Practical application of mitigation measures for existing underground lifelines subjected to liquefaction

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ABSTRACT

The 2011 Tohoku Earthquake at the Pacific coast of Japan caused significant liquefaction-induced damage around Tokyo Bay where floated pipelines and manholes were widely observed. To protect existing underground lifelines from future earthquakes in some other regions, mitigation measures are urgently required. The conventional measures require considerable time and economic resources, which hinders the feasibility of these methods. In this regard, the authors previously proposed new mitigation measures: the vertical drain pipe method, connection with pavement by stiff bars (namely, horn structure method) and the chemical grouting method. The proposed measures can be executed with a partial excavation of backfill soil, resulting in less construction period and cost. Satisfactory performances of the proposed measures were demonstrated through 1-g shaking model tests by the authors. The present study aims to develop a framework for practical application with the proposed measures. To evaluate the performance of the proposed measures, challenges of developing a simple analytical method based on simple beam models were discussed. The proposed analytical models qualitatively captured the experimental results.

Keywords: earthquake, liquefaction, lifelines, mitigation, design

1 INTRODUCTION

Underground lifelines such as sewer pipes are vulnerable to liquefaction hazard. Floatation of a buried pipe takes place when the buoyancy force generated in liquefied subsoil exceeds the gravity force acting on the structure. Koseki et al. (1997) developed a factor of safety for floating of an embedded pipe. The conventional mitigation measures such as densification of backfill soil, use of aggregate materials, and cementation of sandy soil were suggested after the Niigata Chuetsu Earthquake in 2004 (Technical Committee on Earthquake Resistant Design of Sewage Lifelines, 2004). Satisfactory performances of these measures have been reported after several actual earthquakes although minor damages were reported (Technical Committee on Earthquake Resistant Design of Sewage Lifelines, 2008). On the other hand, pipeline floating was widely observed around Tokyo Bay due to the 2011 Tohoku Earthquake (Mw=9.0), where the suggested mitigation measures were not implemented. The problem of these conventional measures is that considerable construction time and economic resources are required for overall excavation of backfill soil. Moreover, it is difficult to apply these methods to existing lifelines today in earthquake prone areas. Thus, new mitigation measures to reinforce existing underground lifelines are required. The present paper proposes a simple analytical method based on simple beam models to evaluate the performance of the new mitigation measures.

2 EXAMINED MITIGATION MEASURES

The authors previously proposed new measures to mitigate liquefaction-induced damage in existing lifelines (Towhata et al., 2013): the vertical drain pipe method, connection with pavement by a stiff bar (namely, horn structure method) and the chemical grouting method (Fig. 1). These measures are economical and swiftly to be implemented compared with the conventional measures. For instance, installation of the horn structure can be executed with a partial excavation of backfill soil. Vertical drain pipes that prevent onset of liquefaction around them can be installed along buried lifelines with an interval. Chemical grout that solidifies with surrounding soil can be injected into ground through tubes.

In this study, colloidal silica was used (Towhata & Kabaishima, 2001). The vertical drain pipes and the tubes to inject colloidal silica can be installed by the percussion method without excavating backfill soil.
4 SIMPLE BEAM MODELS FOR THE PROPOSED MITIGATION MEASURES

4.1 Reference case without any mitigation measure

Simple analytical models are desired for practical application of the present mitigation measures. Thus, this chapter proposes simple beam models to evaluate the performance of these measures. The deflection of a pipe ($\delta_0$) under a uniformly distributed load with hinges at both ends is generally calculated based on Euler–Bernoulli beam theory, given by:

$$\delta_0 = \frac{w}{24EI} (x^4 - 2Lx^2 + L^2x)$$

(1)

where $x = \text{distance from a hinge}$, $E = \text{Young’s modulus (2.94 GPa)}$, $I = \text{moment of inertia of area (3.04×10^7 m^4)}$, $L = \text{length of a pipe (2.56 m)}$, $w = \text{uniformly distributed uplift force caused by the balance between the buoyancy and the gravity forces acting on the pipe (38.6 N/m)}$. The shown values in the brackets are those from the experiments. Noteworthy is that connections between a pipeline and a manhole were considered as a hinge boundary since mitigation measures for manholes flotation have been developed (Japan Sewage Works Association, 2006).

4.2 Vertical drain pipe method

A simple beam model with multiple springs is shown in Fig. 2a where stiff soils around the installed drain pipes are modelled as springs with a constant ($K_D$). Fig. 2b shows the location of the drain pipes with an effective radius ($R_e$) in which sufficient reduction of excess pore water pressure is expected. For the drain pipes used in the experiments an effective radius of $R_e = 10$ cm was found. $D$ stands for the spacing between a set of drain pipes. A buried pipe should be located within $R_e$ to ensure effective performance of the drain pipes. According to the experiments, no improvement was observed when $D = 2R_e$ ($= 20$ cm). The spring constant probably depends on the strength of subsoil and location and performance of the drain pipes ($D$ and $R$). In this study $K_D = 1$, 5 and 50 kN/m were chosen and compared to the number of springs ($N_D$), between 1, 3, 5 and 7 where the springs are equally distributed. The concerned problem can be reduced to a statically indeterminate beam with springs. The deflection of pipe at every spring location ($\delta_i$) can be calculated as below:

$$\delta_i = \left[ \begin{array}{c} \delta_1 \\ \vdots \\ \delta_n \end{array} \right] = \left[ \begin{array}{cccc} 1 + K_D \cdot \delta_{i1} & \ldots & K_D \cdot \delta_{i1} & K_D \cdot \delta_{i1} \\ \vdots & \ddots & \vdots & \vdots \\ K_D \cdot \delta_{i1} & \ldots & 1 + K_D \cdot \delta_{in} & K_D \cdot \delta_{in} \end{array} \right] \cdot \left[ \begin{array}{c} \delta_1 \\ \vdots \\ \delta_n \end{array} \right]$$

(3)
where $\delta_{ij}$ = unit deflection of pipe at $i$ caused by the unit load applied at $j$ and $\delta_{iw}$ = deflection of pipe at $i$ caused by the uniform uplift force ($w$). Substitution of $K_D = 0$ into Eq. (2) results in Eq. (1). The result of the calculation is summarized in Fig. 3 where the vertical axis is the reduction ratio of pipe deflection at the centre over the deflection without any springs (Eq. 1). More reduction in floating was observed as the number of installations (springs) increased. Efficient reduction in floating was also observed with larger value of $K_D$. The experimental results captured the trend of analytical results. $D = 10$ cm showed better performance compared to $D = 14$ cm although $D = 20$ cm did not show any mitigation. However, two data of $D = 10$ cm did not follow the same curve of $K_D$. One possible reason is that the analytical model solves the force equilibrium (Eq. 2) given that the subsoil completely liquefies except for improved volume around the drain pipes. In contrast, the shaking time was limited in the experiments and possibly the equilibrium of the forces was not yet achieved. Further research with both experimental and analytical approaches is needed to improve this model. Note that a permeable layer should be situated at the top of drain pipes to dissipate pore water pressure properly.

4.3 Horn structure method

The horn structure transmits the uplift force acting on a buried pipe to a pavement situated stably above ground water table. Satisfactory resistance from the pavement can prevent floatation of the pipe. To evaluate the resistant force, a simplified factor of safety was proposed by the authors (Towhata et al., 2013). This paper assumes that the safety factor for a punching shear failure is sufficiently high. In such a condition, the simple beam model with springs can be also applicable to this method by increasing the value of spring constant ($K_D$). If the deflection of a pipe at the horn’s location is considered as null ($K_D \rightarrow \infty$), the maximum deflection of pipe does not occur at the centre. In this case, the length of pipe ($L$) in Eq. (1) can be reduced to $L' = L/(N_D+1)$ and the maximum deflection can be calculated. It is clear that floating is reduced by 1/16 times when a horn structure is installed at the centre of the pipe ($L' = L/2$). This approximation is not exactly correct due to the neglect of the moment occurred at the connection between a horn structure and a pipe. However, this overestimates the pipe deflection, which can be applicable to practice.

4.4 Chemical grouting method

Solidified soil that is sufficiently stiff to resist liquefaction is considered in this study. The density of the colloidal silica used in the experiments was merely 7% greater ($\rho_{col} = 1.07$ g/cm$^3$) than the density of water ($\rho_w = 1.00$ g/cm$^3$). The injected colloidal silica slowly permeated through the voids, and solidified with time. The bulk density of the grouted soil ($\rho_{sol}$) can be calculated as below.

$$\rho_{sol} = \frac{G_s \rho_v + \rho_{col} \varepsilon}{1 + \varepsilon}$$  \hspace{1cm} (4)

where $G_s$ = specific gravity of sand grain (= 2.64) and $\varepsilon$ = void ratio (1.093). Thus, the bulk density of solidified soil is slightly larger than that of liquefied subsoil ($\rho_{liq}$): $\rho_{sol}/\rho_{liq} = 1.02$. A factor of safety ($F_s$) is calculated based on the ratio of gravitational force to buoyancy force acting on the pipe and the solidified soil.

$$F_s = \frac{\rho_{pipe} V_{pipe} + \rho_{sol} V_{sol}}{\rho_{sol} (V_{pipe} + V_{sol})}$$ \hspace{1cm} (5)

where $\rho_{pipe}$ = bulk density of the pipe and $V_{pipe}$ = volume of the pipe. To achieve $F_s = 1.0$, $V_{sol} \geq 40V_{pipe}$ should be satisfied, which is not feasible in practice due to costs of colloidal silica. Alternatively, Towhata et al. (2013) suggested bonding a buried pipe with surrounding underground structures by solidified soil if the entire subsoil has a risk of liquefaction. For instance, colloidal silica can be injected widely so that a stiff natural soil or a pavement can prevent pipe floating (Fig. 4).
To understand the experimental results a factor of safety over the length of solidified soil ($F_{SC}$) is introduced as below:

$$F_{SC} = \frac{\rho_{soil} A_{pipe} d + \rho_{pipe} V_{col}}{\rho_{soil} (A_{pipe} d + V_{col})} \quad (6)$$

where $A_{pipe}$ = cross sectional area of the pipe, and $d$ is the diameter of the solidified soil mass. Note that colloidal silica was injected at both sides of the pipe to make a pair of solidified soil masses (Fig. 5). The spacing of the two injection points was 10 cm in the experiments. The diameter of a solidified soil mass ($d$) can be calculated from the amount of injected colloidal silica, $V_{col}$ as below:

$$d = 2 \left( \frac{3V_{col}}{4\pi} \right)^{\frac{1}{3}} = 2 \left( \frac{3}{4\pi} \frac{100V_{col}}{n} \right)^{\frac{1}{3}} \quad (7)$$

where $n$ = porosity of soil (= 100e/(1+e)). The solidified soil reduces the uniformly distributed uplift force ($w$) depending on $F_{SC}$ over the length of $d$. Herein, an equivalent downward force to reduce the buoyancy force ($P_c$) can be estimated as below:

$$P_c = w d F_{SC} \quad (8)$$

Noteworthy is that increase in $E$ or $I$ of the pipeline due to addition of chemical products was not taken into consideration in this study.

Figure 6 shows the results of analysis with different number of chemical injections. The total amount of colloidal silica injected into the model ground was 4 or 8 litres and it was divided equally into every injection location. As the number of injection increased, i.e. a small amount of colloidal silica was injected at many locations along a buried pipe, pipe floating decreased. Less significant difference was observed between 4 and 8 litres of injection for analytical results, which agreed with the experimental results when the colloidal silica was injected only at the centre of the pipe. However, experimental results showed greater mitigation compared with analytical results, particularly at larger number of injection points. Satisfactory results were observed using unliquefiable pavement above the liquefied layer or a stiff soil layer beneath the pipe as shown in Fig. 4.

A possible reason of the difference between experimental and analytical results regarding the amount of reduction in floating would be attributed to the viscous nature of liquefied soil. Floating of a pipe is time-dependent. Drag force between liquefied soil and floating pipe reduces the velocity of floating, resulting in underestimation of floating with limited duration of liquefaction (around 15 s) in experiments. Fig. 7 clearly illustrates the different velocities of floating with the same amount of colloidal silica injected (4 litres). Possibly, larger floatation can be predicted with longer shaking time, which would lead to a better agreement between the analysis and experiments. Thus, the current simple model estimates a lower bound of reduction in floating with chemical grouting method.
Compared with the conventional methods that require excavation of the entire backfill soil along an embedded pipeline. Chemical grouting would be more expensive due to its material costs than the use of drain pipes if the same level of mitigation is required. This trend may reverse if the surrounding stiff structures such as a pavement or a stiff soil layer are connected to the pipe with less amount of colloidal silica as discussed above.

6 CONCLUSION

This paper aimed to develop a framework to evaluate performances of vertical drain pipes, horn structure and chemical grouting to protect existing underground lifelines from a liquefaction disaster. These measures are excellent in terms of time and costs for construction compared with the conventional mitigation measures. An analytical method based on simple beam models was proposed to evaluate performances of the examined mitigation measures. The results of the calculation using the proposed method qualitatively captured the previous experiments. Further development on these models is expected for practical application. A flowchart was provided to select an optimum mitigation measure among the suggested options.

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