One-dimensional Site Response Analysis and Liquefaction Evaluation of Can Tho City, Vietnam

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Abstract-Can Tho is the biggest city in the Mekong River Delta, one of the five national central cities in Vietnam. However, it has not been studied regarding seismic hazard estimation. For this purpose, one-dimensional nonlinear site response analysis of this city was performed in this paper. The measured in-situ profiles and corresponding geotechnical site investigation and laboratory test data were utilized to develop the site model for site-specific ground response analysis. A suite of earthquake records compatible with the Vietnam rock design spectrum (TCVN 9386:2012) was used as input ground motions at the bedrock. The results show that Peak Ground Acceleration (PGA) increases from the bedrock to the surface. Maximum PGA is 0.083g for O Mon district (P1) and 0.073g for Cai Rang district (P2). The maximum shear strain is reported to be 0.35% for P1 and 0.45% for P2. The recommended amplification factors are 1.7 for P1 and 1.9 for P2. Even though Can Tho city is composed of soft layers, liquefaction is unlikely to occur.

Keywords-response spectra; one-dimensional nonlinear site response analysis; peak ground acceleration; shear strain; liquefaction assessment

I. INTRODUCTION

Seismic hazard assessment plays an important role in the sustainable development of urban infrastructure. Site response analysis and evaluating soil liquefaction potential is commonly used for that purpose. A total of 1645 earthquakes recorded in Vietnam with magnitude (M_L) of 3.0 or higher were considered in [1, 2]. In the 20th century, up to 90% of earthquakes occurred in northern Vietnam because that area lies on Indochina and South China faults [3]. The studies on seismic effects in southern Vietnam are almost non-existent. In recent years, several earthquakes have been recorded in south Vietnam, such as the earthquake that occurred on the coast of Vung Tau province in 2005, with M_L = 4.5, shaking the whole Ho Chi Minh City, the earthquake at the coast of Binh Thuan province in 2020, with M_L = 4.7. These earthquakes are related to the Thuan Hai – Minh Hai fault, which has a closet distance of about 45km from Can Tho city. There is a need for studies that assess the effects of earthquakes on the southern region of Vietnam, especially in large cities with dense population.

Regarding the central of Can Tho city, and to the best of our knowledge, no studies have been conducted on one-dimensional site response analysis and evaluation of ground behavior under earthquake effects. Can Tho is one of the 5 biggest cities in Vietnam, with nearly 1.3 million residents (2019). Can Tho is also the capital of the Southwest region and the economic center of the Cuu Long River Delta. The stratigraphy is mainly clay and sandy clay with large thickness, interspersed with fine sand layers with a thin thickness and deep. The groundwater level is shallow and changes seasonally due to the interlaced system of canals. This study aims to perform site response analysis and then propose design spectra for the ground in the area of Can Tho city. Two site classes, C and D, and 15 earthquakes, were adopted in this research. Also, the liquefaction assessment of this city is evaluated.

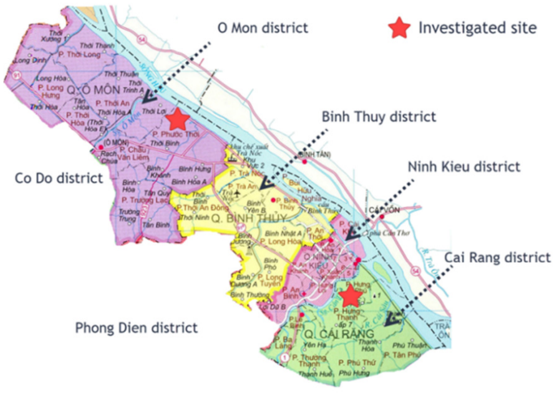

Fig. 1. Central of Can Tho city and investigated sites.

II. INVESTIGATED SOIL PROFILES

Figure 1 shows the two investigated soil profiles of Can Tho city considered in this study. These profiles represent site class C (O Mon district, named P1) and D (Cai Rang district, named P2) according to the Vietnam code [4]. The typical soil stratigraphy includes fill, clay or sandy clay/clayey sand, and gravel. The groundwater level is 0.6m and 2.2m depth for the soil profiles of P1 and P2. Details of the soil properties and shear wave velocity is presented in Tables I-II and Figure 2.

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TABLE I. SOIL PROPERTIES OF P1

No	Material type	Thickness (m)	Depth (m)	Density (kg/m^3)	Shear wave velocity (m/s)
1	Fill	1.3	1.3	1500	100
2	Sandy clay	5.1	6.4	1570	100
3	Clay	6.2	12.6	1940	214
4	Clay	4.8	17.4	1940	219
5	Clayey sand	5.3	22.7	2000	259
6	Clay	3	25.7	2000	263

TABLE II. SOIL PROPERTIES OF P2

No	Material type	Thickness (m)	Depth (m)	Density (kg/m^3)	Shear wave velocity (m/s)
1	Fill	1.2	1.2	1500	100
2	Sandy clay	1.4	2.6	1680	100
3	Clay	6.8	9.4	1650	100
4	Clay	6.5	15.9	1790	141
5	Sandy clay	7.8	23.7	1720	209
6	Clay	5.7	29.4	1950	214

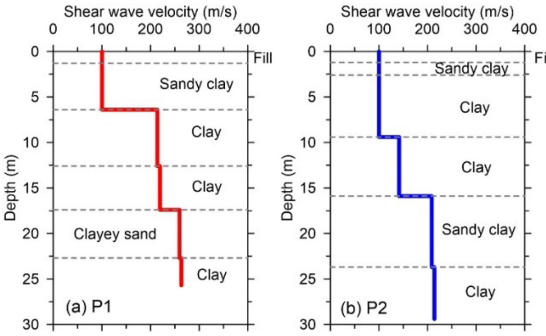

Fig. 2. Shear wave velocity profiles.

III. INPUT GROUND MOTIONS

The critical unknown in site response analysis is earthquake ground excitation. Due to the lack of available earthquake records in Can Tho city, ground motions recorded in other regions, but with similar characteristics to those likely to occur in Vietnam are selected. The recorded ground motions were selected from the NGA-west2 database (https://ngawest2.berkeley.edu/). The ground motions with the following features were selected:

- Range of moment magnitudes M_L 5.5-7.0.
- Rupture distance R_{rup}: 0.92-85.17km.
- Time-averaged shear-wave velocity to a depth of 30m (V_{S30}): 760-1500m/s.

A total of 15 motions with Peak Ground Acceleration (PGA) ranging from 0.04g to 0.25g were selected from 12 earthquake events, as shown in Table III. The ground motions were scaled to the representative shaking intensity levels of each analyzed district, in which PGA levels at P1, and P2 were scaled to 0.0546g, and 0.0515g respectively. Also, the mean response spectrum of selected ground motions was matched closely to the spectrum of site class A of the Vietnam design code (MoC, 2012), as shown in Figure 3. These motions were also used in [1, 2].

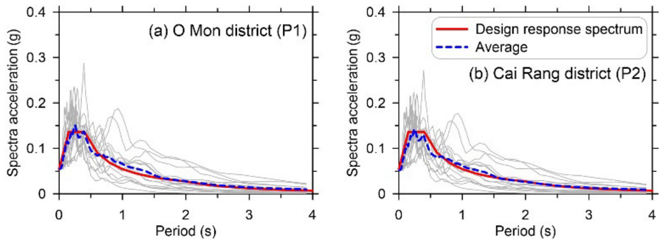

Fig. 3. Input motions and design response spectra (a) P1, (b) P2.

TABLE III. SELECTED EARTHQUAKE EVENTS [1, 2]

Event	Year	Station	M_w	Mechanism	R_{rup} (km)	V_{S30} (m/s)
San Fernando	1971	Pasadena - Old Seismo Lab	6.61	Reverse	21.5	969.07
Whittier Narrows-01	1987	Pasadena - CIT Kresge Lab	5.99	Reverse	18.12	969.07
Loma Prieta	1989	Piedmont Jr High School Grounds	6.93	Reverse oblique	73	895.36
Loma Prieta	1989	Point Bonita	6.93	Reverse oblique	83.45	1315.92
Loma Prieta	1989	SF - Pacific Heights	6.93	Reverse oblique	75.96	1249.86
Loma Prieta	1989	So. San Francisco_ Sierra Pt.	6.93	Reverse oblique	63.15	1020.62
Northridge-01	1994	LA - Wonderland Ave	6.99	Reverse	20.29	1222.52
Northridge-01	1994	Vasquez Rocks Park	6.99	Reverse	23.64	996.43
Chi-Chi_ Taiwan-05	1999	TTN042	6.20	Reverse	85.17	845.34
Umbria-03_ Italy	1984	Gubbio	5.60	Normal	15.72	922
Kobe_ Japan	1995	Kobe University	6.90	Strike slip	0.92	1043
Chi-Chi_ Taiwan-05	1999	HWA002	6.20	Reverse	45.03	789.18

IV. ONE-DIMENSIONAL ANALYSIS

One-dimensional (1D) nonlinear analysis was performed using DEEPSOIL v7.0 [5-9]. The most widely used pressure-dependent hyperbolic, the Modified Kodner–Zelasko (MKZ) model [10], implemented in DEEPSOIL, was used. Darendeli model [11] was employed to generate the dynamic properties of soil layers. The variables required to generate the nonlinear curves for each layer are the coefficient of lateral earth pressure (K_o), Plasticity Index (PI), number and frequency of cycles (N), loading frequency (f), and the Over Consolidation Ratio (OCR). Due to the unavailability of site-specific index properties, PI and OCR were taken as 0 and 1 respectively. N and f were set to 10 and 1 as recommended in [11], whereas K_o was specified as 0.5. Figure 4 presents a flowchart of the site response and liquefaction evaluation procedure implemented in this study. The flowchart illustrates that a suite of nonlinear site response analyses are used to determine the site-specific spectral shapes, PGA, and maximum shear strain profiles of the P1 and P2 areas. Moreover, the proposed response spectra resulting from this study provide insights into the discrepancies between the design and site-specific spectra. The Cyclic Stress

Ratio (CSR) was also obtained from 1D site response analysis and was used for the liquefaction evaluation purpose.

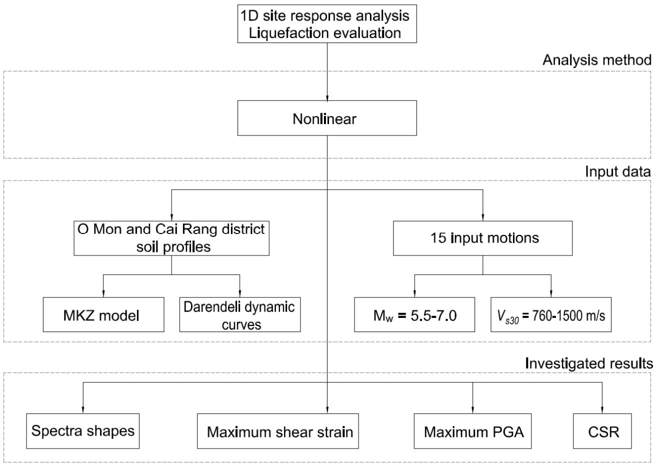

Fig. 4. Response spectra comparison of the surface, input motion, and bedrock design.

V. RESULTS AND DISCUSSION

Figure 5(a) shows the PGA along with the depth of P1 and P2 soil profiles. PGA increases from the bedrock to the surface except at the 2.5 – 7.5m depth. The decrease at that depth can be attributed to a significant thickness of the soft soil. Surface PGAs are 0.083g at P1 and 0.073g at P2. Shear strain profiles are shown in Figure 5(b). The shear strain of P1 and P2 reached 0.35% and 0.45% respectively. The maximum value of shear strain occurs at the approximate position of the boundary between the soil layers with a significant shear wave velocity difference.

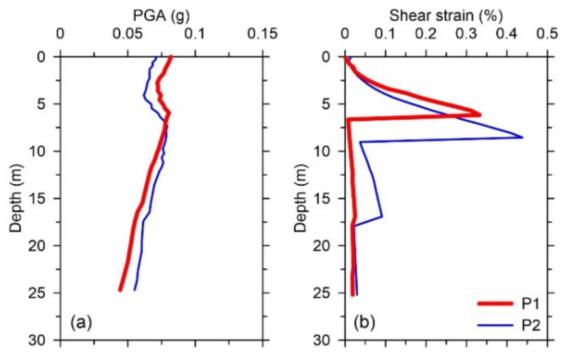

Fig. 5. PGA and shear strain profiles.

Figure 6 presents the surface response spectra for P1 and P2. The average response spectrum of the input earthquakes and the design spectrum according to the Vietnamese code (bedrock) are also shown. The maximum values of spectra acceleration of P1 and P2 are at periods of 0.4s and 0.3s respectively. Figure 7 depicts the proposed surface response spectra against the calculated and design ones. The proposed surface response spectra were computed based on [12]:

$$S = \frac{I_{soil}}{I_{rock}} \quad (1)$$

where S is the soil factor and I_{soil} and I_{rock} are the spectral acceleration index of soil and rock respectively:

$$I_{soil\ or\ rock} = \int_{0.05}^{2.5} Sa(T) \quad (2)$$

S was 1.7 and 1.9 for P1 (site class C) and P2 (site class D). In the Vietnam design code [4], these values are 1.15 and 1.35 for site class C and site class D.

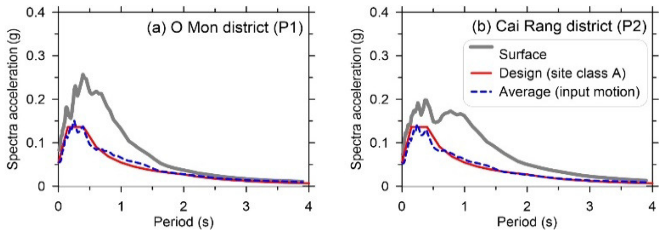

Fig. 6. Response spectra comparison of the surface, input motion, and bedrock design.

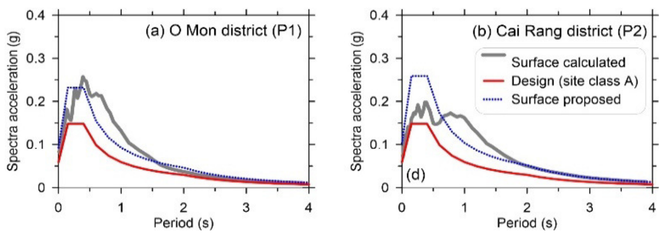

Fig. 7. Response spectra comparison of the calculated surface, proposed surface, and bedrock design.

In this study, the empirical correlation proposed in [13] is used to assess the liquefaction potential of the soil, which relates V_S to CCR as follows:

$$\mathrm{CRR} = \left\{ a\left(\frac{K_c V_{S1}}{100}\right)^2 + b\left(\frac{1}{V_{S1}^* - K_c V_{S1}} - \frac{1}{V_{S1}^*}\right)\right\} \mathrm{MSF} \quad (3)$$

where a = 0.022, b = 2.8, K_C = correction factor for high value of V_{S1} caused by cementation, MSF is the Magnitude Scaling Factor, and $V_{S1} = V_S$ corrected for the effect of overburden pressure. They are determined as follows:

$$\mathrm{MSF} = \left(\frac{M_w}{7.5}\right)^{-2.56} \quad (4)$$

$$V_{S1} = V_S \left(\frac{P_a}{\sigma_v'}\right)^{0.25} \quad (5)$$

where M_w is the moment magnitude, P_a is the atmospheric pressure, σ_v' is the effective overburden stress (kPa), and V_{S1}^* is the critical value of V_{S1} that separates the contractive and dilative behavior of soil. The recommended V_{S1}^* value is dependent on the Fines Content (FC) as follows:

- V_{S1}^* = 215m/s for sand and gravels with FC ≤ 5%.
- V_{S1}^* = 215 - 0.5(FC - 5)m/s for sand and gravel with 5% < FC < 35%.
- V_{S1}^* = 200m/s for sand and gravels with FC ≥ 35%

For a conservative estimate, FC ≤ 5% and $V_{S1}^* = 215$m/s are used. V_{S1} based on the measured V_S profiles are shown in Figure 8. Liquefaction potential assessment for Can Tho city area was performed utilizing the maximum CSR calculated from the nonlinear site response analyses presented in Section IV and the CSR presented in this section.

Because not the entire soil profile is susceptible to liquefaction, liquefaction potential was assessed only for the fill, sandy clay layer with $V_{S1}^* < 215$m/s. The averaged results from the input motions are shown in Figure 9. The liquefaction assessment shows that eventhough the soil profiles at Can Tho city are composed of soft layers, liquefaction is unlikely to occur due to the low seismic hazard of the region. It should also be noted that the assessment is based on the significantly conservative estimate that even layers with clay content are susceptible to liquefaction.

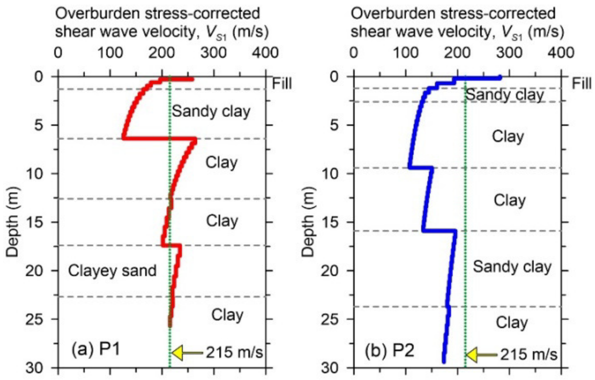

Fig. 8. Overburden stress-corrected shear wave velocity V_{S1} profiles.

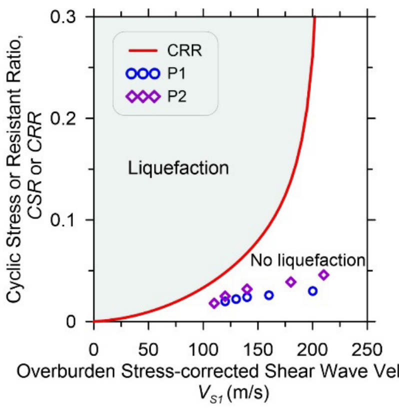

Fig. 9. Overburden stress-corrected shear wave velocity V_{S1} profiles.

VI. CONCLUSIONS

This study evaluates the behavior of the ground under earthquake loading at the center of Can Tho city, Vietnam. A total of 15 recorded input motions and two investigated soil profiles were considered and one-dimensional nonlinear ground analysis and liquefaction potential assessment were performed. The following conclusions were drawn:

- PGA increases from the bedrock to the surface. The maximum values of PGA are 0.083g for P1 (site class C) and 0.073g for P2 (site class D).
- The maximum shear strain occurs at the boundary between two soil layers with a large difference in shear wave velocities. The maximum values of the shear strain are 0.35% for P1 and 0.45% for P2.
- Design response spectra are proposed in this study area. The recommended soil factor is 1.7 for P1 and 1.9 for P2.
- The liquefaction potential is assessed using the empirical liquefaction triggering chart based on the in-situ measured V_S. The results demonstrated that there is no liquefaction susceptibility in Can Tho city.

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