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A fundamental procedure and calculation formula for evaluating gravel liquefaction

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Abstract: Field investigations following the 2008 M_s8.0 Wenchuan earthquake identified 118 liquefaction sites, most of which are underlain by gravelly sediment in the Chengdu Plain and adjacent Mianyang area, in the Sichuan Province. Gravel sediment in the Sichuan province is widely distributed; hence it is necessary to develop a method for prediction and evaluation of gravel liquefaction behavior. Based on liquefaction investigation data and in-situ testing, and with reference to existing procedures for sandy soil liquefaction evaluation, a fundamental procedure for gravel liquefaction evaluation using dynamic penetration tests (DPT) is proposed along with a corresponding model and calculation formula. The procedure contains two stages, i.e., pre-determination and re-determination. Pre-determination excludes impossible liquefiable or non-liquefiable soils, and re-determination explores a DPT-based critical N_{120} blows calculation model. Pre-determination includes three criteria, i.e., geological age, gravel contents, gravel sediment depths and water tables. The re-determination model consists of five parameters, i.e., DPT reference values, gravel contents, gravel sediment depths, water tables and seismic intensities. A normalization method is used for DPT reference values and an optimization method is used for the gravel sediment depth coefficient and water table coefficient. The gravel liquefaction evaluation method proposed herein is simple and takes most influencing factors on gravel sediment liquefaction into account.

Keywords: gravel soil; liquefaction; dynamic penetration test; Wenchuan earthquake

1 Introduction

Soil liquefaction under earthquake loading is an important topic in soil dynamics and engineering practice. Thus, prediction and evaluation of liquefaction behavior is important, especially in highly seismic areas. Field investigations and in-situ testing techniques are effective ways to develop a better understanding of liquefaction phenomena. In the 1960s and 1970s, liquefaction was observed in several devastating earthquakes that occurred in mainland China. Through detailed investigation and systematic research, soil liquefaction evaluation methods for China have been developed and are adopted by Chinese seismic design codes (Liu, 2002; IEM, 1979).

On May 12, 2008, a devastating M_s8.0 earthquake struck the Sichuan Province, China. Through systematic and detailed field investigations, it was found that the liquefaction macro-phenomena is quite different from previous observations and new features were observed that created new areas of study (Cao *et al.*, 2010; Chen *et al.*, 2009). One such prominent feature was gravel liquefaction, which has been confirmed through specific investigations. Analysis also indicates that gravel liquefaction was predominant in this event. Geologically, gravel is widely distributed in the Sichuan Province, e.g., more than 8400 km^2 is located in the Chengdu basin alone (He, 1992). Furthermore, gravel is commonly used as bedding material in earth dams in China. Therefore, gravel liquefaction prediction and evaluation are important for engineering site selection and seismic fortification.

Compared with sandy liquefaction, documentation and experience with gravel liquefaction is very rare and relevant evaluation methods are not well developed. The current liquefaction prediction and evaluation methods that are established based on sandy soil liquefaction data and documentation are not applicable for gravel liquefaction prediction and evaluation, because widely-used techniques such as the standard penetration test (SPT) and cone penetration test (CPT) cannot be conducted on gravel sites. In addition, shear wave velocity testing can be used on both sandy sites and gravel sites. However, sand and gravel belong

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to different soil categories. Consequently, different physical soil properties result in different relative densities even though the shear wave velocities are equal. For example, sandy soil generally cannot liquefy when its shear wave velocity exceeds a certain value (e.g., 220 m/s), since it tends to be very dense under certain values; however, gravel with the same values remains loose and can possibly liquefy. According to preliminary research (Yuan *et al.*, 2009), the successful liquefaction determination rates for gravel sites are only about 30% accurate using current liquefaction prediction models that are based on shear wave velocities (Shi *et al.*, 1993). Thus, these determination results are obviously dangerous. Therefore, evaluation techniques for sandy soils are not suitable and evaluation methods and procedures to determine the potential for liquefaction of gravel are needed.

Japanese scholars have proposed gravel liquefaction evaluation methods based on large dynamic triaxial tests (Liu, 1998). Nevertheless, current large dynamic triaxial testing techniques are complicated and require the use of expensive equipment. In addition, scholars and engineers in the USA use Becker penetration tests (BPT) for gravel liquefaction evaluation (Youd *et al*., 2001). The BPT-based procedures rely on field testing data and have not yet been recommended and/or used in China. Furthermore, in the BPT method, penetration blows are converted into SPT penetration blows and then the possibility of site liquefaction is assessed using the SPT procedures. The SPT procedures were developed based on sandy soils, but the properties of gravel or cobble are quite different from sand. Hence, a straightforward conversion method from BPT to SPT is not reliable and requires further investigation.

In this paper, gravel liquefaction behavior in the Wenchuan earthquake is presented and then a fundamental procedure and a formula to evaluate gravel liquefaction through in-situ investigation and field testing data are proposed.

2 Gravel liquefaction investigation

Field investigations show the area affected by liquefaction in the Wenchuan earthquake is about 500 km long and 200 km wide, in which a rectangular area of 160 km × 60 km contains most of the liquefied sites (Yuan *et al*., 2009). The liquefied sites were distributed in Chengdu, Mianyang, Deyang, Meishan, Leshan, Suining, Ya'an and Guangyuan but were mainly located in Chengdu, Deyang and Mianyang areas. Moreover, liquefaction phenomena was observed in areas that experienced different intensities but mainly in Intensity VIII regions.

To investigate soil conditions in liquefied and nearby non-liquefied areas, more than 40 boreholes were drilled with continuous core sampling. The retrieved samples were logged to develop soil profiles as plotted on Fig.1. The boreholes were drilled with rotary equipment and core samples were cut and extracted with 90 to 100 mm diameter core barrels equipped with diamond bits. The bits commonly cut through cobbles and other large particles encountered and parts of the extracted core were disturbed by the rotary action. Intact sections of core without cut cobbles were selected for laboratory grain size testing. Typical extracted core samples are shown in Fig. 2. Therefore, liquefaction of gravel can be confirmed.

An upper layer of clayey fill, 1 m to 4 m thick, caps the soil profile of each borehole log as shown in Fig.1. The fill is underlain by thick sequences of gravelly sediment, the upper part of which is generally loose. Few non-gravelly sand layers were penetrated beneath the Chengdu plain. In the Mianyang area, however, gravelly coarse sand was commonly penetrated between depths of 1.2 m and 3.5 m as illustrated by the soil profile for Borehole E and F (Fig. 1). Thick deposits of dense gravel lie beneath the coarse sand.

3 Model and procedure

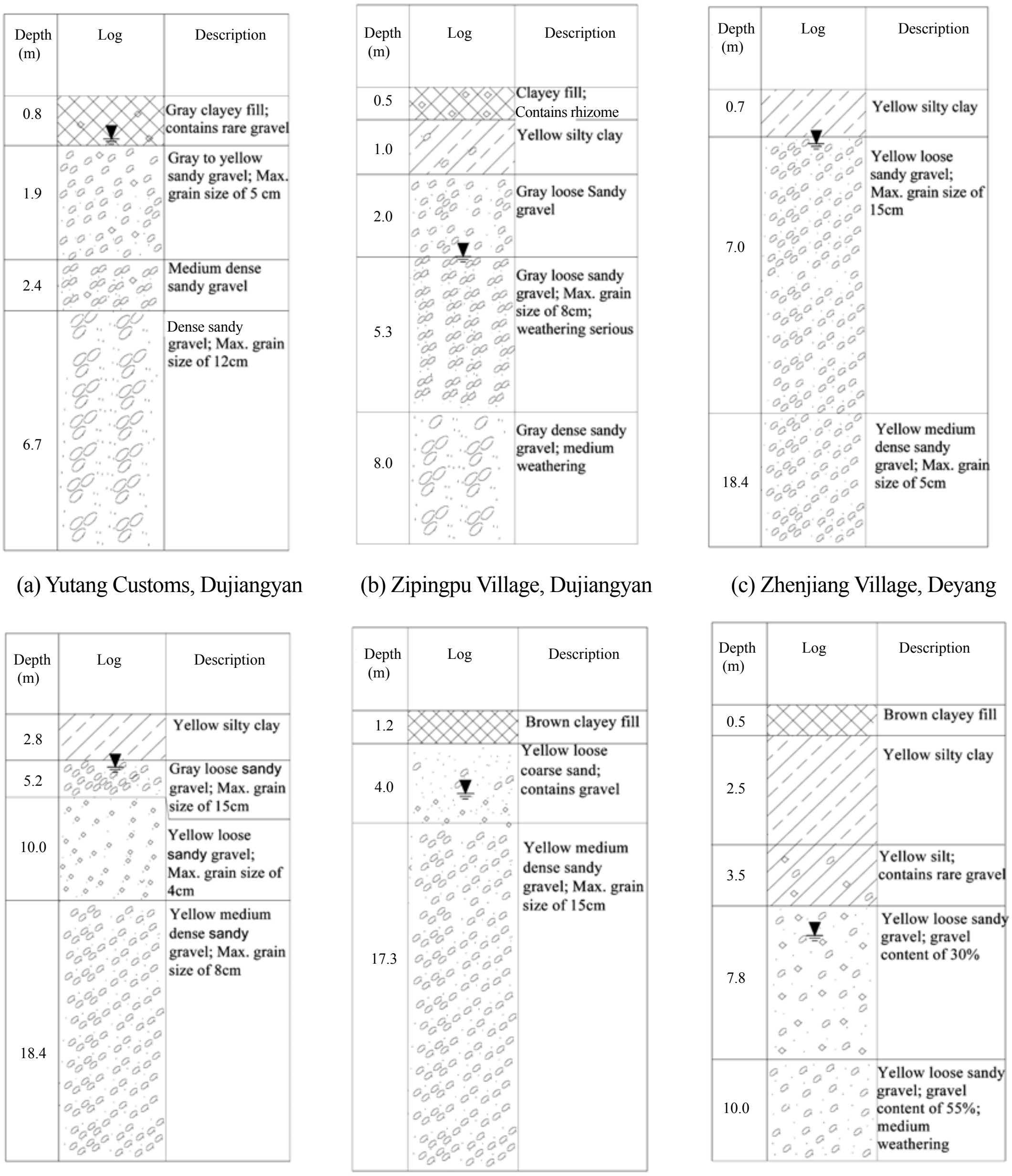
3.1 Index selection

The indices to evaluate liquefaction of the gravel layer have to be tested principally from in-situ investigations and testing; meanwhile, the testing techniques must be well developed and widely used. In China, the current fundamental liquefaction evaluation index is SPT blows. But SPT and CPT testing cannot be conducted in gravely layers. In Sichuan province where gravel is widely distributed, the dynamic penetration test (DPT), which is commonly used in China to measure penetration resistance of gravels during foundation investigations (Administration of Quality and Technology supervision of Sichuan Province PRC, 2001; The Ministry of Water Resources of the People's Republic of China, 1999), are used in engineering practice with an index of N_{120}, i.e., the number of blows required to achieve a 30 cm penetration of the sampler. Herein, N_{120} is selected as an index for evaluating gravel liquefaction. DPT is an ordinary technique for gravel investigation and N_{120} is a continuous variable which can represent many properties of course-grained soils.

However, the components of course-grained soil such as gravel soil are complicated, and an empirically simple N_{120} cannot fully represent its liquefaction potential. Therefore, gravel content is selected as another fundamental index. Furthermore, investigation results show that seismic intensities, water tables and depths of liquefiable gravel layers also have an important effect on gravel liquefaction. Therefore, earthquake intensity and soil conditions must be accounted for in the new model for gravel liquefaction evaluation.

3.2 Dynamic penetration tests

During this investigation, the Chinese DPT was used



(a) Yutang Customs, Dujiangyan (b) Zipingpu Village, Dujiangyan (c) Zhenjiang Village, Deyang

(d) Qifu Primary School, Deyang (e) Sanyuan Village, Deyang (f) Lingfeng Machine Co. Ltd., Mianyang

Fig. 1 Typical borehole logs from drilled liquefaction sites.

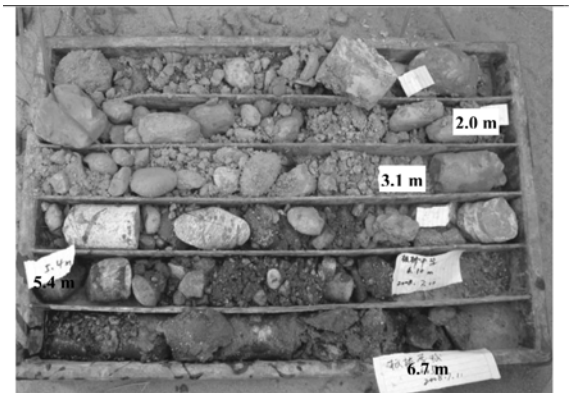


Fig. 2 Typical extracted core samples

for the first time to measure the penetration resistance of gravels that liquefied. DPT profiles were compiled from 36 soundings at localities where liquefaction effects were or were not observed. These gravels were too coarse to allow effective use of either SPT or CPT, the most commonly used penetration tests for liquefaction investigations worldwide. The DPT equipment consists of a 120 kg hammer, with a free fall height of 100 cm, dropped onto an anvil attached to 60 mm diameter drill rods which are in turn attached to a solid cone tip with a diameter of 74 mm and a cone angle of 60 degrees. The drill rods have a smaller diameter than the cone tip to reduce the friction between the rods and the soil. DPT blow counts are defined as the number of hammer drops required to advance the cone tip 10 cm. However, N_{120} is the number of blows required to drive the tip 30 cm and is simply calculated by multiplying raw blow counts by a factor of 3, thus preserving the 10-cm detail of the raw blow counts. The 30 cm drive length was specified in Chinese codes to be consistent with the drive length specified for standard penetration tests (SPT). A diagram of the penetrometer tip and DPT apparatus is shown in Fig. 3. DPT logs from three of the four selected sites on the Chengdu Plain are plotted in Fig. 4. The lowest DPT resistance below the water table was the primary measure used to determine the soil layers that liquefied. At these four sites, DPT resistance of less than 5 blows/ 10 cm were generally indicative of liquefaction. These lower resistances were measured at shallow depths (<10 m) at sites that were strongly shaken by the earthquake (Intensity VII to IX with estimated a_{max} between 0.15g and 0.45g). The results and analyses of the DPT tests are the subject of a subsequent paper on liquefaction during the Wenchuan earthquake.

The DPT is a very rugged instrument, capable of penetrating dense gravel layers and breaking or displacing cobbles as it is driven. In loose gravels ($N_{120} \leq$ 4), interference of large particles to penetration generally causes narrow penetration spikes, such as those plotted on the penetration logs in Fig 4. After a large particle was fractured or pushed aside, the penetration resistance returned to the matrix value for the deposit. Penetration depths up to 14 m were easily attained at most sites; however large cobbles and boulders proved to be impenetrable.

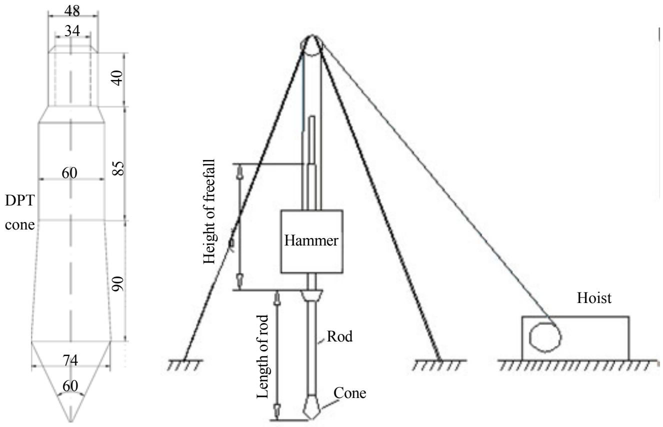
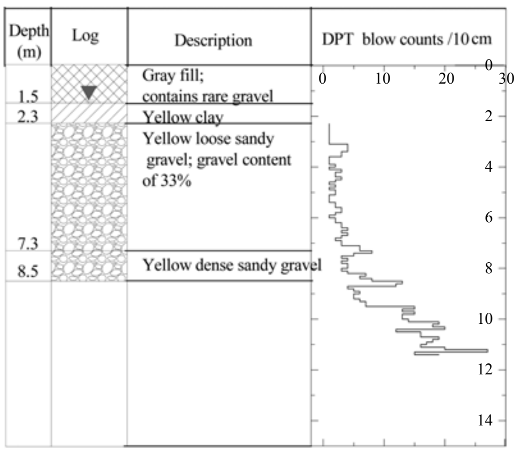
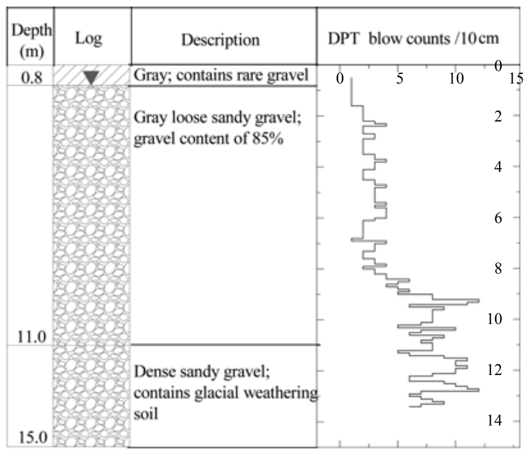


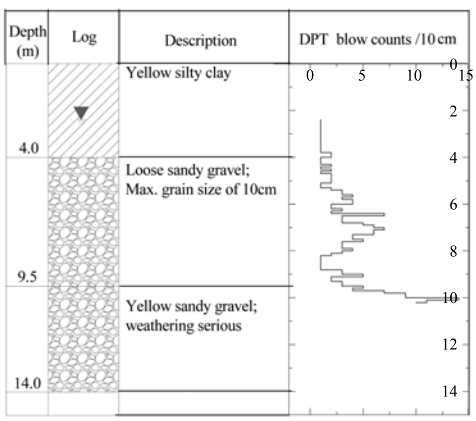
Fig. 3 Components of the DPT apparatus



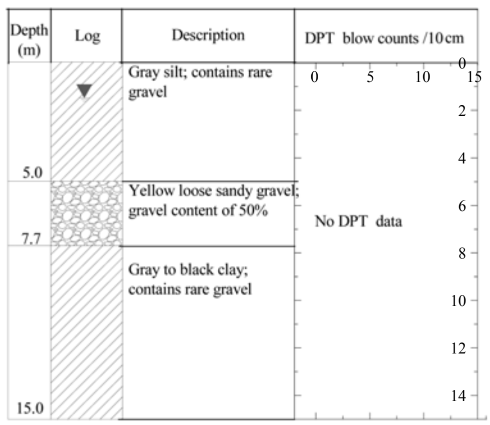
(a) Site 1—Pilu Elementary School



(b) Site 2 — Songbai Village



(c) Site 3 — Xinglong Village



(d) Site 4 — Wudu Village

Fig. 4 Soil log and DPT blows for selected liquefaction Sites 1 through 4

3.3 Test data

In this paper, 35 typical liquefied sites, on which liquefaction generated obvious ground failure or caused serious damage to structures, are selected for the in-situ investigation and testing. The site distribution is shown in Fig. 5, and includes 14 liquefied sites and 21 non-liquefied sites. The selected sites are located in different intensity regions, including eight sites in intensity VII, 17 sites in intensity VIII and 10 sites in intensity IX. Table 1 lists the site data, including seismic intensities, water tables, gravel sediment depths and N_{120}.

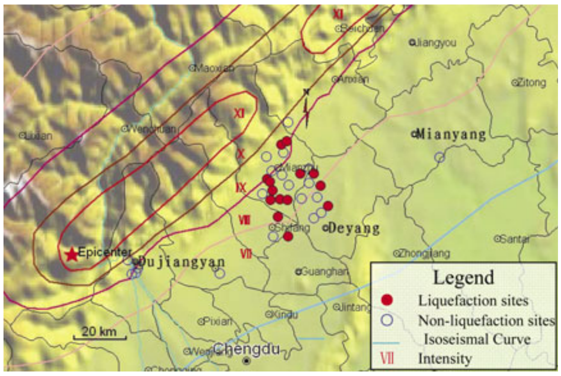


Fig. 5 Distribution of DPT sites

Table 1 Gravelly liquefied sites data

No.	Location	Liquefied or nonliquefied	Intensity	d_s (m)	d_w(m)	N_{120}
1	Pilu Elementary School	Liq.	VII	2.3-8.0	1.4	7.5
2	Guoyuan	Liq.	VII	1.5-2.2	1.5	9.0
3	Jinqiao	Liq.	VII	4.0-6.1	2.2	6.3
4	Xinshi School	Liq..	VIII	2.5-3.5	1.0	6.3
5	Banqiao School	Liq.	VIII	3.0-6.1	3.0	10.2
6	Songbai	Liq.	VIII	0.8-8.3	0.8	7.5
7	Xinglong	Liq.	VIII	4.0-9.5	2.4	8.7
8	Shihu	Liq..	VIII	2.9-5.8	2.9	11.4
9	Qifu Elementary School	Liq.	VIII	3.5-7.0	3.5	11.1
10	Guihua	Liq.	VIII	0.6-3.7	0.6	8.1
11	Zhenjiang	Liq.	VIII	1.8-2.9	0.9	8.7
12	Sangyuan	Liq..	VIII	2.8-4.2	2.8	11.7
13	Xiangliu	Liq.	IX	3.4-6.2	3.4	17.4
14	An’ren	Liq.	IX	4.0-6.0	4.0	14.1
15	Wulang	Non	VII	5.0-13.0	5.0	13.8
16	Quezhu	Non	VII	6.0-15.0	6.0	24.6
17	Yangjia Railway Station	Non	VII	6.1-8.7	6.1	22.5
18	Nangui	Non	VIII	9.8-14.0	4.7	14.1
19	Pharmacy Factory	Non	VIII	3.4-7.4	3.4	14.1
20	Pinghe	Non	VIII	9.6-12	3.7	27.0
21	Bayi	Non	VIII	6.2-7.2	6.2	15.9
22	Yongning	Non	VIII	8.1-12.2	1.4	37.5
23	Dacheng	Non	VIII	5.7-7.8	4.5	23.1
24	Min’an	Non	VIII	7.3-9.0	3.7	17.7
25	Wufang	Non	VIII	3.6-5.6	2.0	18.3
26	Chuanmu	Non	IX	8.5-9.9	8.0	22.8
27	Tonglin	Non	IX	9.4-11.0	2.0	22.5
28	Technology College	Non	IX	2.3-4.6	2.3	18.0
29	Guankou Financial building	Non	IX	2.7-4.9	2.7	23.7
30	Kaiqiao	Non	IX	2.4-5.8	0.8	41.4
31	Tianfu Luxin Kindergarten	Non	VII	1.4-2.8	1.4	21.9
32	Ruikang Garden	Non	IX	5.4-8.3	5.4	48.0
33	Zipping	Non	IX	3.0-5.3	3.0	23.4
34	Yutang Customer Hotel	Non	IX	1.5-2.5	1.5	22.5
35	Lingfeng	Non	VII	4.1-8.1	4.1	6.3

3.4 Procedure

The gravel liquefaction evaluation method consists of stages similar to the evaluation method for sandy soils, i.e., pre-determination and re-determination. Pre-determination excludes impossible liquefiable or non-liquefiable soils. After that, the actual evaluation of liquefaction potential for gravel is conducted using the re-determination calculation formula.

3.4.1 Pre-determination analysis

(1) Geological aging. Geological aging is one of the initial criteria for pre-determination, In this event, deposits on pre-Quaternary layers (Quaternary included) have not liquefied. Thus, they are non-liquefiable.

(2) Gravel content. Grain size distribution curves from extracted samples at different intensity sites are plotted in Fig. 6. The upper limits of the gravel content are used as a pre-determination criterion. If the gravel content in intensity VII, VIII and IX is greater than 70%, 75% and 80%, respectively, liquefaction will not be considered.

(3) Gravely sediment depths and water tables. Figure 7 shows gravel layer depths on liquefied sites and non-liquefied sites along with corresponding water tables. The characteristic gravel depths are as given in Table 2. Liquefaction is taken into account if the thickness of the non-liquefied caps and water tables are larger than the values presented in Fig.7.

3.4.2 Re-determination model

The former sand liquefaction formula in Chinese seismic design codes is,

$$N_{\mathrm{cr}} = N_0[1+\beta_{\mathrm{w}}(d_{\mathrm{w}}-2)+\beta_{\mathrm{s}}(d_{\mathrm{s}}-3)] \quad (1)$$

where N_{cr} is a critical SPT blow; N_0 is a referring SPT blows; d_{s} is a sandy layer depth; d_{w} is a water table; β_{w} is a water table influencing coefficient; and β_{s} is a sand layer depth coefficient. The formula represents a fine-grain soil liquefaction evaluation model which was established based on liquefaction investigation data from the 1975 Haicheng earthquake and the 1976 Tangshan earthquake, and has been widely used in engineering practice in China.

Adopting Eq. (1), a gravel liquefaction evaluation model can be developed using N_{120} as a basic index as follows,

$$N_{\mathrm{cr-120}} = N_{0-120}[1+\alpha_{\mathrm{w}}(d_{\mathrm{w}}-2)+\alpha_{\mathrm{s}}(d_{\mathrm{s}}-3)] \quad (2)$$

where $N_{\mathrm{cr\text{-}120}}$ is a critical DPT blow; $N_{0\text{-}120}$ is a referring DPT blow; d_{s} and d_{w} are the same as in Eq. (1); α_{w} is a water table influencing coefficient; and α_{s} is a gravely layer depth coefficient. The evaluation of the coefficients in Eq. (2) is discussed in the next section.

3.5 Coefficient evaluation

3.5.1 $N_{0\text{-}120}$

In the Chinese seismic design codes, liquefaction data for fine sands where water tables are around 2 m and sandy soil depths are commonly 3 m are used to determine N_0 so that the relationship of SPT blows and intensities can be established. The boundary depth between the liquefied and non-liquefied areas can be easily delineated and N_0 can be obtained.

However, gravel liquefaction data from the Wenchuan earthquake indicated that gravel layer depth and water tables vary remarkably. Hence, it is difficult to easily establish the relationship between DPT blows and intensities. Adopting the current correcting formula for shear wave velocity (Sykora, 1987; Shi *et al*., 1993), the measured DPT blows can by corrected to values with 3 m gravel depth and 2 m water table. The formula is,

$$N'_{120} = N_{120}(47/\sigma'_v)^{0.5} \quad (3)$$

where, N'_{120} is corrected DPT blows; and N_{120} is a measured DPT blow. Figure 8 delineates the boundary between the liquefied and non-liquefied areas. The

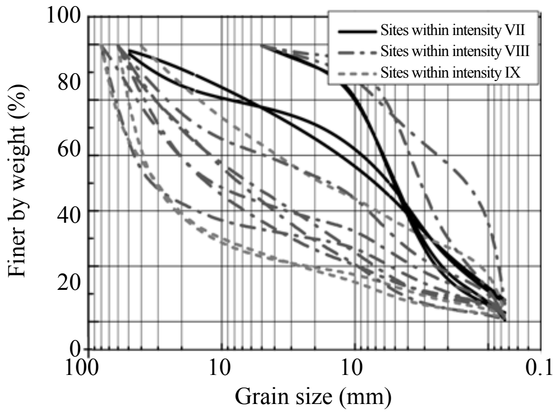


Fig. 6 Upper limit of gravel content to liquefy with different intensities

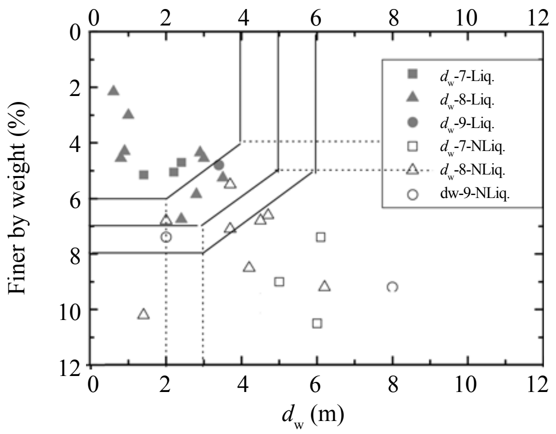


Fig. 7 Relationships of water tables and gravel sediment depths on liquefied and non-liquefied sites

Table 2 Characteristic depths of the liquefied gravel soils

Intensity	VII	VIII	IX
Characteristic depth (m)	6	7	8

reference values can be read from the plot, and are shown in Table 3.

3.5.2 Coefficients of gravel depth and water tables, α_s and α_w

According to the field testing results, the liquefied gravely layer depths and water tables vary within a considerable range. Furthermore, the data are very limited, so directly deducing α_s and α_w will be uncertain at best. Therefore, an optimization method is explored to minimize the uncertainty.

Figure 9 shows the charts of dynamic penetration ratios (DPR) with gravel layer depths defined as the measured DPT blows N_{120} divided by DPT reference values $N_{0\text{-}120}$. The slopes of the boundary between the liquefied sites and the non-liquefied sites in Fig. 9 represent the influencing coefficients of gravely layer depth. Similarly, Fig. 10 presents the results of DPR with respect to water tables, from which the influencing coefficients of the water tables can be observed.

Note that in Fig. 9 and Fig. 10, the obtained α_w and α_s vary within a certain range due to a small number of data. To achieve the best values, successful determination rates for liquefied sites and non-liquefied sites under various α_w and α_s, are plotted in Fig. 11 and Fig. 12. The results in Fig. 11 and Fig.12 are combined, so that the overlapping area (shadow in Fig. 13) presents the best values of α_w and α_s where the successful determination rate for all liquefied and non-liquefied sites exceed 90%. To simplify Eq. (2), the values for α_w and α_s are 0.05 and -0.05, respectively.

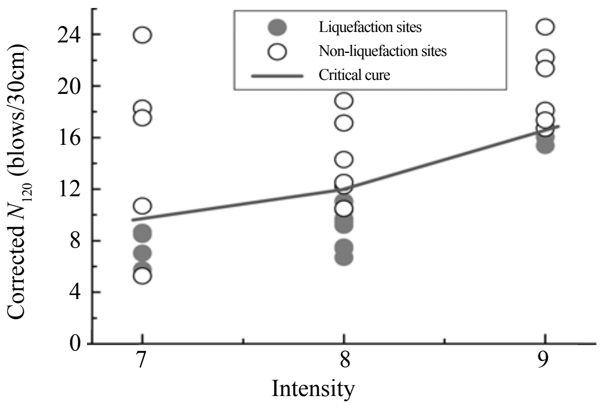


Fig. 8 Critical cure of N_{120} to determine liquefaction sites from non-liquefaction sites

Table 3 Reference $N_{0\text{-}120}$ values

Intensity	VII	VIII	IX
$N_{0\text{-}120}$	9	12	16

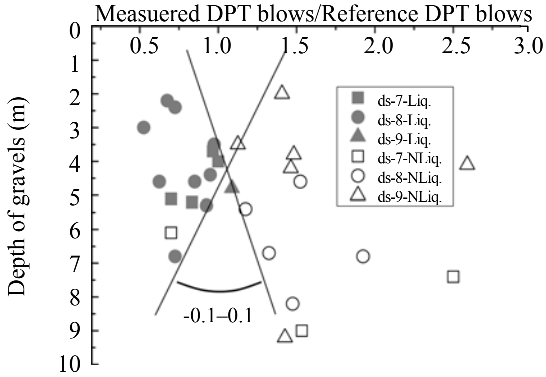


Fig. 9 Gravel depth influence coefficient to gravel liquefaction resistance

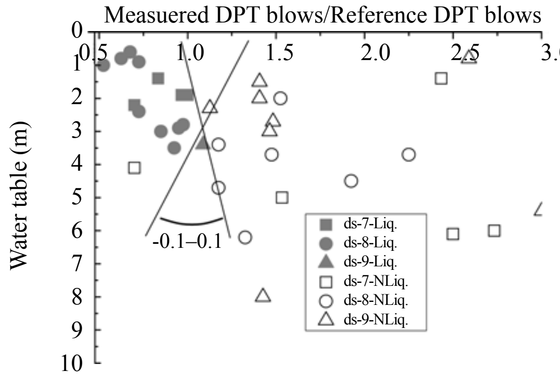


Fig. 10 Water table influence coefficient to gravel liquefaction resistance

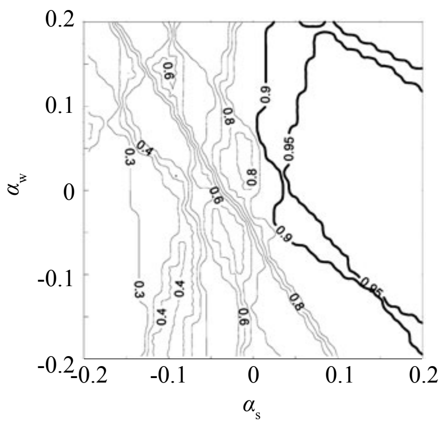


Fig. 11 Successful determination rates of liquefied sites under various α_w and α_s

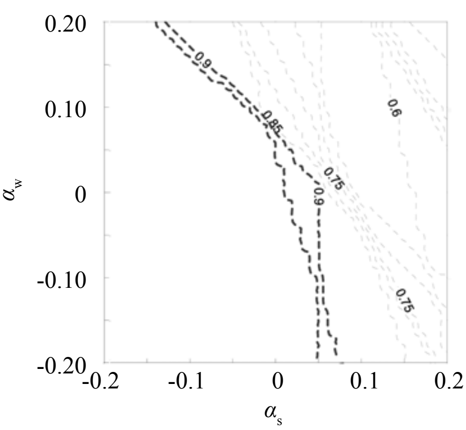


Fig. 12 Successful determination rates of non-liquefied sites under various α_w and α_s

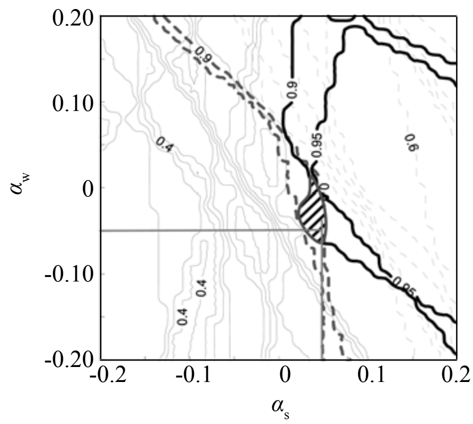


Fig. 13 Optimized values for α_w and α_s

3.5.3 Gravel content P_5

A gravel content P_5 is defined as the percentage of grain with particle sizes larger than 5 mm. From the investigation results, the influence of P_5 on gravel liquefaction potential currently can be applied in the pre-determination stage, since field testing data on the influence of different gravel contents on gravel liquefaction resistance is difficult to obtain and quantitative analytical results cannot be proposed.

Currently, large dynamic triaxial tests have been used to evaluate the influence of gravel contents on gravel liquefaction potential (Wang *et al*., 2000). The results illustrate that gravel liquefaction strength increases approximately linearly as the gravel contents increase, e.g., the liquefaction strength with 80% gravel content is 5% to 10% greater than with 35% gravel content, and 1% to 6% greater than with 50% gravel content. As a result, the coefficient of gravel content influence is

$$\alpha_p = 1 + 0.2\left(P_5 - 50\%\right) \tag{4}$$

4 Calculation formula

Based on the discussion in the previous section, the critical DPT-based gravel liquefaction re-determination formula can be written as,

$$N_{\mathrm{cr-120}} = N_{0-120}[1 + \alpha_{\mathrm{w}}(d_{\mathrm{w}} - 2) + \alpha_{\mathrm{s}}(d_{\mathrm{s}} - 3)] \cdot [1 + \alpha_p(p_5 - 50\%)] \tag{5}$$

The coefficients α_s, α_w and α_p are 0.05, -0.05 and 0.2, respectively. Eq. (5) can be simplified as,

$$N_{\mathrm{cr-120}} = N_{0-120}[1 - 0.05(d_{\mathrm{w}} - 2) + 0.05(d_{\mathrm{s}} - 3)] \cdot [1 + 0.2(p_5 - 50\%)] \tag{6}$$

or,

$$N_{\mathrm{cr-120}} = N_{0-120}[0.95 + 0.05(d_{\mathrm{s}} - d_{\mathrm{w}})] \cdot [1 + 0.2(p_5 - 50\%)] \tag{7}$$

Using Eq. (7), if the measured N_{120} blows is greater than $N_{\mathrm{cr-120}}$ then the gravel layer is deemed as liquefied, otherwise it is non-liquefied. Meanwhile, Eq. (7) is used to determine the testing sites in reverse, and the successful determination rates are 93% for liquefied sites and 90% for non-liquefied sites. The reliability of the procedure and formula is verified and confirmed.

5 Conclusions

Through liquefaction investigation and in-situ testing, a DPT-based gravel liquefaction evaluation method is proposed. The principles and a procedure are developed and a formula is established. The conclusions are as follows:

(1) SPT-based liquefaction evaluation methods developed on sandy soil liquefaction data are not applicable to gravel layers or coarse grain soil. Moreover, SPT cannot be conducted on gravel or cobble layers. In this paper, a new index DPT N_{120} is selected for gravel layer liquefaction evaluation.

(2) Gravel liquefaction evaluation includes two stages, similar to sandy soil liquefaction evaluation, i.e., pre-determination and re-determination. Pre-determination exempts non-liquefiable cases and re-determination explores a DPT N_{120}-based calculation formula.

(3) Pre-determination takes into account geological age, gravel layer depths and gravel contents. The criteria are: areas underlain by pre-Holocene soils will not liquefy; characteristic liquefiable gravel layer depths are 6 m, 7 m and 8 m with respect to seismic intensities of VII, VIII and IX; and gravel content limits are 70%, 75% and 80% with respect to seismic intensities of VII, VIII and IX.

(4) The gravel liquefaction evaluation model considers five parameters, i.e., DPT reference values, gravel contents, gravel layer depths, water tables and seismic intensity. An optimization method is used to obtain the influencing coefficients of gravel layer depths and water tables and a normalization method is used to obtain DPT reference values.

(5) The proposed evaluation model and formula are simple and can be easily applied in engineering practice. However, further study is needed to improve the model and formula, including further verification by using more investigation data, consideration of gravel content influence and adoption of the peak ground acceleration (PGA) instead of seismic intensities, etc.

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