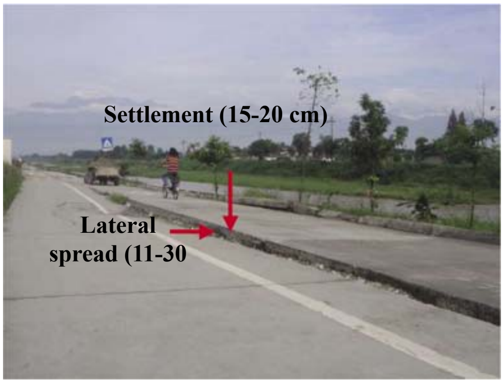
Fig. 2 Map of Banqiao School showing ground failure and boring locations

(a) Settlement of the east corner (P1)

(b) Tilt and knocking of two adjacent buildings (P2)

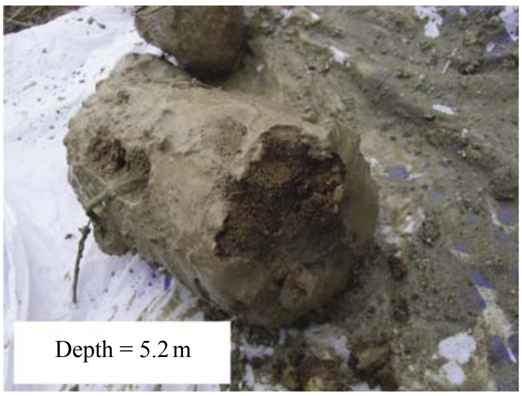


(c) Settlement of the west corner (P3)



(d) Settlement and lateral spreading of road (P4)

Fig. 3 Typical phenomena of liquefaction-induced ground failure



(a) Sand sample at depth = 5.2 m

(b) Gravel sample at depth = 8.5 m

Fig. 4 Typical samples at different depths (BH-2)

silty clay (thickness 0.80 m –1.3 m, containing 5%–10% sand), medium sand (thickness 0.30 m –1.0 m, containing 5% m –10% fines), clayey gravel (thickness 0.4 m –0.8 m, containing 30%–40% gravel and 5%–10% sand), medium sand (thickness 2.0 m –3.0 m, containing 5% –10 % fines and 5% –10% gravel), very loose gravel (thickness 0.3 m – 2.6 m, containing 10%–20% sand and 5% –10% fines), and moderate dense to dense gravel (containing 10%–20% sand and fines). By comparing the ejection of sand boils and sandy samples from the borehole, it seems that the liquefied layer should be the sandy layer at a depth of 3 m to 6 m or the upper part of the loose gravelly sandy layer at 5–8 m. Thus, more reliable evaluation based on field indexes is required to accurately identify the liquefaction potential of different layers at this site.

3 V_s-based simplified procedure for liquefaction evaluation

There exist a significant number of simplified procedures for evaluating soil liquefaction potential based on in-situ tests such as the SPT and CPT (Chen *et al.*, 2002; Samui, 2007). A shear wave velocity-based procedure is preferable as its implementation provides a more rapid measurement than the penetration tests if the Rayleigh wave method or seismic CPT are adopted (Andrus and Stokoe, 2000; Youd *et al.*, 2001). This is especially promising in view of the widespread gravel formation in the Chengdu Plain, where penetration tests are not available or reliable.

In previous studies, the authors (Chen *et al.*, 2005; Zhou and Chen, 2007) established one semi-empirical CRR-V_s correlation based on tremendous observations of cyclic liquefaction tests with initial liquefaction as a failure criterion (Marcuson, 1978) and the power relationship between small-strain shear modulus and confinement (Zhou and Chen, 2005), and this correlation under field conditions could be expressed as follows

$$CRR = r_c \frac{1}{P_a}\left[\frac{k_N \rho}{F(e_{\min})}\right]^{1/n} (V_{s1})^{2/n} \tag{1}$$

where r_c = a constant of multidirectional shaking (0.9–1.0); P_a = reference overburden stress (= 100 kPa); k_N = fitting value for a given failure cycle number N from cyclic triaxial test; n = power exponent in Hardin equation [i.e., $G_{\max} = AF(e)(\sigma'_m)^n$], usually n = 0.5 is adopted for sandy soils; $e_{\min}$ = minimum void ratio and $F(e)$ is void ratio function, $F(e) = 1/(0.3+0.7e^2)$; ρ = total mass density of the soil; V_{s1} = overburden stress-corrected shear wave velocity, which is expressed as:

$$V_{s1} = V_s C_V = V_s\left(\frac{P_a}{\sigma'_v}\right)^{0.25} \tag{2}$$

where V_s = field measured shear wave velocity; σ'_v = effective overburden stress at the depth in question; C_V = factor to correct in-situ measured velocity for σ'_v, and a maximum C_V value of 1.4 is applied at shallow depths.

The power-law relationship in Eq. (1) reveals that CRR will vary proportionally with $(V_s)^4$ in the statistical level, which was verified by comprehensive experimental investigations and the global (CSR, V_{s1}) database (Kayen *et al.*, 2004). For different types of sandy soils with or without fines content, the value of k_N is recommended in Table 1 for practical use (Zhou and Chen, 2007).

Based on the correlation in Eq. (1), a simplified procedure for liquefaction potential evaluation using shear wave velocity is developed as follows:

(1) Determine the earthquake-induced shear stress ratio, CSR, at a particular depth in a level soil deposit by (Seed and Idriss, 1971):

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d \tag{3}$$

where $a_{\max}$ = peak horizontal ground surface acceleration (PGA); g = acceleration of gravity; σ_v = total overburden stress at the depth in question; and r_d = shear stress reduction coefficient to adjust for the flexibility of the soil profile, and usually being determined by the following expressions (Liao and Whitman, 1986):

$$r_d = 1.0 - 0.00765z, \qquad z \le 9.15 \text{ m} \tag{4a}$$

$$r_d = 1.174 - 0.0267z, \qquad 9.15 \text{ m} < z \le 23 \text{ m} \tag{4b}$$

(2) Determine the liquefaction resistance (CRR) of soil at the same depth based on shear wave velocity and other parameters such as k_N, ρ and $e_{\min}$. Once the V_{s1} is calculated from Eq. (2) using in-situ measured V_s, the liquefaction resistance CRR can be readily obtained by using Eq. (1). Note that although it is an ideal way to obtain all required parameters via in-situ sampling and laboratory test, they could be estimated from the empirical relationships as the first approximation. For example, the average values of $e_{\min}$ are 0.65 for sands with fines content less than 20%, 0.75 for silty sands, and 0.95 for sandy silt (Tokimatsu and Uchida, 1990). And, k_N values could be adopted from Table 1.

Table 1 Value of k_N for different sandy soils 10^{-4} kPa$^{-0.5}$

Earthquake magnitude, M_w	Equivalent failure cycles, N	Clean sand $FC \le 5\%$	Silty sand $5\% < FC < 35\%$	Silty sand $FC \ge 35\%$
8-1/2	26	0.932	0.912	0.938
7-1/2	15	0.997	0.959	0.982
6-3/4	10	1.073	1.024	1.042
6	5	1.173	1.113	1.132
5-1/4	3	1.300	1.216	1.225

(3) Evaluate the liquefaction potential, i.e., the factor of safety against liquefaction, F_s, based on a comparison of the values obtained in Steps (1) and (2), that is

$$F_S = \frac{CRR}{CSR} \tag{5}$$

A soil layer will be predicted to be liquefied/liquefiable when $F_s \le 1$ and is not liquefied/nonliquefiable when $F_s > 1$. Figure 5 illustrates the flowchart of the proposed procedure.

4 Field wave testing and results

After all school buildings were removed in September 2008, two borings (BH-1 and BH-2) were made around the main building. Meanwhile, seismic wave tests were carried out to obtain the V_s profiles of this site. The following section is presented to describe the details of both the SASW and down-hole tests.

4.1 SASW testing

The spectral-analysis-of-surface-waves (SASW) method is a prevailing in situ method for determining shear-wave velocity (Brown *et al*., 2002). The impact source used in this study was a sledge-hammer or a drop weight depending on the receiver spacing. The recording device was a CRAS Analyzer interfaced with a portable computer. In the computer, a software package implements the standard signal analysis such as averaged cross spectral density function, transfer function and coherence function. Two geophones were used as receivers. Different receiver spacings were used at each test location. With these spacings, the soil profiles down to a depth of about 12.5 m could be sampled. To check the effect of horizontal variation in the soil deposits, data from both north-south and east-west directions were collected around the borehole. The test arrangement is illustrated in Figs. 6 (a) and (b).

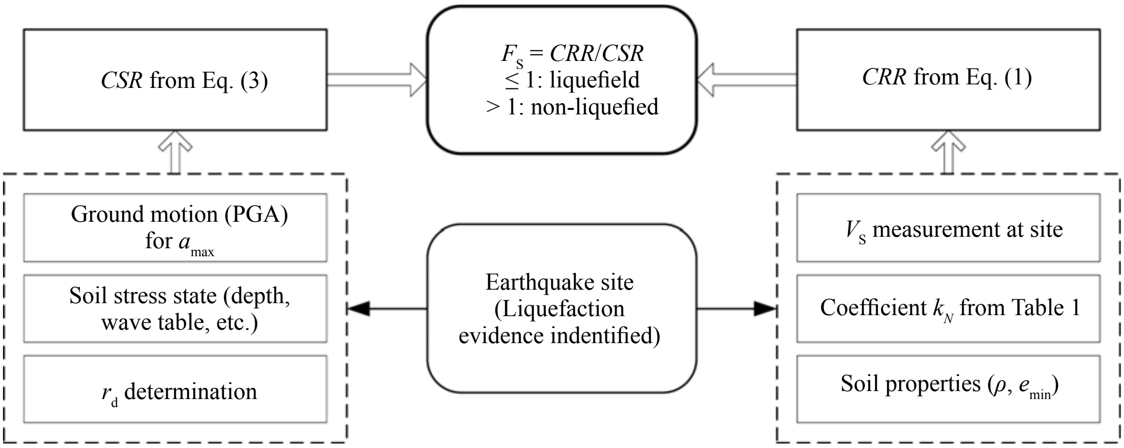
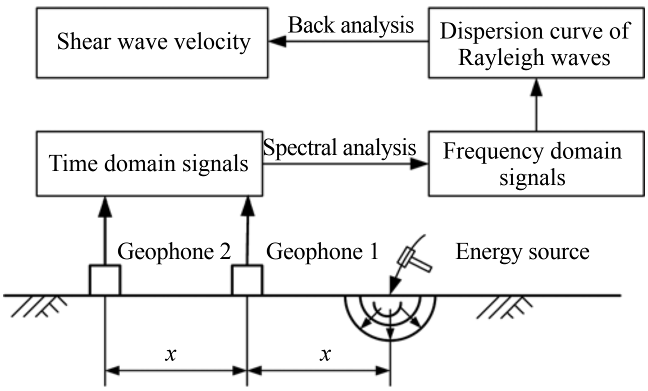


Fig. 5 Flowchart of liquefaction evaluation based on shear wave velocity



(a) Schematic of test arrangement

(b) Photo of field test

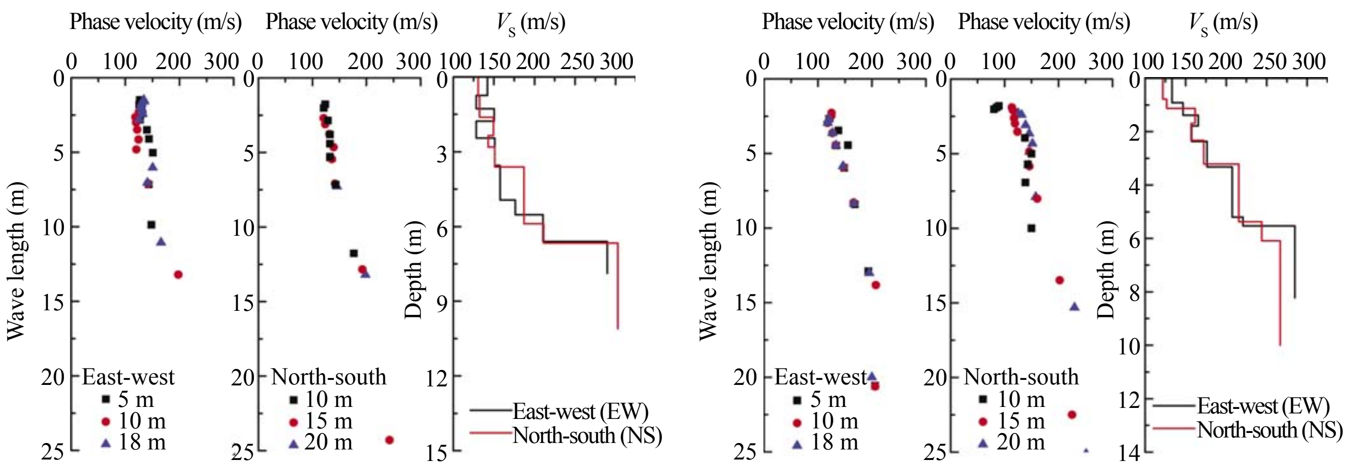
Fig. 6 Test arrangement of SASW testing

By analyzing the phase information of the transfer function at each frequency, a set of dispersion data were obtained for that receiver spacing. From the dispersion curve, the variation in shear wave velocity with depth was determined through a back-calculation process (Chen *et al*., 1992; Yuan and Nazarian, 1993). Two sets of raw dispersion data and the corresponding shear-wave velocity profiles are demonstrated in Fig. 7(a) and (b). As shown in Fig.7, the V_s profiles in both directions are very similar, indicating that the soil deposit is relatively uniform.

4.2 Down-hole testing

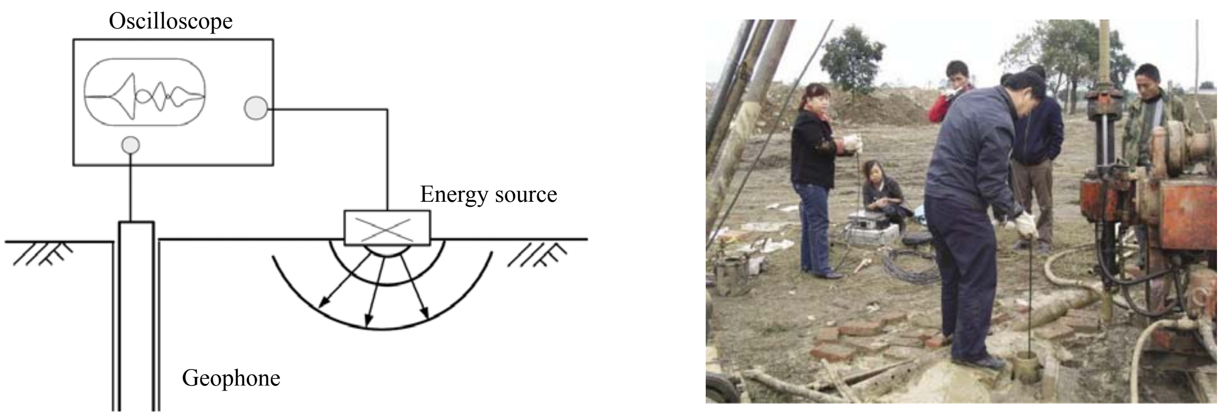
The seismic down-hole test consists of lowering a geophone to a specified depth in the borehole and clamping it to the casing. An energy source is placed at the surface near the borehole. Travel time of the body waves (S- and P-waves) between each geophone and the source are recorded, and then the maximum shear and compression wave velocities (i.e., V_s and V_c) of all soil layers are determined (Woods, 1994). In this test, low velocity layers were detected as long as geophone spacing was sufficiently close. Thus, the results of the down-hole test were treated as the reference value for the SASW test as introduced above. Fig. 8(a) shows the arrangement of the down-hole test; note that the impact source was located about 1.25 m away from the collar of each borehole. The recording device was an RSM-24FD system. A borehole geophone of wall-lock function was used as the receiver. Readings were taken at a constant depth interval of 0.5 m in each borehole.

In the present study, the difficulty of picking up the first arrival of shear waves was resolved by reversing the polarity of the source generating the wave pattern. In this manner, the shear wave patterns were distinguished from compression wave patterns. Then, the V_s profiles and soil information were plotted in Fig. 9(a) and (b) for BH-1 and BH-2, respectively. As shown in Fig. 9, the V_s profiles obtained from the down-hole test showed good



(a) BH-1 (b) BH-2

Fig. 7 Dispersion data and shear wave velocity profile from SASW tests



(a) Schematic of test arrangement (b) Photo of field test

Fig. 8 Test arrangement of down-hole test

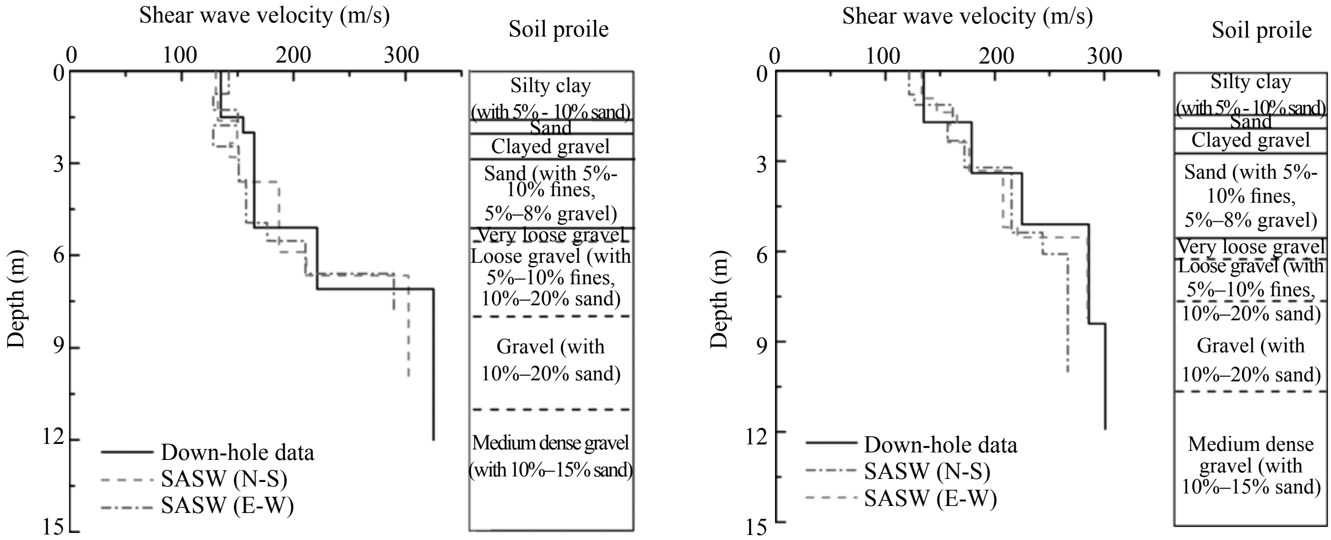
agreement with those from SASW test. In other words, the SASW testing and analysis carried out in this study was a reliable method for characterizing geophysical profiles of earthquake sites.

5 Liquefaction evaluations

To illustrate the identification of the liquefaction layer to better characterize the seismic hazard, the liquefaction potential at both boreholes were evaluated by the proposed simplified procedure. The soil profile and average values of V_s and V_{s1} are presented in Fig. 10. And, the *CRR* and *CSR* values were calculated assuming soil densities of 1.80 g/cm^3 above the water table and 1.95 g/cm^3 below the water table, and k_N = 0.959 (10^{-4} kPa$^{-0.5}$) in view of the fines content. As mentioned above, a geometric mean value of 0.40 g was assumed for a_{max}. The value of r_d at different depths was calculated by Eq. (4). Note that *CSR* values in Fig. 10 are the converted ones at M_w = 7.5, wherein an *MSF* = 0.86 is assumed according to Youd *et al*. (2001).

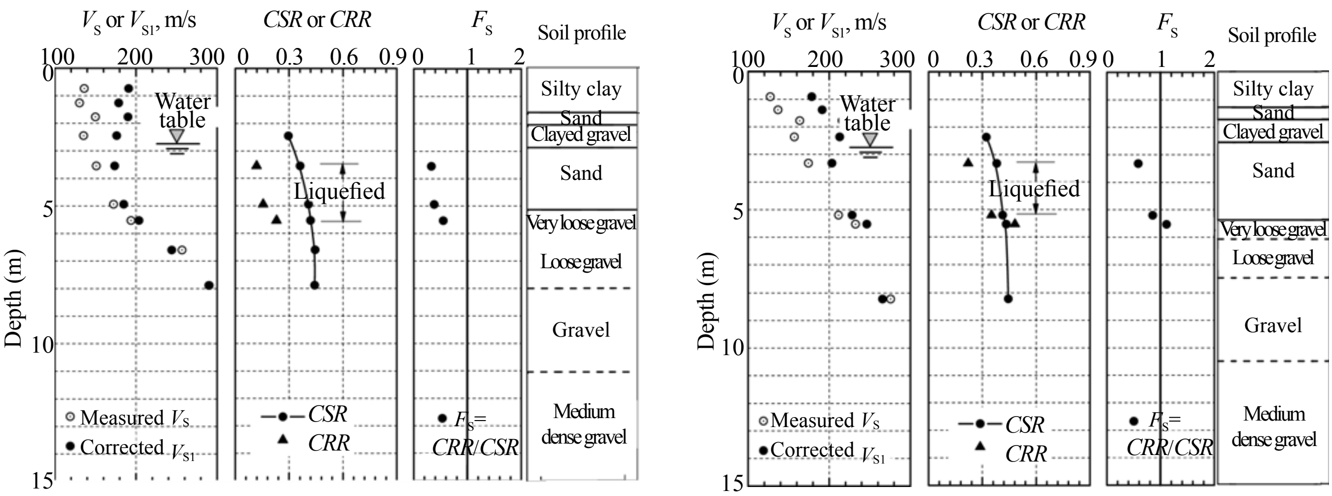
Values of F_S shown in Fig. 10(a) are less than 1 for the depths of 3.5–5.5 m in BH-1, where the liquefiable sandy layer exists. It is almost the same case for BH-2 in Fig. 10(b), except that the liquefied depth varies a bit. Thus, by the simplified V_s procedure, the layer predicted likely to liquefy, or the critical layer, lies between the depths of 3 and 6 m for the boreholes studied herein.

It is interesting to note that in Fig. 10(a), there is a narrow zone of gravelly soil underlying the sand layer, which is also predicted to liquefy. As mentioned above,



(a) BH-1 (b) BH-2

Fig. 9 V_s profiles from down-hole test



(a) BH-1 (b) BH-2

Fig. 10 Application of simplified procedure to Banqiao School

the evaluation is primarily based on the parameters of sandy soils, therefore predictions for the very loose gravel layer are not yet reliable. Previous studies (Hatanaka *et al.*, 1997) show that liquefaction behavior of gravelly soil is different from typical sandy soils to some extent. This is also the case for sandy soils with high fines (e.g., silt or clay) content (Gratchev *et al.*, 2006). Therefore, the present evaluation should be treated only as a preliminary judgment for non-typical sandy soil encountered at this site. Further research that accounts for the effects of gravel or fines (especially clay) contents on the liquefaction behavior of sandy soils is required to achieve a reliable evaluation. Moreover, the liquefaction susceptibility of gravelly soils should also be considered, since it might contribute to liquefaction damage in view of its low density and high contents of sand or fines throughout most of the Chengdu Plain.

6 Conclusions

In this paper, a V_s-based simplified procedure for evaluating the liquefaction potential of sandy soils is proposed, and a preliminary case study of Banqiao School in the Chengdu Plain is introduced. In situ soil profiles were obtained based on two borings after the earthquake. The field borings show that the alluvial deposit at the Banqiao School site mainly consists of sandy and loose gravelly soils at shallow depth, which are susceptible to liquefaction under strong earthquake ground motions. Seismic wave measurements by SASW and down-hole methods were conducted to obtain shear wave velocities. Based on the in-situ soil information and V_s profiles, the liquefaction potential of this site was evaluated. The main conclusions are summarized as follows:

(1) Detailed comparisons between the SASW and down-hole method indicate that the SASW method is a cost-effective and reliable in-situ method for determining shear wave velocity of soil deposits. By properly analyzing the test signals, such method provides a highly efficient tool in characterizing geophysical profile for the determination of liquefaction hazard mapping during the post-earthquake reconstructions;

(2) The critical layer predicted to liquefy lies between the depths of 3 and 6 m at the Banqiao School site, and this estimation is reasonably consistent with actual field behavior during the earthquake, where the ejected sands were similar to those sampled at this depth. The preliminary evaluation indicates that the proposed V_s-based simplified procedure is effective for liquefaction evaluation of sandy soils;

(3) Further effort on the effects of gravel or fines contents on the liquefaction behavior of sandy soils should be devoted to achieving a more reliable evaluation of ground deposits in the Chengdu Plain. Moreover, the liquefaction susceptibility of loose gravel formations should also be considered, since it might contribute to liquefaction damage in view of its low density and high contents of sand or fines in this area.

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