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Numerical analysis on mitigation of liquefaction induced uplift of shallow tunnels

Analyse numérique sur la réduction de la liquéfaction a induit le soulèvement des tunnels peu profonds

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ABSTRACT: Shallowly buried structures have a wide range of application including conveyance of natural gas, water supply, telecommunication to large structures such as railway subways, and highway tunnels. Among which those structures built in loose sand is prone to liquefaction-induced damages and they can be very catastrophic so as to bring normal life to a standstill. Therefore, there exists a need to come up with various methods in order to shun the liquefaction-induced uplift of buried structures. The paper presents a numerical study of the performance of a shallowly buried tunnel using a Mohr –Columb model with pore pressure measured using a FINN-Byrne model and is implemented into a finite difference code, FLAC3D. The effectiveness of a remediation technique using gravel drains is considered in the study. In the present study, a typical circular concrete structure of diameter 5 m is buried at a shallow depth of about 5.5 m from the ground surface in potentially liquefiable soil. The buried tunnel is subjected to the 1995 Kobe Earthquake and 2015 Nepal Earthquake motions.The installation of gravel drains are found effective in reducing the pore pressure accumulated at the invert of the shallowly buried tunnel as s result reduces the liquefaction-induced uplift.

RÉSUMÉ: Les structures peu profond enterrées ont un large éventail d'application comprenant le transport du gaz naturel, l'approvisionnement en eau, télécommunication à de grandes structures telles que les souterrains ferroviaires, et la route perce un tunnel. Parmi ce que ces structures établies en sable lâche est à dommages causés par la liquéfaction enclins et elles peuvent être très catastrophiques afin d'apporter la vie normale à un arrêt. Par conséquent, là existe un besoin de proposer de diverses méthodes afin d'éviter le soulèvement causé par la liquéfaction des structures enterrées. Le papier présente une étude numérique de la représentation d'un tunnel peu profond enterré utilisant l'aMohr – modèle de Columb avec de la pression de pore mesurée utilisant un modèle de Finlandais-Byrne et est mis en application dans un code de différence finie, efficacité de FLAC3D.The d'une technique de remédiation utilisant des drains de gravier est considéré dans l'étude. Dans la présente étude, une structure en béton circulaire typique de diamètre 5 m est enterrée à une profondeur d'à environ 5,5 m de la surface au sol dans le sol potentiellement liquéfiable. Le tunnel enterré est soumis les mouvements 1995 à Kobe Earthquake et 2015 de tremblement de terre du Népal. L'installation des drains de gravier sont trouvées efficace en réduisant la pression de pore accumulés à l'inverti du tunnel peu profond enterré pendant que le résultat de s réduit le soulèvement causé par la liquéfaction.

Keywords: Buried structures; Numerical modelling; Uplift of the structure; Remediation; Gravel drains.

1 INTRODUCTION

Due to urbanisation, the use of shallowly buried structures has increased. These structures are mainly to maintain a better quality and standard of life. However, these structures are more prone to damages by floatation during liquefaction of the soil after ground shaking. Such floatation can cause large damages to buried lifelines such as utility lines (power, water, and gas), communication networks, uplifted sewage manholes, transportation systems which can bring daily life to a halt. So a thorough knowledge of the uplift mechanism and its mitigation are important criteria to be taken care of when placing a buried structure in liquefiable soil.

Though many cases have been reported in the past about the liquefaction-induced uplift of various buried structure, initial studies on this were conducted by Koseki et al., (1997). Following this many studies were conducted on various buried structures including lifeline structures using both numerical approach (Ling et al., 2008; Chian et al., 2014; Madabhushi and Madabhushi 2015) and experimental approach (Orense et al., 2003; Tobita et al., 2011; Otsubo et al., 2016). All the studies proposed that the liquefaction-induced uplift occurs mainly due to three phenomena: i) lateral deformation of the soil, ii) movement of pore fluid and iii) the reconsolidation of the soil. During liquefaction of the soil, the reduced effective stress of the soil, buried structures will be uplifted due to lateral deformation of the soil to the bottom of the structure. Due to the structure, a hydraulic gradient is being formed in the region below the structure. It causes the movement of the pore fluid to the bottom of the structure which causes the structure to be further lifted from its position. However, the dissipation of the pore fluid, led to the reconsolidation of the soil causing the structure to attain its final position. Most of these studies highlighted that the primary reason for the liquefaction-induced uplift of various buried structure is the accumulation of the pore water pressure at the base of the structure and there exists a critical pore pressure beyond which the structure will get uplifted (Orense et al., 2003).

A number of studies have been conducted for liquefaction mitigation which includes (i) increasing the density of the soil (Hatanaka et al., 1987; Olarte et al., 2018), (ii) increasing the confining pressure of the soil (Funahara et al., 2015), (iii) increasing the material stiffness (Gallagher and Mitchell 2002; Ozner et al., 2015), and (iv) controlling the development of the pore pressure (Seed and Booker 1977; Brennan and Madabhushi 2002). Seed and Booker (1977) were pioneers to propose the effectiveness of the vertical drains for liquefaction mitigation and were able to propose a design chart for the effective mitigation of the liquefaction of the soil. But studies intent to arrest the liquefaction inducted uplift of the buried structures by controlling the pore pressure developed around the structure have caught the attention of few researchers (Orense et al., 2003; Ostubo et al., 2006; Taeseri et al., 2016). Sasaki and Taniguchi (1982) in their shake table study were able to find the effectiveness of gravel drains at the bottom of the structure in reducing the displacement of a partially buried structure. Few researchers including Adalier et al., (2003) in their study was able to point out that gravel drain at both the sides of the structure serveas a suitable mitigation technique against the uplift of the buried structures by accelerating the drainage and confinement of the structure. Orense et al., (2003) in their centrifuge study proposed that a critical width of replacement with gravel drain exists so that maximum pore water will be dissipated from the vicinity of the structure. While most of the studies were mainly the replacement of the soil around the structure using gravely materials Ostubo et al., (2016) studied the effect of drain pipes to arrest the uplift of the buried structure during liquefaction and the presence of surface gravel layer helped in further reducing the uplift.

A review of the literature suggests that there is a limited understanding of the use of gravel drain for the accelerating the pore pressure accumulated so as to prevent the uplift of buried structures. Therefore, there exists a need to come up with a mitigation technique to prevent the uplift of these buried structure, which could be easily applied in the field. Numerical studies onthe seismic response of an underground structure and its mitigation strategy are limited. The present study was aimed at developing a numerical model for studying the effectiveness of gravel drain to reduce the liquefaction-induced uplift of a buried structure using finite difference code FLAC3D.

2 PROBLEM DEFINITION

In the present study, poorly graded Houston sand deposit of a 60 m long 15 m wide is considered. The 16 m thick homogeneous sand deposit is considered to be resting on hard rock. A concrete hollow buried tunnel of diameter 5 m is embedded at a depth of 5.5 m from the ground surface. The entire sand profile considered is saturated with water i.e. the groundwater table is at the ground surface. The layout of the profile under study is demonstrated in Figure 1.

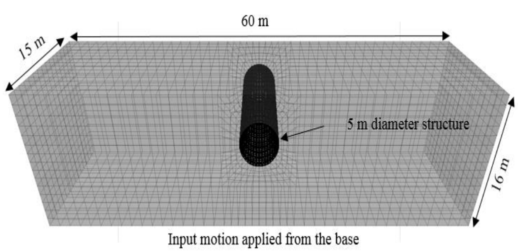

Figure 1. Schematic diagram of a buried tunnel embedded in the saturated soil medium

3 NUMERICAL MODELING

A numerical analysis was carried out to study the response of an underground structure buried in liquefiable soil with and without gravel drains. To study the response of a buried tunnel in the event of the liquefaction of the soil, a coupled fluid flow analysis is carried out using a three-dimensional finite difference formulation, FLAC3D which is capable for large displacement and strains. The whole simulation stage is done in two stages i.e. attaining static equilibrium of the 3D soil-structure model and the dynamic input motion. Dynamic analysis is based on the explicit finite difference formulation which can be coupled to structural elements and to the groundwater flow thus enabling to conduct a fully couple soil structure problems due to ground shaking.

The most critical aspect of finite difference modelling is choosing the correct mesh and boundary conditions so as to represent the fieldcondition. Based on criteria proposed by Kuhlemeyer and Lysmer (1973), the entire domain was discratized into 10800 zones in X Y and Z directions. The final shape of the model was defined using radially cylindrical elements near the structure and remaining region using brick elements. A 5 m diameter tunnel is embedded at a depth of 5.5 m from the ground surface is modelled using 1152 shell type element.

To simulate the boundary conditions the bottom boundary of the numerical model is fixed in both vertical and horizontal displacement. The vertical boundaries are restrained in horizontal direction. Additionally, the bottom boundary is assumed to be impermeable whereas water table is placed at the ground surface as a freely permeable boundary.

4 MATERIAL MODELLING

In the present study, the behaviour of the geomaterial is considered in the numerical analysis using an elastic-perfectly plastic Mohr-Coulomb model. The pore pressure build-up within the saturated soil medium during the cyclic loading under the undrained condition is computed using a Finn-Byrne model (Finn, 1981; Bryne, 1991). Finn-Byrne model is being incorporated with a Mohr-Coulomb plasticity model with which the pore pressure is being computed from the volumetric strain (Byrne, 1991; Azadi and Hosseini 2010). Byrne (1991) proposed a two-parameter equation that relates the increment of volumetric strain, $\Delta\varepsilon_{\vartheta d}$, to the cyclic shear strain amplitude, γ. The irrecoverable volume contraction within a fully saturated soil medium leads to increase of the pore pressure which is represented mathematically using two parameters C_1 and C_2 (FLAC3D; Byrne, 1991; Gregor and Shobayry 2011; Meshkinghalam et al., 2017). For the present numerical analysis, for 52% relative density soil, C_1 and C_2 calculated from Equation 2 and Equation 3 as 0.39 and 1.02 respectively. These parameters are thus used in the analysis to predict expected volume change and the pore pressure rise using the relation given in Equation 5 (Byrne, 1991).

$$\frac{\Delta\varepsilon_{\upsilon d}}{\gamma} = C_1 \exp\left(-C_2 \frac{\Delta\varepsilon_{\upsilon d}}{\gamma}\right) \tag{1}$$

$$C_2 = \frac{0.4}{C_1} \tag{2}$$

$$C_1 = 8.7(N_1)_{60}^{-1.25} \tag{3}$$

$$D_r = 15(N_1)_{60}^{-0.5} \tag{4}$$

$$\Delta u = E_r \Delta\varepsilon_{\vartheta} \tag{5}$$

Where Δu is the incremental pore water pressure and E_r is the rebound modulus of the sand skeleton.

The general properties of the sand used in the analysis are adopted from Chian and Madabhushi (2013) in their experimental studies and is shown in Table 1. The interface between the soil and the structure is characterized using Mohr-Coulomb strength criteria which mainly depends on the friction angle (21.8^0) between the structure and the soil.

Table 1. Property of the soil used

Parameters	Houston Sand	Gravel drain
Relative density (%)	52	52
Modulus of Elasticity (MPa)	14	55
Angle of internal friction	33	43
Density (kg/m^3)	1490	1662
Permeability (m/s)	10^{-4}	10^{-1}
Porosity	0.45	0.201

5 INPUT MOTION

To study the uplift response of a buried structure earthquake motions considered are the 2015 Nepal-Gorkha earthquake and 1995 Kobe earthquake. The 2015 Nepal-Gorkha earthquake motion considered has a peak ground acceleration of about 0.155g and alow predominant frequency of 0.23 Hz. Whereas, the 1995 Kobe earthquake motion considerd has a peak ground acceleration of 0.3g and maximum predominant frequency of 2 Hz. Both the input motions are applied to the base of the model. The input motion applied to the base of the model in the present study is shown in Figure 2.

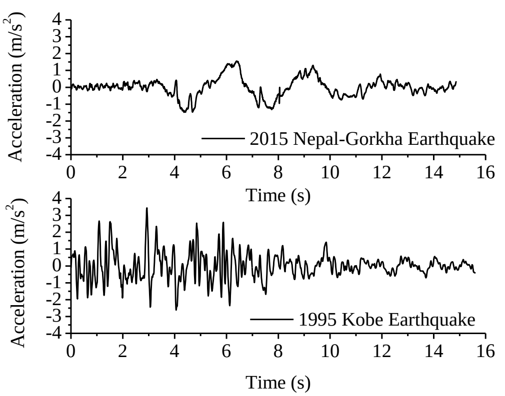

Figure 2. Earthquake input motions

6 RESULTS AND DISCUSSION

6.1 Pore pressure response away from the tunnel

The pore pressure response observed in the soil away from the structure at a depth of 7.5 m, 5.5 m and 1.7 m from the ground surface is shown in Figure 3. The pattern of pore pressure developed away from the tunnel is almost the same for both the input motion. In both the input motions, a rapid increase of pore pressure is observed as soon as the shaking starts. The pore pressure ratio started to increase as the shaking started and reached a maximum of 0.6 in the case of 2015 Nepal-Gorkha earthquake and 1.0 in the case of 1995 Kobe earthquake motion. Once the pore pressure ratio reaches the maximum it remains more or less a constant throughout the entire duration of shaking. The point from where the pore pressure remains more or less the same indicates the point of liquefaction. From both the figure it is clearly seen that the entire depth of the soil liquefied due to the ground shaking.

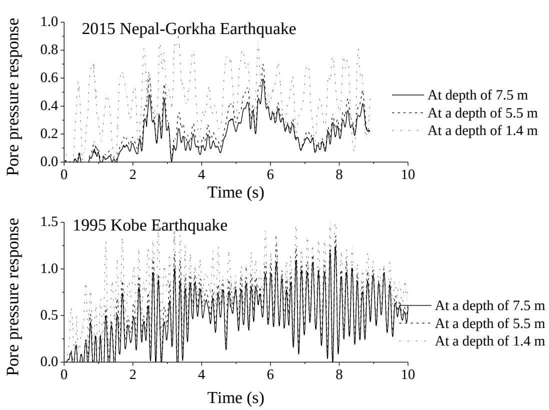

Figure 3. Pore pressure response observed away from the tunnel

6.2 Uplift Response

The uplift response of the tunnel buried at a depth of 5.5 m from the ground surface is shown in Figure 4. Even though the pore pressure accumulation occurs as the shaking starts, the uplift occurs after few seconds only. Due to the 2015 Nepal-Gorkha motion, the tunnel showed an uplift within 2 sec whereas 1995 Kobe earthquake with larger predominant frequency experienced an uplift after 3 sec. In both cases, the tunnel gets uplift once the uplifting force exceeds the resisting force (Koseki et al., 1997). However, due to the larger PGA of the 1995 Kobe Earthquake motion compared to 2015 Nepal-Gorkha earthquake a higher uplift of about 0.215 m is observed. Whereas a maximum uplift of about 0.05 m is observed in the case of 2015 Nepal-Gorkha earthquake. In both cases the tunnel got uplifted from its mean position depending on the characteristics of the input motion. However, a more effective mitigation need to be proposed to reduce the liquefaction-induced uplift of the buried structure.

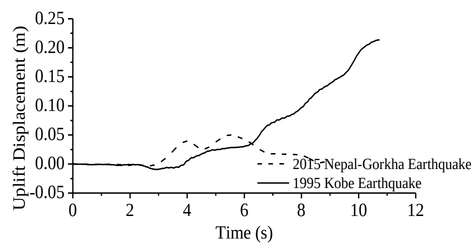

Figure 5.Uplift obtained without any drains

7 MITIGATION USING GRAVEL DRAINS

To mitigate the uplift of the tunnel as a result of liquefaction of the soil, the method of accelerated drainage of pore pressure accumulated from the vicinity of the structure is employed. In the present study, for the purpose of reducing the liquefaction-induced uplift of the buried structure gravel drains are employed. As the gravel drains of greater permeability are considered, the dissipation occurs faster through the drains compared to the surrounding region.

In the present study gravel drains of diameter 0.5 m is placed at a distance of 1.5D from the centre of the structure on both sides. The gravel drains are considered to be of more permeable well-graded soil of the same relative density of

52% as shown in Figure 6. The general properties of the fraction B sand used for the study are adopted from that used by Ambily and Gandhi (2007) in their study and the corresponding range is given in Table 1. The response of the structure to both the input motions i.e. 1995 Kobe earthquake and 2015 Nepal-Gorkha earthquake are studied.

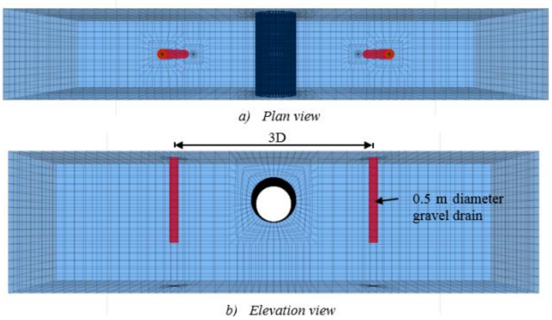

Figure 6. Mitigation used in the present study

7.1 Pore pressure response at the invert

The pore pressure ratio obtained at the invert of the tunnel with and without the gravel drains subjected to both the input motions are compared in Figure 7. It is seen that a maximum pore pressure ratio of about 0.36 is observed when no drains are used and is subjected to 1995 Kobe earthquake. In the presence of gravel drains it reduced to about 0.239. Similar observation are seens when subjected to 2015 Nepal-Gorkha earthquake. In both the cases by using the gravel drains the pore pressure ratio obtained at the invert is reduced by about 40%. Presence of drains with better permeability, the pore pressure dissipation at the invert of the structure was accelerated. It can be further explained using Figure 8 which shows the flow vectors for both the cases with and without gravel drains. The pore pressure accumulated at the vicinity of the tunnel is indicated by the black arrows. It is seen that the case when no gravel drains are considered the density of the black arrows are more compared to when gravel drains are used. It clearly shows that in the presence of the gravel drains pore pressure accumulation at the invert of the structure is reduced considerably.

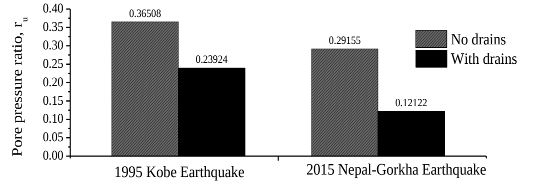

Figure 7. Pore pressure obtained at the invert of the tunnel

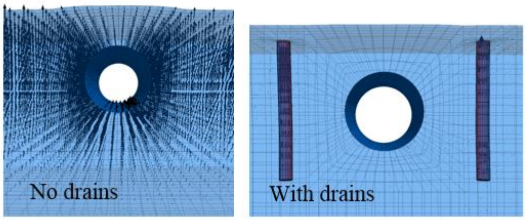

Fig 8. Flow vector within the model, a) without any gravel drains b) with gravel drain

7.2 Uplift displacement

The maximum uplift of the tunnel with and without the gravel drains using both the input motion is studied and is given in Table 2. When no drains are considered, a maximum pore pressure ratio of about 0.36 and 0.29 is observed near the invert using the 1995 Kobe earthquake and 2015 Nepal-Gorkha earthquake respectively. As a result, an uplift of about 0.215 and 0.05 m is being observed due to 1995 Kobe earthquake and 2015 Nepal-Gorkha earthquake respectively. In the case when gravel drains are considered the maximum uplift observed in both cases were reduced. A reduction in the uplift of about 65% and 98% was observed when subjected to 1995 Kobe earthquake and 2015 Nepal-Gorkha earthquake respectively. It is also to be noted that the gravel drains were able to arrest the uplift completely in the case when subjected to 2015 Nepal-Gorkha earthquake. However, a small magnitude of uplift is observed using 1995 Kobe earthquake since a small pore pressure was accumulated near the invert of the structure and the resisting force erted against the upward force was not enough to resist the uplift caused. It clearly shows that the gravel drains can be used to reduce the liquefaction-induced uplift to a great extent.

Table 2. The uplift displacement of the tunnel

Input motion	Without drains	With drains
2015 Nepal-Gorkha earthquake	0.036	0.002
1995 Kobe earthquake	0.215	0.077

8 CONCLUSION

In this paper, the numerical simulation of the uplift of an tunnel buried within a saturated soil was carried out using FINN-Byrne model and its results in term of pore pressure developed within the model and the uplift displacement are studied. The feasibility of improvement by using the gravel drains near the structure subjected to 2015 Nepal-Gorkha earthquake and 1995 Kobe earthquake is studied. The most important findings are as follows:

- It is found that the uplift of the tunnel due to low-frequency input motion occurs early compared to high-frequency input motion. The maximum uplift of the tunnel occurs for earthquake motion with high PGA.
- Installation of the gravel drains around the tunnel accelerates the pore pressure dissipation from the vicinity of the structure and there was a considerable reduction in the pore pressure accumulated at the invert of the structure.
- The use of gravel drains reduces the liquefaction-induced uplift by 65% and 98% for 1995 Kobe earthquake and 2015 Nepal-Gorkha earthquake respectively.

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