EFFECT OF THE PAVEMENT ROADBED THICKNESS ON THE EARTHQUAKE INDUCED SETTLEMENT FOR NON-LIQUIFIABLE CASE

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Introduction:

The 2011 Great East Japan Earthquake caused severe damages in Tokyo Bay area. A Light Detection and Ranging (LIDAR) which consists of (i). laser sensor (ii) GPS and (iii) Inertial Measurement Unit (IMU, was used to collect the elevation data before and after the earthquake 2011 (Konagai et al., 2013).The subsidence map was prepared by comparing the Digital Surface Models (DSM).Liquefaction induced road subsidence extracted from the DSMs showed that the under the same liquefaction potential, liquefaction induced settlement in structural-type roads, typically design for heavy traffic, were significantly less than that occurred in ordinary-type roads in residential areas (Fig.1a) (Kajihara et al.,2015). Therefore, the effect of roadbed thickness on liquefaction induced settlement for liquefiable soils is also need to study. To do so, a series of 1-g shaking table tests on models were conducted to understand the effect of pavement road-bed thickness and their induced settlements due to the dynamic shaking. The results show that the seismic induced settlements of thick pavements are comparatively lesser than thin pavements. This information will be helpful to understand the liquefaction induced subsidence/mechanism of roads which has the different road bed thickness.

Experimental Methodology: Two 1-g model tests were conducted at 1/20 scale. The scale down of the models followed a similitude law (Iai, S., 1989). In the test, (i) a thin pavement (thin=3cm) and (ii) a thick pavement (thick=6cm) were used. A one-directional laminar soil box (100cm x40cm x70cm) which contained the liquefiable sand and pavement models was mounted on the shaking table base. The laminar box was made up of the 17 laminae placed on each other and their movement was guided via one directional bearings and restrained in perpendicular direction. A thick (3mm) membrane was installed inside the laminar box (Fig. 1a). Dry Silica Sand 5 (Gs=2.638, γ_{max}=15.493kN/m^3, γ_{min}=12.748kN/m^3, D_{50}=0.64mm) was dropped at the predefined height by air dry pluviation method to produce the specific relative density (Dr=47%), later the sand was saturated by injecting water into the laminar box before the shaking tests. The pavement model (21kN/m^3) was made by mixing ordinary Portland cement, Kaolin clay and Silica Sand 5 (ratio 1:2:8). Accelerometers (A1-9), pore water pressure meters (PW2-9) and laser sensors (LS1-3) were placed at relevant locations to monitor the behaviour of the model (Fig. 1b). Successive seismic sinusoidal loading of f=10 Hz, t=3s, cycles=30 and amplitude from100-900 gal were selected.

Figure 1: (a) Suyama et al., 2016(b) Laminar soil box (c).Sensors location (all dimensions in mm)

Results and Discussions: Excess pore water pressure (u) recorded by means of pressure meters at different levels. Seismic induced shaking caused the excess pore water pressure build up. Excess pore water pressure ratio (u/σ'$_v$) at A2 location is not reaching to 1 for both the cases, hence the soil was not liquefied. Further, the pore water pressure build up beneath the thin pavement is little higher as compare to the thick pavement (Fig.2 a).

Settlements of the pavements measured manually, shows that the thick pavement experienced less settlement as compare to thin pavement (Fig. 2b). This mainly due to the reason that the rate of shear strength degradation in the former case is lower in contrast with the latter. This can be shown by the history of the stress-strain curves in Fig. 2c at the A2 level.

Stress and strains were back-calculated from the accelerometer readings following the method for downhole array (Zeghal et al., 1995) according to Eq. (1) and Eq. (2). The obtained acceleration were corrected by linear baseline correction and filtered by the forth order Butterworth filter between the band pass of 3Hz and 25Hz keeping the actual nature of response. The linear baseline correction was done by SeismoSignal following least square fit method (regression analysis) with 3rd degree polynomial curve that best fits the time-acceleration pairs of values. The objective of baseline correction is to remove the false baseline trends that is well noticeable in the displacement time history, obtained by double integration of uncorrected acceleration.

$$\tau_i(t) = \sum_{k=1}^{i-1} \rho \frac{\ddot{u}_k + \ddot{u}_{k+1}}{2} \Delta z_k \ , i = 2, 3, \ldots, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

$$\gamma_i(t) = \frac{1}{\Delta z_{i-1} - \Delta z_i} \left[(u_{i+1} - u_i) \frac{\Delta z_{i-1}}{\Delta z_i} + (u_i - u_{i-1}) \frac{\Delta z_i}{\Delta z_{i-1}} \right] \ldots\ldots\ldots\ldots \text{(2)}$$

where subscript i refers to level z_i; $\tau_i = \tau(z_i, t)$; $\ddot{u}_i = \ddot{u}(z_i, t)$; and Δz_k is the soil slice thickness in Eq. (1) and $u_i = u(z_i, t)$ is the absolute displacement at the level z_i in Eq. (2).

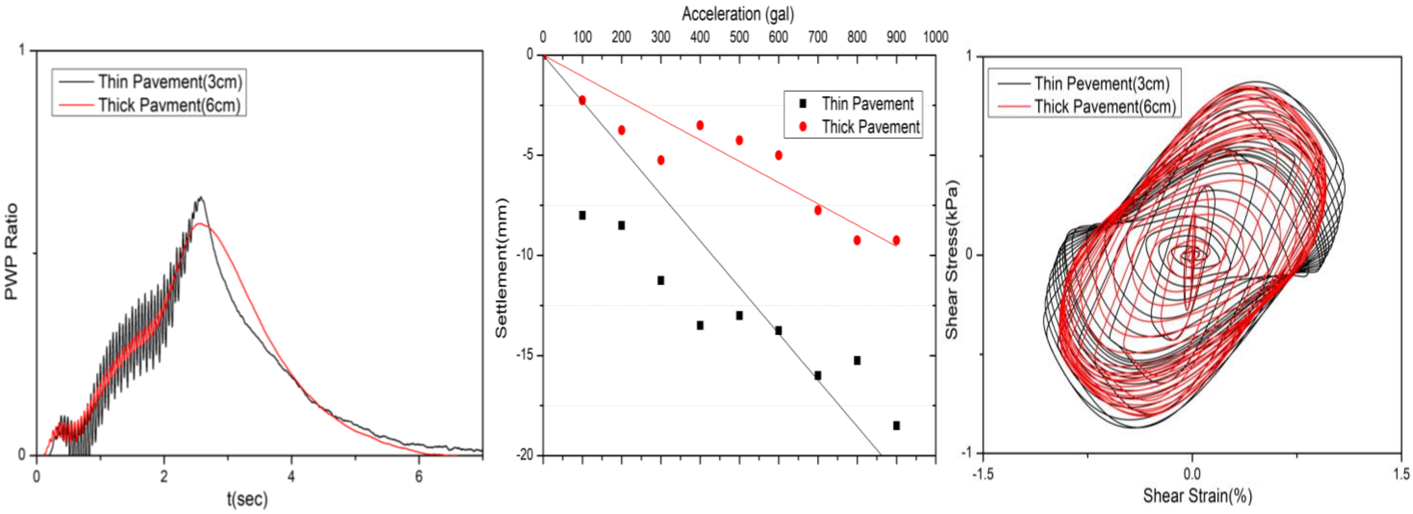

Fig 2. (a).PWP ratio (b). Settlement curves (c). Stress strain histories

The shear stress-strain hysteresis at A2 level shows that the sand softening behaviour increases with an increase in both the number of loading cycles and acceleration amplitude. The shear strength degradation rate is higher under the thin pavement as compared to the thick one (Fig. 2c). Additionally, effective vertical stress for both the cases is not approaching to zero, hence in both cases there is no signs of liquefaction. The stiffness of thin pavement compromised during the shaking, results the development of more shear strain and excess pore water pressure beneath the thin pavement as comparison to the thick pavement.

Conclusion: Two 1-g shaking table tests at 1/20th scale were performed overlaid by the two pavement model (thin pavement=3cm, thick pavement=6cm). It was noted that the shaking induced settlements is lesser for the thick pavement as compared with the thinner pavement. The less stiffness of the thin pavement contributed to develop the shear strains and pore water pressure under the pavement.

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