Journal of Modern Transportation
Volume 19, Number 2, June 2011, Page 134-142
Journal homepage: jmt.swjtu.edu.cn
DOI: 10.1007/BF03325750

Characteristics of seismic responses at liquefied and non-liquefied sites with same site conditions

Jianjing ZHANG [1,2*], Changwei YANG

1. The Institute of Geological & Nuclear Science, Lower Hutt 5040, New Zealand
2. School of Civil Engineering, Southwest Jiaotong University, Chengdu 610031, China

Abstract: To understand the characteristics of seismic response at liquefied sites, a liquefiable site and a non-liquefiable site were selected, separated by about 500 m and having the same site conditions as Class D in the National Earthquake Hazards Reduction Program (NEHRP). A suite of earthquake records on rock sites are selected and scaled to the spectrum of the Joyner, Boore, and Fumal (JBF) attenuation model for a magnitude 7.5 earthquake at a distance of 50 km. The scaled records were then used to excite the two sites. The effect of pore-water pressure was investigated using the effective stress analysis method, and nonlinear soil behavior was modeled by a soil bounding surface model. Comparisons for spectra, peak ground acceleration (PGA), peak ground displacement (PGD) and permanent displacement were performed. Results show that the mean ground response spectrum at the non-liquefied site is close to the estimated ground response spectrum from the JBF model, but the mean ground response spectrum at the liquefied site is much lower than the estimated ground response spectrum from the JBF model for periods of up to 1.3 s. The mean PGA at the non-liquefied site is about 1.6–1.7 times as large as that at the liquefied site, but the mean peak ground displacement (PGD) at the non-liquefied site has a slight difference with that at the liquefied site. The mean permanent displacements at the liquefied site are larger than those at the non-liquefied site, particularly at the liquefied layer.

Key words: seismic responses; liquefied and non-liquefied; excessive pore-water pressure

1. Introduction

As an important seismic hazard, liquefaction has attracted considerable attentions. However, due to a lack of earthquake records on liquefied sites, in practice, the design spectra or site-specific probabilistic seismic hazard (PSH) spectra for non-liquefiable sites are often applied to liquefiable ones. This is despite the fact that a few strong motion records measured on liquefied sites have shown the ground response at the liquefied site is different from that at the non-liquefied site in vibration amplitude, frequency, and duration.

Design spectra or PSH spectra are not only dependent on earthquake magnitude and source distance, but also site class [1]. Generally, a site class consists of a group of sites with similar properties, such as an average shear wave velocity of 30 m depth. This is one reason why the spectral acceleration estimated from strong motion attenuation relations has uncertainties for a specific site class. However, when design spectra or PSH spectra from the non-liquefiable sites are applied to the liquefiable sites (even at the same site class), the uncertainties for the replacement and its effect on structural design are unknown, and this aspect has been given little attention.

In recent years, Youd et al. [2] have performed some comparative studies between the records from the Wildlife site after the 1987 Elmore Ranch and Superstion Hills earthquakes and numerical analysis. Through the comparison, Youd et al. [2] pointed out that, due to soil stiffness softening in liquefaction, spectral accelerations attenuate for periods less than 0.7 s, but for periods larger than 0.7 s, spectral accelerations are amplified. Similar work has been performed by Lopez [3] for the Port Island in the 1995 Kobe earthquake, the Wildlife site in the 1987 Superstition Hills earthquake, and the Treasure Island-Yerba Buena Island in the 1989 Loma Prieta earthquake. But Lopez obtained different results from those of Youd et al., except for the Wildlife site. From the results of Youd et al. and Lopez [2], we note: 1) equivalent linear models were used to model soil nonlinear response, in that pore-water pressure was not taken into account, and 2) liquefied site response is dependent on the strength of excitation, indicating that it is very difficult to use one excitation to obtain a reliable

Received Nov. 16, 2010; revision accepted Apr. 22, 2011
*Corresponding author. E-mail: jianzhang1102@sina.com (J.J. ZHANG)
doi: 10.3969/j.issn.2095-087X.2011.02.008

result, because even if an estimated earthquake magnitude and source distance are given, the selected excitation will contain a number of uncertainties.

To understand the uncertainties in replacing non-liquefied site response to liquefied site response, a better way is to compare site responses at liquefied and non-liquefied sites, with similar conditions at two types of sites, i.e., experiencing a similar natural period and the same suite of excitations selected based on design spectra or PSH spectra. Meanwhile, we note that liquefiable site can be generally classified into Class D in the National Earthquake Hazards Reduction Program NEHRP classification and investigations from strong earthquakes have shown that the soils in Class D have a nonlinear response in this situation. Relative to the nonlinear soil response, whether liquefaction occurs will depend on the ratio of pore-water pressure increment to effective confined pressure, as shown by Seed and Idress [4]. Normally, like most site responses, liquefiable sites show linear elastic response under weak shaking. As the strength of shaking increases, soil response gradually enters inelastic state, and if the strength of shaking arrives at a point, where pore-water pressure equates the overburden pressure and the sand losses its strength, liquefied state occurs. Youd et al. [2] and Lopez [3] modeled the first and second states, but failed to model the third state. To clarify the characteristics of liquefying site response, understanding all three states is necessary.

In this study, we attempt to investigate mean values using non-liquefied site response instead of liquefied site response. For the purpose, we choose two sites which are all classified into Class D in NEHRP, one for liquefiable site and the other for non-liquefiable site, the distance between the two sites is about 500 m, and the natural periods are 1.31 s and 1.32 s, respectively. Effective stress analysis method is used to investigate the change of pore-water pressure, and simplified bounding surface models are used to model truly nonlinear soil behavior. A suite of earthquake records on rock sites have been selected from the Next Generation Attenuation strong motion dataset, and then scaled to the Boore, Joyner and Fumal (BJF) [5] acceleration spectra in a period range of $0.2T_0$ to $1.5T_0$, where T_0 is the natural period of the site, for a magnitude 7.5 earthquake at a distance of 50 km. The reason for selecting a suite of records is that we attempt to obtain the mean values for various intensity-measure parameters. These scaled records are then used to excite the two sites. We have noted that different scaling approaches have been proposed [6], but we prefer to use the scale approach, because the scale approach not only captures the effect of the first mode, but also accounts for soil stiffness softening and possible higher mode effect. The comparisons of seismic responses at the liquefiable and non-liquefiable sites are performed from the following aspects: mean site response spectra, PGA, PGD, and permanent displacements on the ground surface and along the depth.

2. Liquefiable and non-liquefiable sites and soil models

To assess the liquefaction potential, a common approach used in liquefaction studies is to compare liquefied and non-liquefied site responses, such as the cyclic stress ratio method [4] or Arias Intensity method [6]. Sometimes liquefied and non-liquefied sites are close to each other; for this reason, it is convenient to carry out the comparison of liquefied and non-liquefied site responses. A typical example for this kind of sites is boreholes 1 (BH1) and 3 (BH3) in the Marina district, California, where the two boreholes are only 500 m away. In the 1989 Loma Prieta earthquake, no building damage and sand boils occurred near BH1, but did near BH3. The reason for the difference is that the top of soil layers are different at the two sites and water table lies at 3.1 m deep, as shown in Table 1.

Table 1 Site profiles (water table 3.1 m)

No. of layers	BH1			BH3		
	Soils	Shear wave velocity V_s (m/s)	Depth (m)	Soils	V_s (m/s)	Depth (m)
1	Loose sand	138	3.1	Loose sand	138	6.9
2	Dense sand	179	7.2	Younger bay mud	141	10.7
3	Younger bay mud	141	10.7	Very dense sand	378	16.9
4	Very dense sand	378	13.1	Silty clay	–	17.4
5	Sandy clay/silty clay	–	14.6	Very dense sand	378	24.1
6	Very dense sand	378	22.9	Older bay mud	252	79.5
7	Older bay mud	252	79.5	–	800	>79.5
8	–	800	>79.5			

In this study, we selected the two sites to investigate the characteristics of liquefied and non-liquefied site responses. The parameters of bounding surface models for each site have been derived from the laboratory and listed in Table 2 [7]. Effective stress analysis software, SUMDES [8], was used in this study, which has been verified as an efficient tool to perform liquefied site response analysis [9].

3. Selection of ground-motion records

To perform the comparison, we assumed that the two sites are subjected to a magnitude 7.5 earthquake at a source distance of about 50 km, and then used the Boore, Joyner and Fumal (BJF) [5] acceleration spectral attenuation model to control the strength of ground shaking. The reason for choosing the BJF model is that site amplification in the model is a continuous function of average 30 m-depth shear wave velocity (V_{s30}), which can reflect the slight difference of the two site conditions. Site profile (in Table 2) shows that engineering rock lies below the ground about 79.5 m deep and has a shear wave velocity of 800 m/s.

On the basis of the required magnitude and source distance, some ground-motion records were selected from the Next Generation Attenuation Model database. Because of a lack of sufficient recordings on sites with V_{s30}=800 m/s, some records recorded on sites with V_{s30} less than 800 m/s were selected, but the minimum V_{s30} is larger than 553 m/s. The selected records are listed in Table 3.

Table 2 Parameters for soil bounding surface models

Property	Value				
	Younger bay mud	Older bay mud	Loose sand (fill and hydraulic fill)	Dense sand (beach sand)	Very dense sand (hard pan)
Specific gravity	2.7	2.7	2.65	2.65	2.65
Void ratio	1.18	1.04	0.78	0.66	0.60
K_0 (earth pressure coefficient)	0.50	0.50	0.73	0.66	0.66
Coefficient of permeability (m/s)	1×10^{-9}	1×10^{-9} (assumed)	1×10^{-5} (assumed)	1×10^{-5} (assumed)	1×10^{-5} (assumed)
Elastic shear modulus (MPa)	35.8	127	32.6	61.1	286
λ (soil's compressibility along the isotropic virgin loading path)	0.21	0.29	0.023	0.019	0.019
K (soil's compressibility along the isotropic unloading-reloading paths)	0.02	0.04	0.002	0.001	0.001
M_e	1.35	1.35	1.0	1.0	1.0
M_e/M_c	0.8	0.8	0.8	0.8	0.8
M_u	–	–	1.4	1.4	1.4
I_0/I	–	–	1.3	2.0	20.0
R_c	2.0	2.0	2.5	2.5	2.5
A_c	0.4	0.4	–	–	–
C	0.3	0.3	0.0	0.0	0.0
α	–	–	2.0	5.0	5.0
h_e	6.0	6.0	0.5	0.5	0.5
h_e/h_c	1.0	1.0	1.0	1.0	1.0
H_u	–	–	10.0	10.0	10.0

Table 3 Earthquake records used in this study

No.	Station	Earthquake	Date	V_{s30}	Radius (km)	Magnitude	Direction
1	TWENTYNINE PALMS	HECTORMINE	16/10/1999	685	42	7.13	E
2	TWENTYNINE PALMS	LANDERS	28/06/1992	685	41	7.28	E
3	CARLO (TEMP)	ALASKA	03/11/2002	964	50	7.9	N
4	R109 (TEMP)	ALASKA	03/11/2002	964	43	7.9	N
5	HWA023	CHI-CHI	20/09/1999	553	47	7.6	N
6	HWA046	CHI-CHI	20/09/1999	553	48	7.6	N
7	HWA057	CHI-CHI	20/09/1999	553	46	7.6	N
8	HWA058	CHI-CHI	20/09/1999	553	41	7.6	E
9	ILA063	CHI-CHI	20/09/1999	553	58	7.6	W
10	NSK	CHI-CHI	20/09/1999	553	55	7.6	E
11	SILENT VALL	LANDERS	28/06/1992	685	51	7.28	E
12	TCU017	CHI-CHI	20/09/1999	559	54	7.6	N
13	TCU025	CHI-CHI	20/09/1999	553	52	7.6	N
14	TCU085	CHI-CHI	20/09/1999	553	55	7.6	N
15	TCU094	CHI-CHI	20/09/1999	590	55	7.6	N
16	TTN024	CHI-CHI	20/09/1999	553	56	7.6	N

The selected records were then scaled to the spectra constructed from the BJF model. Different from some approaches, where the scaling is based on either PGA, arias intensity or CAV (cumulative absolute velocity) [10], in this study, a similar scaling approach as used in building loading standards was applied. In this approach, first a target spectrum was derived from an acceleration response spectra attenuation model or from a probabilistic seismic hazard model; the natural period, T_0, of the site is then determined based on four times the average propagation time of seismic waves. In a period range of $0.2T_0$ to $1.5T_0$, the spectra of selected records are scaled to the target spectrum, under a condition that the scale factor from the calculation is less than 3.0. This strategy was adopted because seismic wave amplification closely depends on the natural period of the site, and the period range accounts for soil stiffness degradation and the effect of higher modes in sites. The target and mean scaled spectra are shown in Fig. 1. In the scaled period range, the mean scaled spectrum better matches the target spectrum.

4. Liquefied and non-liquefied seismic site responses

Liquefaction occurs when, during strong ground shaking, pore-water pressure equates the overburdened pressure. The selected sites, BH1 and BH3, are of Class D in NEHRP. When these sites are subjected to strong ground shaking, a nonlinear soil response may occur. If liquefaction occurs for the liquefiable site, the liquefaction will make the soils generate a stronger nonlinear soil response [11]. To show these characteristics, the relationships of shear stress and strain at a depth of 4.6 m at BH1 and BH3 under the shaking of the HWA023 record are shown in Fig. 2. Compared with the response at BH1 site, BH3 site shows a stronger nonlinear response. Note that BH1 site also shows a nonlinear soil response,

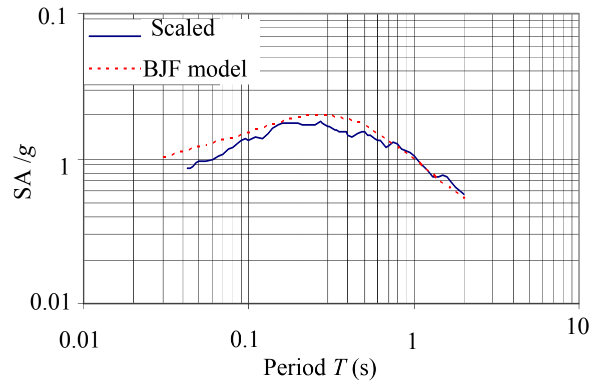
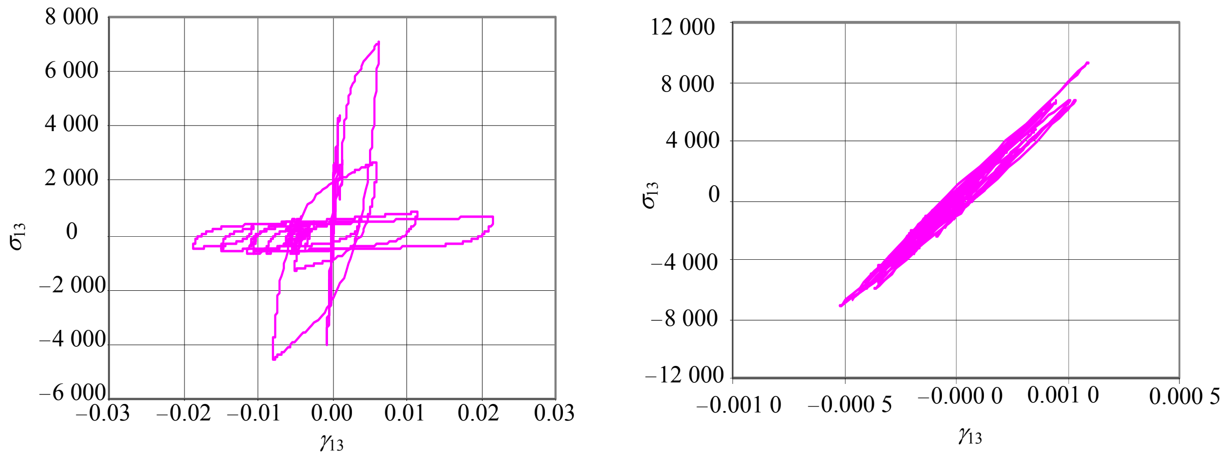


Fig. 1 Comparison between the spectra from the BJF model and the mean scaled acceleration spectra

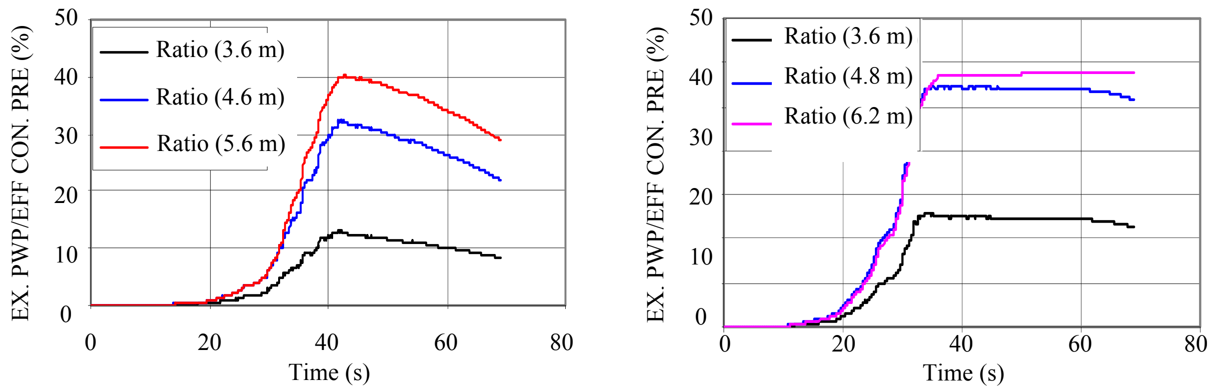
though no liquefaction occurs. At BH3 site, initially the soils display a nonlinear response, but once the pore-water pressure is beyond the overburdened pressure, the shear strain of the soils quickly increases. As shown in Fig. 2(a), where the maximum shear strain is 0.000 4 at BH1 site, but 0.021 at BH3 site. The relationship between shear stress and strain only shows the extent of the soil nonlinear response, rather than the pore-water pressure increment. Fig. 3 shows the ratio of excessive pore-water pressure to effective confined pressure against time at three depths, 3.6 m, 4.8 m and 6.2 m, for BH1 and BH3 sites. The ratios at depths of 4.8 m and 6.2 m at BH3 site are slightly larger than 100%, indicating that liquefaction has occurred. With respect to that at BH3 site, the ratio at BH1 site (even if at 5.8 m deep) is less than 50%, indicating that the overburdened soil pressure is much larger than the excessive pore-water pressure, and therefore liquefaction does not occur. Fig. 3 also shows the ratio at a depth of 3.6 m at BH3 site, where the ratio of excessive pore-water pressure to effective confined pressure is less than 50%. This result illustrates that the liquefaction at BH3 site mainly occurs only for places where depth is larger than 3.6 m, which is comprehensive because the water table is at 3.1 m depth. To further understand the difference between liquefied and non-liquefied site response, Fig. 4 shows the comparison of ground response time histories from BH1 and BH3 sites for record HWA023.

From the starting point to 32 s, the two ground responses are nearly similar, but after the strongest ground shaking, the amplitude of the ground shaking from BH3 site is obviously smaller than that from BH1 site, and the vibration period at BH3 site increases. Note that the vibration at BH3 shows the characteristic of free vibration, like an oscillator, once liquefaction occurs. These characteristics shown in ground response time histories are significant for evaluating structural damage, and therefore further discussion is carried out in the following sections.



(a) BH1 (b) BH3

Fig. 2 Relationships of shear stress (σ_{13}) and strain (γ_{13}) (excitation is HWA023 record.)



(a) BH1 (b) BH3

Fig. 3 Ratio of excessive pore-water pressure to effective confined pressure against time (excitation is HWA023 record.)

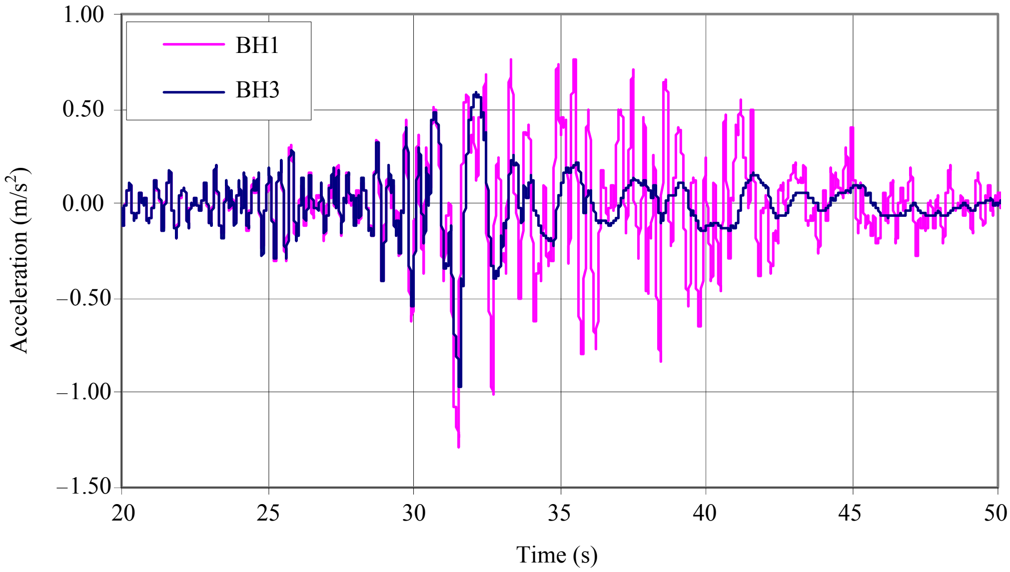
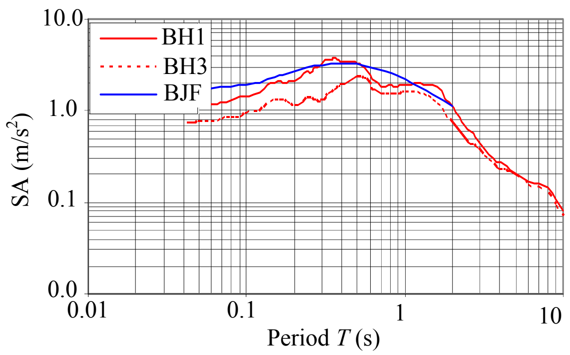


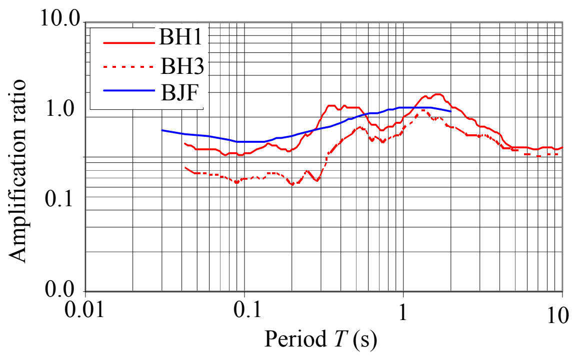
Fig. 4 Comparison of liquefied and non-liquefied ground responses at BH1 and BH3, excited by record HWA023

5. Liquefied and non-liquefied site response spectra

To demonstrate the characteristics of response spectra at liquefied and non-liquefied sites, the scaled earthquake records were used to excite BH1 and BH3 sites. In order to reduce the effect of record-to-record variation on ground response spectra, mean ground responses spectra were derived, as shown in Fig. 5(a), together with the estimated ground response spectrum of the BJF model for a magnitude 7.5 earthquake at a distance of 50 km. Fig. 5(a) shows that the spectrum of the BJF model better matches that at BH1 site at a period range of 0.1 s to 2.0 s, covering the natural period of the site. This is expected because site response at BH1 site only displays slightly nonlinear response as shown in Fig. 2(b). Compared with the spectra from the JBF model and BH1 site, the mean spectrum at BH3 site (liquefied site) is lower for short to moderate spectral periods of up to 1.3 s, and for longer spectral periods the difference becomes small. This is because the strong nonlinear soil response at BH3 site softens soil stiffness and results in site period lengthening. As a result, some waves with short and moderate periods are filtered out. In addition to this, strong nonlinear soil response dissipates the energy of input motions, such that spectral acceleration reduces. We note that the results in this study are similar to our previous work [12] in assessing nonlinear soil response at soft soil sites and the work of Rodriguez-Marek et al. [13] for short and moderate periods, but relative to normally nonlinear soil response of soft soil sites, the period range of spectral acceleration attenuation is larger for the liquefied site response and the amplitude of shear strain for the liquefied site is larger, such as is shown in Fig. 2.



(a) BJF model



(b) Amplification factor

Fig. 5 Comparison of ground response spectra from liquefied and non-liquefied sites

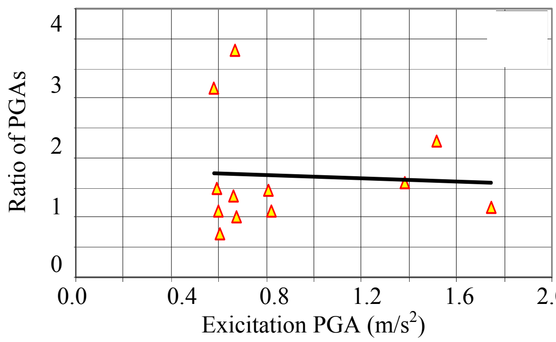
To further show the characteristics of response spectra, we calculated the amplification factor, i.e., the ratio of the spectral acceleration of ground response to that of excitation at a period, as shown in Fig. 5(b), together with the amplification factor curve of the JBF model. The amplification factors for BH1 site and the JBF spectrum are all larger than 1, indicating that the site amplifies the excitation, close to 1 at shorter and longer periods, and about 2 for periods from 0.3 s to 3.5 s. In contrast to BH1 site, the amplification factors at periods less than 0.32 s is less than 1 at BH3 site, indicating that deamplification occurs. This result also appears for soft soil site response. For example, Idriss [14] has shown that deamplification occurs when excitation PGA is within 0.3*g* to 0.4*g*. For the scaled records, all PGAs are less than 0.3*g*, but deamplification still occurs, implying that for liquefied sites deamplification may occur at lower excitation amplitude than those for soft soil sites. At moderate periods, amplification factors are larger than 1, even if at longer periods the amplification factor is still slightly larger than 1. This result shows that even for liquefied sites, site amplification for excitation cannot be neglected.

6. PGA at liquefied and non-liquefied sites

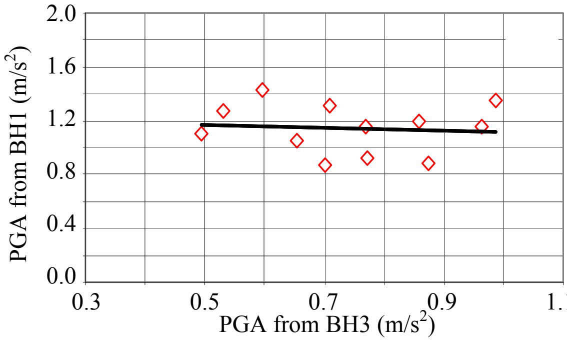
PGA is an important factor in assessing liquefaction potential, in that PGA is directly used as a representative of seismic loading. Due to lack of records on liquefied sites, earthquake records on near-liquefied sites are assumed to be similar to those on liquefied sites. Thus, it is acceptable to use the records of near-liquefied sites in assessing a probabilistic liquefaction analysis. In practice, however, this is not easily fulfilled because there are no enough strong motion records. An alternative is to use strong motion attenuation models to estimate PGA at one site. Nevertheless, in this case, it is very difficult to take account of the effect of nonlinear soil response, particularly for class E in NEHRP. Regardless of which of the two methods being selected to determine PGA on a liquefiable site, a question arises: how large is the difference between the estimated PGA and actual PGA for the site if liquefaction at the site occurs? Three cases exist: 1) $PGA_{NONL} > PGA_{LIQ}$, 2) $PGA_{NONL} = PGA_{LIQ}$, and $PGA_{NONL} < PGA_{LIQ}$ (subscript $_{NONL}$ denotes nonlinear site response and $_{LIQ}$ denotes that liquefaction occurs). Intuitively, the third case generally is impossible, so we only consider cases (1) and (2). The BH1 site is near the BH3 site and both have the same site conditions, Class D; therefore PGA estimated from the BH1 site is a better estimate at the BH3 site if liquefaction is not taken into account. The ratio of PGAs from BH1 and BH3 sites against excitation PGAs are plotted in Fig. 6(a), together with a thin line denoting mean value, which is derived through better fitting the ratios. Fig. 6(b) shows that PGAs from BH1 site are about 1.6–1.7 times as large as those from BH3 site on average. From the point view of the mean values for this case, the effect of excitation PGAs on the ratio of PGAs is negligible. This is because we scaled all records based on a range of around the natural periods of the sites. However, it is noted that the scatter of the ratios of PGAs is large, and thus more records are needed to further confirm the above results. As is well known, different scaling methods produce different scatters for different intensity-measure parameters and the scatter is also earthquake magnitude-distance-dependent, but this topic is not discussed here.

7. PGD and permanent displacements at liquefied and non-liquefied sites

Because of liquefaction, PGD is considered to be larger at liquefied sites than at non-liquefied sites. In this study, PGDs derived from BH1 and BH3 sites are plotted in Fig. 7, where it is not obvious that PGDs from BH3 site are larger than those from BH1 site. We also computed PGD mean values for BH1 and BH3 sites and found that the mean PGD from BH3 site is slightly larger than that from BH1 site, and the difference is almost of no significance in engineering practice.



(a) Ratios of PGAs from BH1 and BH3 against excitation PGA



(b) PGAs from BH1 and BH3

Fig. 6 Distribution of PGAs from BH1 and BH3

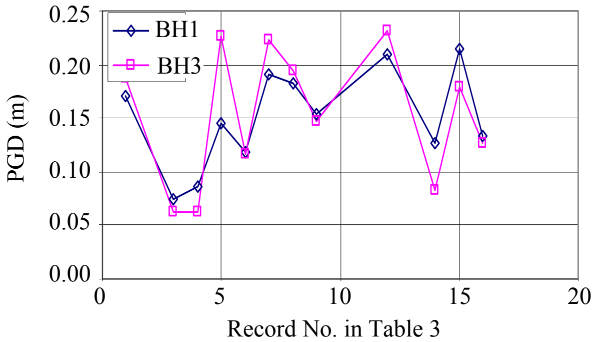


Fig. 7 Comparison of PGDs at BH1 and BH3 sites

Actually, this result should be expected because the top of layers for the sites is non-liquefiable soil. PGD is an important factor for judging the maximum transient displacement in design. Permanent displacement for post-earthquake is also an important factor for judging structural damage states. We plot the mean permanent displacements along the depth of BH1 and BH3 sites in Fig. 8.

On the ground surface, the permanent displacement from BH3 site is larger than that from BH1 site. At depths from 3.1 m to 6.7 m, where liquefaction occurs, the permanent displacements from BH3 site are much larger than those from BH1 site. This result illustrates that liquefaction does lead to large permanent displacements in liquefied layers. Even if the top of the soil layer does not liquefy (for example, it is not liquefiable soil or water table is below the layer, such as in this case), the ground permanent displacement for liquefied site is larger than that for non-liquefied sites due to the motivation from liquefied layers located below. Because large permanent displacement can result in foundation failure, the significant increase in the permanent displacement at liquefied sites should be given attention.

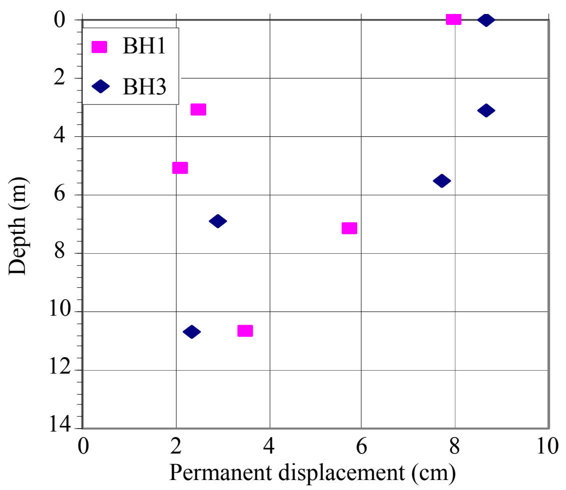


Fig. 8 Comparison of mean permanent displacements from BH1 and BH3 sites along depth

8. Conclusions

To investigate the characteristics of liquefied and non-liquefied site responses, effective stress method and simplified soil bounding surface model were used to analyze ground responses at two sites, which are close to each other and have similar natural period and one for liquefiable site and the other for non-liquefiable site, and then a suite of scaled earthquake records were used to excite the two sites. From the analyses, some conclusions have been arrived at.

(1) Normally seismic response spectra are estimated from strong motion attenuation models. For the two selected sites, with similar site class and subjected to the same ground shaking, the seismic response spectra estimated from strong motion attenuation model would be the same. However, because of liquefaction at BH3 site, actually the mean response spectrum on the ground surface at BH3 site is lower than that from the BJF model at periods of about up to 1.3 s and deamplification occurs for periods up to 0.32 s. In contrast to the mean spectrum at BH3 site, the mean response spectrum on the ground surface at BH1 site (non-liquefied site) is similar to that of BJF model. At longer periods, the response spectra at BH1 and BH3 sites are similar. Mean PGA at non-liquefied site BH1 is about 1.6–1.7 times as large as that at liquefied site BH3. The lower mean PGA at BH3 site is mainly due to liquefaction at the loose sand layer, rather than the slight different deposits at the two sites. The reason for spectra and PGA at BH3 site being lower than those at BH1 site is that liquefaction softens soil stiffness and increases hysteretic damping, which leads to high or moderate frequency waves being filtered. Normally, in assessing liquefaction potential, resistant cyclic stress (RCS) is assessed by using PGAs from non-liquefied sites. The results obtained show that RCS may have been overestimated for liquefied sites.

(2) PGDs, indicating the transient displacement on ground surface, for sites BH1 and BH3 have been estimated. The results show that the difference of the mean PGDs at the two sites are slight, such that the difference is not of significance in engineering practice. The result is comprehensive, because PGD lasts for long periods, where response spectra for BH1 and BH3 sites are similar.

(3) We further investigated the permanent displacement for post-earthquake at the two sites along the depths of BH1 and BH3 sites. At the liquefied layer (loose sand layer), the permanent displacements at the liquefied site BH3 are quite larger than those at the non-liquefied site BH1. On the ground surface, the permanent displacement at BH3 is also larger than that at BH1. This result is easily understood, because liquefaction causes larger permanent displacement. This result is also consistent with those

field investigations for earthquakes, when building foundations were damaged by liquefaction.

Acknowledgments

This research was supported by the National Natural Science Foundation of China (No. 41030742) and Technology Research of Railway Ministry (No. 2009G010-C).

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(Editor: Yao ZHOU)