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Relationship between earthquake-induced uplift of rectangular underground structures and the excess pore water pressure ratio in saturated sandy soils

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ABSTRACT

The excess pore water pressure (EPWP) ratio is one of the contributing factors to the earthquake-induced uplift displacement of underground structures. To investigate its role in the incompletely liquefied soil, a rectangular underground structure buried at different depths in saturated sandy soils with different densities and under different earthquake loadings was analyzed by a numerical method. The results revealed that the uplift response was related to both the EPWP ratio and its duration, as well as the region of the liquefiable soil under the underground structure. The value of the EPWP ratio that triggers uplift fell in a range rather than remaining constant.

1. Introduction

Earthquake-induced soil liquefaction may result in uplift damage to underground structures, such as sewer pipes, manholes, and tunnels, as has been found in some previous earthquakes, e.g., the 1964 Niigata Earthquake (Matsumoto, 1968), the 1989 Loma Prieta Earthquake (Schmidt and Hashash, 1998), the 1995 Hyogo-ken-Nanbu Earthquake (Shinozuka, 1995), the 1999 Taiwan Earthquake (Tsai et al., 2000), the 1999 Turkey Earthquake (O'Rourke et al., 2001), the 2004 Niigata-ken Chuetsu Earthquake (Yasuda and Kiku, 2006), and the 2011 Tohoku Earthquake off the Pacific Coast (Chian and Tokimatso, 2011), among others. There have been extensive studies by numerical analyses (Zhou et al., 2014), shaking table tests (Watanabe et al., 2016) and centrifuge tests (Chou et al., 2011; Huang et al., 2015; Lee et al., 2017) concerning the uplift behavior and its mechanism for underground structures due to earthquake-induced soil liquefaction. Previous studies identified four mechanisms that might contribute to the uplift: (1) ratcheting of underground structures, (2) pore water migration towards the base of underground structures, (3) bottom heave due to the shear of non-liquefiable soil layers below underground structures, and (4) shear flow deformation of the liquefied soil.

Even though the build-up of excess pore water pressure (EPWP) during earthquakes is not listed as one of the four mechanisms, it was considered as the dominant factor for its triggering (Tokida and Ninomiya, 1992; Koseki et al., 1997; Chian et al., 2014; Zhou et al., 2014; Watanabe et al., 2016). Tokida and Ninomiya (1992) observed in shaking table tests that when the excess pore water pressure ratio underneath the center of the buried rectangular structure increased to 0.8–1.0, the structure began to uplift rapidly. Zhou et al. (2014) indicated that the major cause for uplift response of the shallow buried rectangular structure was an accumulation of pore pressure at the bottom of the structure in both the physical and numerical models, and the corresponding micro-scale responses of a preferred vertical orientation of particle long axis, a sudden decrease in the number of contacts per particle, and a sudden increase in porosity were mainly caused by the build-up and dissipation of excess pore pressure. Additionally, uplift responses of Pipes or manholes were also studied (Tsai et al., 2000; Yasuda and Kiku, 2006; Cheuk et al., 2008; Kang et al., 2013a,b; Chian et al., 2015; Huang et al., 2015). Kang et al. (2013b) investigated more than 1450 damaged sever popes and manholes induced by the uplift after the 2004 earthquake in Niigata-ken Chuetsu, Japan, it was found that the excess pore water pressure at the bottom of the structure was less than the initial vertical effective stress, whereas the corresponding residual uplift displacement was about 0.95 m for the centrifuge modeling. Huang et al. (2015) analyzed the relationship between the maximum uplift displacement of a model pipe and the maximum EPWP ratio in their centrifuge tests and found that the uplift of the pipe occurred when the maximum EPWP ratio (the ratio of excess pore water pressure to initial effective stress of soils) exceeded a threshold value that was related to the pipe burial depth.

In particular, Koseki et al. (1997) investigated the uplift mechanism for a variety of underground structures and discussed the relationship between the uplift displacement and the factor of safety against uplift (F_s) and concluded that uplift occurred when F_s was close to 1.0. As

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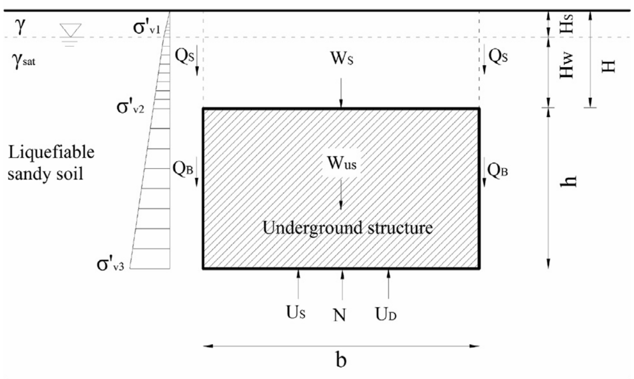

Fig. 1. External forces acting on the underground structure during uplift.

shown in Fig. 1, the F_s for rectangular structures can be calculated as follows (Koseki et al. 1997):

$$F_s = \frac{W_{US} + W_S + 2(Q_S + Q_B)}{U_S + U_D} \tag{1}$$

where W_{US} is the weight of the underground structure, $W_{US} = \gamma_{US}bh$, with γ_{US} being the equivalent unit weight of the structure; W_S is the overburden effective weight of the soil acting on the top of the structure, $W_S = \gamma bH_S + \gamma_s' bH_W$, with γ_s' being the submerged unit weight of the soil below groundwater table and γ being the natural unit weight of the soil above the groundwater table; Q_S is the shear resistance of the soil above the structure, $Q_S = \int_0^{\sigma_{v1}'} \mathrm{K}_0 \tan\varphi dH_S + \int_{\sigma_{v1}'}^{\sigma_{v2}'} (1 - r_{u1})\mathrm{K}_0 \tan\varphi dH_W$; Q_B is the friction resistance at the sides of the structure, $Q_B = \int_{\sigma_{v2}'}^{\sigma_{v3}'} (1{-}r_{u2})K_0 \tan\delta dh$; $\sigma_{v1}' = \gamma H_s$; $\sigma_{v2}' = \gamma H_S + \gamma_s' H_W$ and $\sigma_{v3}' = \gamma H_S + \gamma_s'(H_W + h)$ are the static effective vertical stresses of the soil at the depth of the groundwater table at the top and bottom of the structure, respectively; and r_{u1} and r_{u2} represent the EPWP ratio of the soil above and at the sides of the underground structure, respectively. It is noted that Q_S and Q_B would gradually decrease due to the decrease of the effective stress caused by the build-up of EPWP. K_0 is the coefficient of earth pressure for the rest of the soil, φ is the internal friction angle of the soil, and δ is the interface friction angle between the soil and the structure; U_S is the buoyant force of the underground structure caused by the static water head in the saturated soil, $U_S = \gamma_w bh$; U_D is the uplift force at the bottom of the underground structure caused by the EPWP of the saturated soil; and $U_D = \Delta ub$, in which Δu is the difference between the EPWPs at the bottom and the top of the underground structure, and the EPWP was measured by average of EPWPs of soil elements arranged at the bottom or the top of the structure. r_u is defined as the ratio between the EPWP of the soil and the static effective vertical stress of the soil, $r_u = EPWP/\sigma_{v0}' = 1{-}\sigma_{vt}'/\sigma_{v0}'$. The external forces of the underground structure are shown in Fig. 1. In addition, the equilibrium formula of forces acting on a manhole refers to Tobita et al. (2011).

Most previous studies on the uplift response of underground structures were focused on the completely liquefied soil. However, before the construction of large underground structures, if it is found from site evaluation that the saturated sandy soil may liquefy under the design earthquake, the soil must be treated with anti-liquefaction measures. Such measures are usually not required if the site can pass the liquefaction assessment. The factor of safety against uplift may also be checked using the simplistic method shown in Fig. 1, and if the factor of safety is larger than some designated value, the structure is considered safe against uplift.

Now two critical questions still exist. The first question is what occurs if the soil is dense or medium-dense and it does not liquefy under the design earthquake loading. The second question is whether the EPWP ratio and the simplistic method shown in Fig. 1 are adequate for analyzing the triggering of uplift. Concerning the second question,

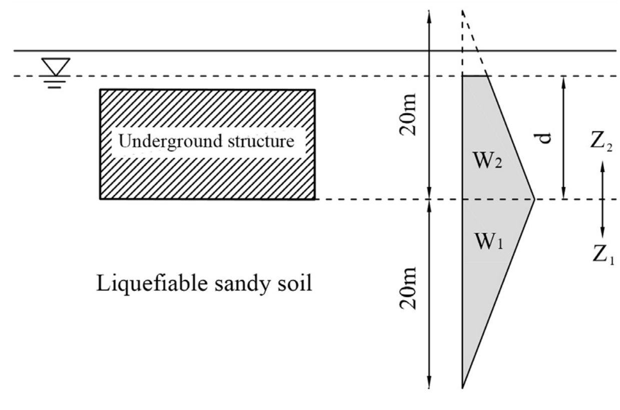

Fig. 2. Weighting function for the liquefaction index for uplifting (P_L'). Reproduced from Watanabe et al. (2016).

Watanabe et al. (2016) proposed a new method to evaluate the stability of open-cut tunnels against uplift by analyzing the relationship between the uplift displacement and the liquefaction index for uplifting (P_L'). The liquefaction index for uplifting (P_L') (Watanabe et al. 2016) can be calculated as follows:

$$P_L' = \int_0^{20} (1{-}F_L)(10{-}0.5z_1)dz_1 + \int_0^{d} (1{-}F_L)(10{-}0.5z_2)dz_2 \tag{2}$$

where Z_1 and Z_2 are the distances measured, respectively, downward and upward from the level of the base of the underground structure (Fig. 2); d is the thickness of saturated soil from underground water table to the bottom of the underground structure, where $0 < d \leqslant 20m$; and F_L is the liquefaction resistance ratio that can be obtained from the liquefaction strength ratio of the soil to the shear stress ratio of an earthquake. It is a function of depth. When $F_L < 1$, the soil is liquefied, whereas $F_L = 1$ if the soil is not liquefied during the earthquake. Here, the liquefaction strength ratio can be calculated by standard-penetration-test (SPT) blow counts or the liquefaction strength curve of the soil (Japan Rail Association method, 1996).

Watanabe et al. (2016) suggested that if the value of P_L' was less than 20, the underground structure would not uplift, even though F_s was less than 1.0. However, this conclusion was based on reduced-scale shaking table tests with idealized sinusoidal shaking, while the seismic response of the structure and the excess-pore-water build-up are largely influenced by scale effect, its applicability to large underground structures under realistic seismic loading deserves further investigation.

In this study, in order to investigate the uplift behavior of the rectangular underground structure in incompletely liquefied soil, to identify the triggering condition and mechanisms of the uplift and to understand the relationship between EPWP ratio and the vertical displacement of the structure, a finite element and finite difference (FE-FD) coupled method (Oka, 1992; Oka et al., 2004) was employed to simulate the response of a two-story subway station buried at different depths in saturated sandy ground, with different relative density, and under various earthquake loadings. Furthermore, the triggering conditions for uplift obtained from the numerical analyses and by Eqs. (1) and (2) were compared.

2. Numerical method

2.1. Discretization of dynamic equations

A soil-water coupled problem in a saturated condition was formulated based on the *u-p* (displacement of the solid phase-pore water pressure) formulation (Zienkiewicz and Bettes, 1982), which has been shown to be effective for two-dimensional earthquake analyses. An FD-FE method was first proposed by Akai and Tamura (1978) to discretize