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TUNNEL AND PIPELINE FLOATATION IN LIQUEFIED SOILS

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ABSTRACT

Tunnels and pipelines have lower unit weight than the surrounding soil and are commonly deemed to float in liquefied soils due to their buoyancy. Centrifuge tests have been conducted to investigate the effect of buoyancy relative to the uplift response of large diameter tunnels and smaller diameter pipes under earthquake loading. The ratio of buoyancy to unit weight is similar for both the tunnel and pipe. Likewise, they have the same ratio for buried depth against diameter in these tests. However, the uplift displacement of the pipe is more significant than the tunnel in liquefied soil. This indicates that the floatation susceptibility of these buried structures is not solely governed by their buoyancy. This paper aims to demonstrate the possible influence of soil mobility affecting the uplift response of buried structures following an earthquake.

Keywords: tunnel, pipe, floatation, liquefaction, centrifuge, buoyancy, uplift

INTRODUCTION

There has been a number of buried structures which were severely damaged in recent earthquakes such the 1990 Luzon earthquake, 1994 Hokkaido Toho-oki earthquake, 1995 Kobe earthquake, 2003 Tokachi-oki earthquake (Towhata, 2008), 2004 Chuetsu earthquake (Tobita et al., 2009), 2009 Padang earthquake (Chian et al., 2010a) and the 2010 Chile earthquake (GEER, 2010). Many of these were associated with the floatation of buried structures in liquefied soils as shown in Figure 1.

Figure 1. Tanks buoyed above the ground surface during the 2010 Chile earthquake (GEER, 2010)

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Pipelines are generally subjected to a buoyant force due to their lower submerged unit weight than the surrounding soil. Therefore, floatation of these structures may occur as soon as the soil liquefies and loses most of its shear strength. In the case of tunnels, they usually serve as transportation facilities such as roadways and subways. Hence, they pose a high degree of risk in terms of human losses should they fail during a major earthquake. However, severe damages on large diameter tunnels were rare. This could be due to the few tunnels present in the vicinity of these earthquakes. Alternatively, soil liquefaction may not have reached the buried depth of tunnels to inflict significant damage to these large structures. Another hypothesis could be their relatively large and heavy nature as compared to smaller diameter pipelines. This paper aims to address this issue and compare the differences between tunnel and pipe floatation in liquefied soils.

EXPERIMENTAL SETUP

The principle of geotechnical centrifuge modelling is to replicate identical stress condition as in the prototype scale. A scaled model is made to correspond with the prototype at the pre-determined centrifuge g-level (Schofield, 1981). As shown in Table 1, a 1:N scaled model experiences the same stress condition as the prototype when subjected to a centrifugal acceleration of N × g level.

Table 1. Centrifuge Scaling Laws

Parameter	Model/Prototype	Units
General scaling laws (quasi-static events)		
Length	1/N	m
Area	$1/N^2$	m^2
Volume	$1/N^3$	m^3
Mass	$1/N^3$	$Nm^{-1}s^2$
Stress	1	Nm^{-2}
Strain	1	-
Force	$1/N^2$	N
Seepage velocity	1/N	ms^{-1}
Time (consolidation)	$1/N^2$	s
Dynamic events		
Time (dynamic)	1/N	s
Frequency	N	s^{-1}
Displacement	1/N	m
Velocity	1	ms^{-1}
Acceleration/acceleration due to gravity	N	ms^{-2}

Details of Centrifuge Testing

The model tunnel and pipe were buried in the sand model prepared to the relative density (D_R) of 65% using the sand pluviation method with the automatic sand pourer using the design charts by Chian et al. (2010b). The layout of the tunnel, pipeline and instruments are illustrated in Figure 2. Mouldable Duxseal was placed on the sides of the container to minimise reflecting incident stress waves (Steedman and Madabhushi, 1991). The material properties of Hostun sand used in the model are described in Table 2.

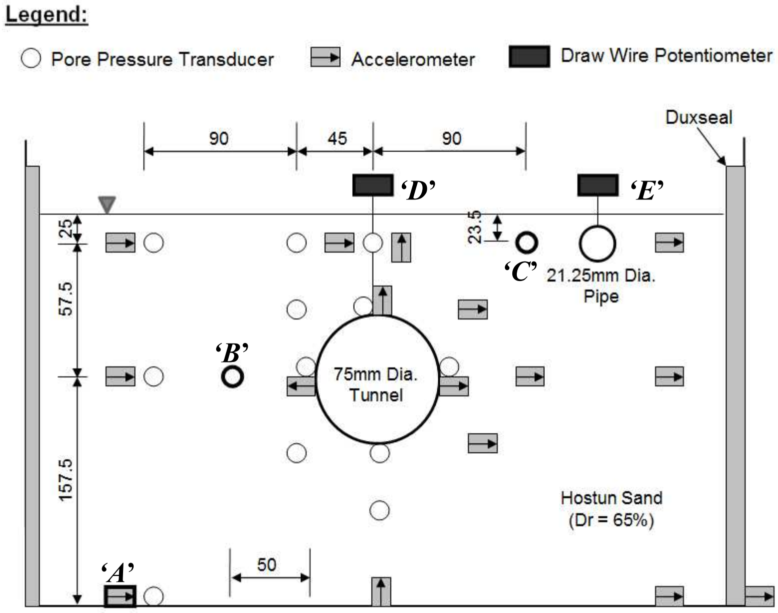

Figure 2. Layout of instruments in centrifuge model, not to scale

Table 2. Material properties of Hostun sand

Properties	Values
Φ_{crit} *	33^o
D_{10}	0.209 mm
D_{50}	0.335 mm
D_{60}	0.365 mm
e_{min} *	0.555
e_{max} *	1.01
G_s *	2.65

* After Mitrani (2006)

Based on centrifuge scaling laws, the diameter of the 75mm model tunnel and 21.25mm model pipe would be approximately 5m and 1.42m in prototype scale in a 66.7g test. They were buried at shallow depths of 1.1 times of their diameters in the sand to ascertain their difference in floatation response with respect to their buried depth. The tunnel and pipe were deemed suitable to be placed in the same model as their difference in centrifugal radial gravity field varies by less than 1.4% with the use of a 10m diameter Turner beam centrifuge at the University of Cambridge.

Draw-wire potentiometers were attached to the crown of the tunnel and pipe to measure their uplift response throughout the centrifuge testing. Securing supports were also put in place so as to avoid any accidental movement of the tunnel and pipe prior to centrifuge testing. The model was subsequently saturated with high viscous methyl cellulose fluid of 66.7cSt, equivalent to the centrifuge g-level of 66.7g to satisfy dynamic centrifuge scaling laws. This saturation process was aided with the computerised system developed by Stringer and Madabhushi (2009). The desired viscosity of Methyl cellulose fluid was achieved with the weight concentration of Hydroxypropyl methylcellulose (HPMC) powder in water proposed by Stewart et al. (1998). During the test, in-flight earthquakes were fired with the Stored Angular Momentum (SAM) earthquake actuator devised by Madabhushi et al. (1998).

Compatibility of Model Tunnel and Pipe

In order to make direct comparison between the uplift response of tunnels and pipes in liquefiable soil during an earthquake, the test conditions have to be similar. Table 3 shows the compatibility of the two buried structures.

Table 3. Compatibility of properties between tunnel and pipe in prototype scale

Properties of buried structure	Tunnel	Pipe	Tunnel/pipe ratio
Outer diameter	5 m	1.42 m	3.53
Flexibility ratio	3.12	2.93	1.06
Cross-sectional unit weight	63.61 kN/m	5.14 kN/m	12.37
Displaced cross-sectional area	19.65 m^2	1.58 m^2	12.46
Buoyancy force	192.81 kN/m	15.48 kN/m	12.46
Net uplift force	129.20 kN/m	10.34 kN/m	12.50 ($\approx 3.53^2$)

According to the scaling law for unit force in Table 1, the desirable net uplift force ratio between the tunnel and pipe is a square of their outer diameter ratio. This criterion has been achieved in the centrifuge modelling, while maintaining similar test conditions of buried depth, cross-sectional unit weight and displaced cross-sectional area as shown in Table 3. In addition, the flexibility ratio (Peck et al. 1972), expressed below in Equation 1, of the tunnel and pipe was comparable as well. As such, the influence of the buoyancy of these buried structures in liquefied soil can be investigated objectively.

$$F = \frac{E_s R^3 (1 - \nu_l^2)}{6 E_l I_l (1 + \nu_s)} \tag{1}$$

where F is the flexibility ratio
E_s is the Young's modulus of soil around the tunnel
R is the tunnel radius
ν_l is the Poisson's ratio of tunnel lining
E_l is the Young's modulus of tunnel lining
I_l is the second moment of area per unit length for the longitudinal section of tunnel lining
ν_s is the Poisson's ratio of soil around the tunnel

RESULTS

In a static condition, the main component encouraging floatation of a buried structure is its buoyancy. This force is equivalent to the weight of displaced fluid based on Archimedes' Principle. For structures buried in liquefiable deposits, the loss of shear strength of the soil due to excess pore pressure generation in an earthquake may encourage floatation to occur. In this centrifuge test, a high degree of soil liquefaction at the depths of both the tunnel and pipe are essential so as to yield similar soil conditions while determining their differences in uplift response.

Figures 3 and 4 illustrate the base earthquake accelerations and progress of liquefaction at depths where the tunnel and pipe were buried. Considerable degree of liquefaction depicted by high excess pore pressure was attained, especially in the first earthquake. At this stage, the shear strength of the soil was reduced significantly, resulting in the floatation of tunnel and pipe as evident in the uplift displacement readings from the potentiometers as shown in Figure 5.

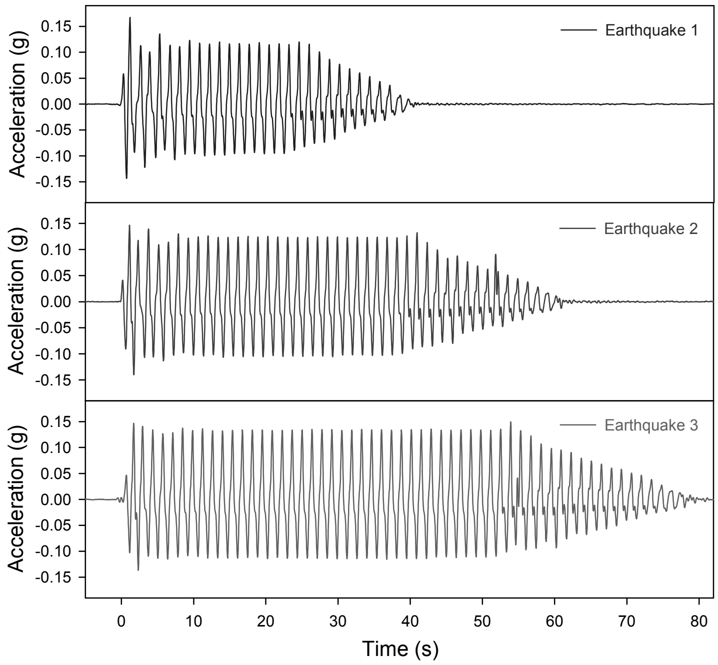

Figure 3. Earthquake accelerations at the base of the model (*'A'* in Figure 2)

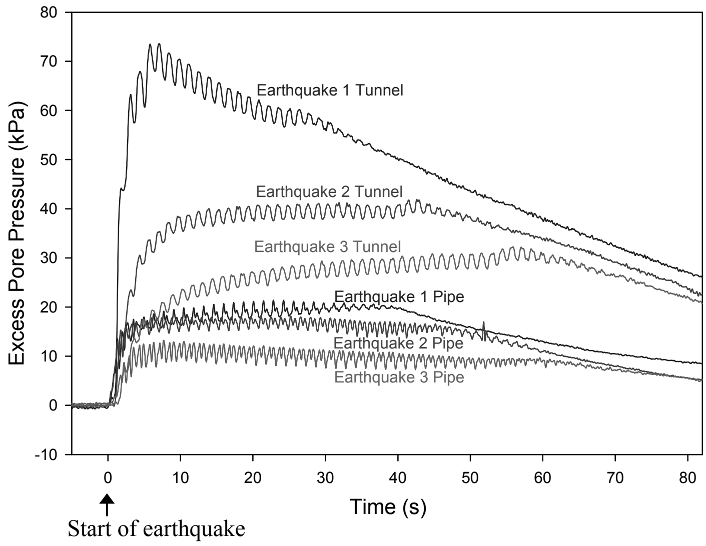

Figure 4. Excess pore pressure generation at the springing depth of tunnels and pipes (*'B' and 'C'* in Figure 2)

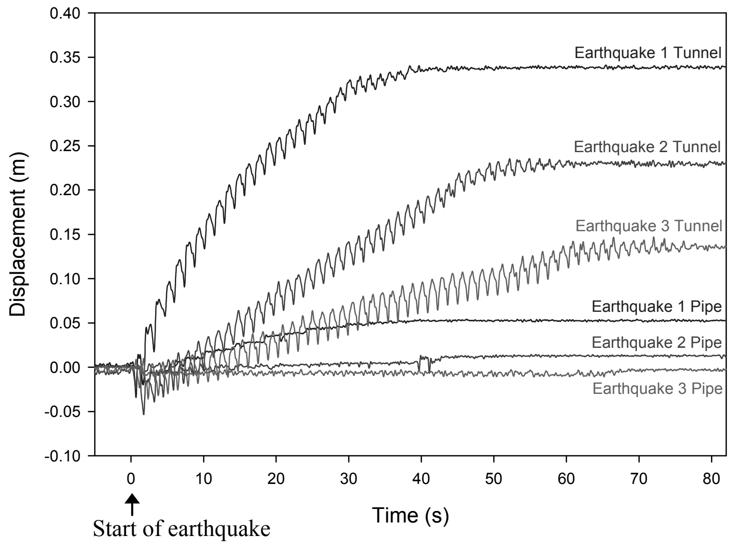

Figure 5. Uplift displacements of tunnels and pipes (*'D' and 'E'* in Figure 2)

Uplift Response of Tunnel and Pipe

Given the close compatibility between the tunnel and pipe, their measured uplift would have been similar to each other if the floatation mechanism is highly dominated by the effect of buoyancy. However, observations showed otherwise. Based on the normalised uplift displacement versus time plot in Figure 6, the pipe has shown to be more susceptible to floatation than the tunnel in all cases. Nevertheless, both the tunnel and pipe showed decreasing uplift displacements with subsequent earthquakes due to soil densification.

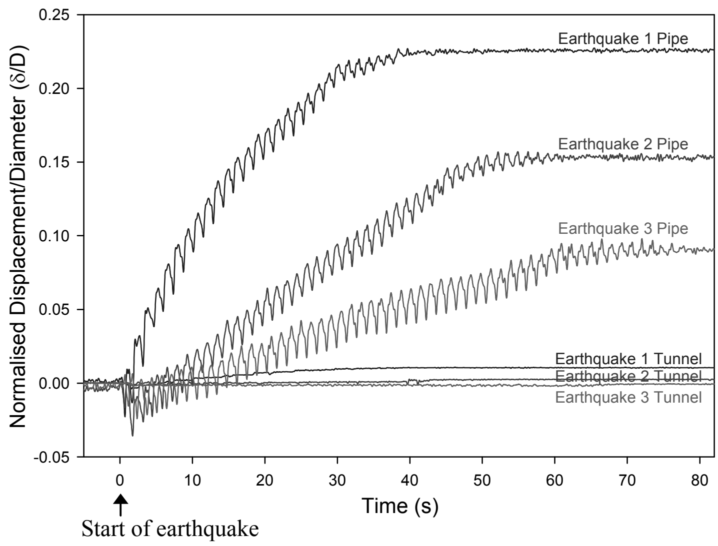

Figure 6. Normalised uplift displacement of tunnels and pipes

In view of the large disproportion of uplift displacement between the tunnel and pipe, there exists a key factor affecting the floatation of buried structures. This factor could be the mobility of liquefied soil. Based on observations from Particle Imaging Velocimetry (PIV) analysis as illustrated in Figure 7, the soil near the surface has a higher mobility than at greater depths due to its lower confining stress. Since the floatation of a buried structure in liquefied soil requires the displacement of soil around the structure (Chian and Madabhushi, 2010), the higher mobility of soil near the soil surface would encourage floatation. Being buried at a shallower depth, the pipe would therefore displace more significantly as compared to the tunnel due to the higher mobility of the soil around the pipe.

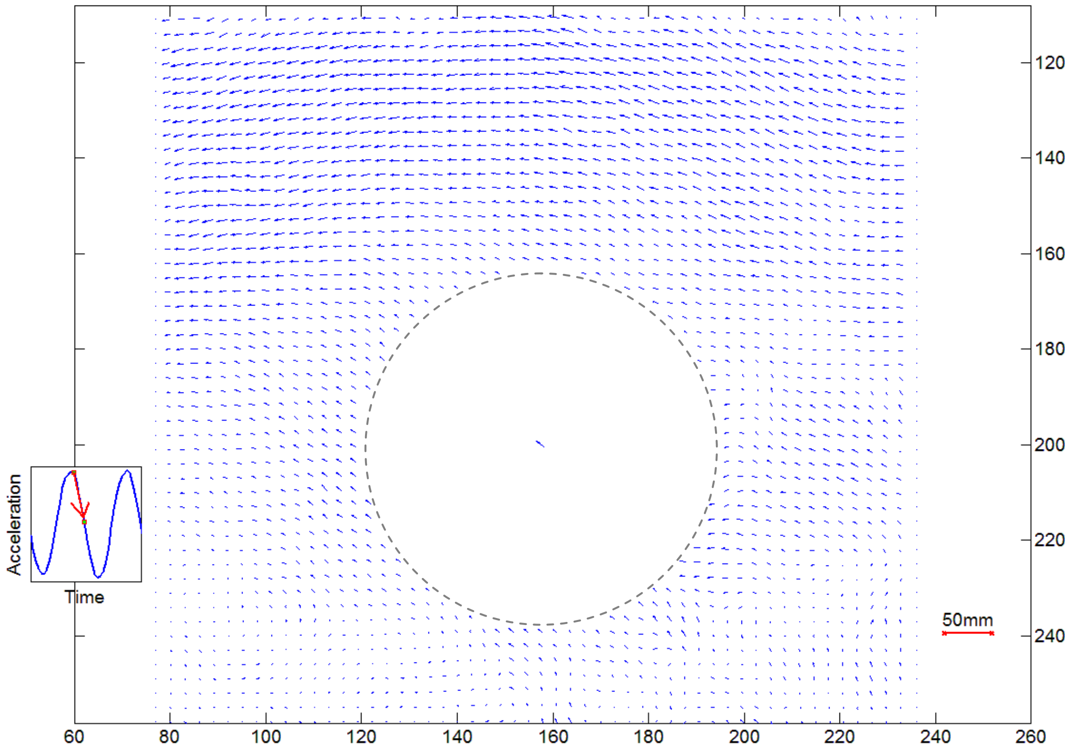

Figure 7. Displacement of soil and tunnel at each phase of the earthquake cycle obtained from PIV analysis (Chian and Madabhushi, 2010)

CONCLUSION

Comparison between compatible buried structures showed a large disparity in their uplift response in liquefiable soil during an earthquake. This indicates that the floatation susceptibility of such buried structures is not solely governed by their buoyancy in liquefied soil. Other factors such the mobility of the liquefied soil may influence the uplift response significantly.

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