1g Shaking Table tests on Tunnel-Soil Interaction under Repeated Shaking Events in Partially Saturated Ground

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Abstract. Recently, underground structures become a vital component in infra-structure systems. Based on the tunnel market survey 2019, the construction of underground structures in India increased tremendously of about 88%. The recent repeated earthquake events i.e., Japan Earthquake (2011) and Kumamoto Earthquake (2016) evidenced the possible occurrence of repeated continuous earthquake and becoming a serious threat to the safety of infra-structures. Also, In India, majority portion of proposed underground projects mainly lies in seismically active regions e.g., North India. Typically, the sub-surface profile of these Indo-Gangetic plains found to have significant portion of silt content with ground water table considerably at shallow depth. The changes in ground water table influence the soil saturation which plays a major role during dynamic events. In this study, 1g shaking table tests were conducted on scaled down square tunnel model embedded in 50% partially saturated ground of 60% relative density under repeated dynamic events. Tests were performed with 0.1g and 0.2g repeated acceleration loading and tunnel soil interaction was evaluated by monitoring contact-based acceleration and pore pressure transducers and non-contact based (2-DIC) instrumentation technique for acceleration, pore pressure, strain and displacement measurement. Based on the results, factors affecting tunnel soil interaction under repeated shaking events were evaluated and compared.

Keywords: Soil Tunnel Interaction; Pore water pressure generation; Repeated shaking events; Digital Image Correlation (2-DIC); Tunnel Displacement and Strain

1 Introduction

During the recent years, limited land space has led to the scope for development of underground infrastructure facilities for transportation purposes. In India, some of these underground developments are under operational and some at construction stage. From the recent past, it is understood that underground tunnels are safer than above ground structures due to its confinement characteristics offered by the surrounding ground, (Hashash, 2001). However, underground structures underwent excessive deformation which is evident from various earthquakes such as Kobe Earthquake (1995), Chi Chi Earthquake (1999), and Wenchuan Earthquake (2008). As most of the ongoing underground infrastructure constructions in India are at shallow depth with considerable functioning structures are in highly seismic zones, assessment of tunnel soil interaction under dynamic loading is highly essential considering earthquake incidence. In addition, few portion at these locations typically North India, the sub-surface soil bed contains silty sand deposits with ground water table at shallow depth. When these silty sand deposits come in contact with water table during dynamic events, the resulting development of pore water pressures results in soil deformation and occurrence of soil liquefaction. Few researchers conducted experimental studies to evaluate the influence of ground water table on seismic behavior of tunnels in sand deposits (Ding et al, 2021). It was observed that, existence of tunnel induced generation of pore water pressures. Cheng et al (2017) conducted shaking table tests on immersed tunnel on fully saturated soil maintaining 200 mm layer of water level over the ground surface. It was observed that larger earthquake excitation influenced the generation of pore water pressures and causing tunnel and soil deformations. From the literatures it was found that, limited studies were available in assessing the tunnel soil interaction under dynamic conditions considering the effect of partial saturation of soil surrounding the tunnel system. Hence, in this experimental work, a scale down model tunnel of 280 mm ×280 mm× 740 mm installed in a silty sand bed of 60% relative density and 50% saturation was used for estimating soil-tunnel interaction under repeated shaking events. Sinusoidal input motion of 0.1g and 0.2g was chosen and given as input sequentially to the prepared ground. Major factors affecting the tunnel soil interaction under repeated shaking events is assessed from the obtained contact based and non-contact-based instrumentation scheme and discussed.

2 Methodology

2.1 Uniaxial Shake table

For conducting the experimental study, an uniaxial shake table having dimension of 2 m ×2 m was used. The maximum load carrying capacity and actuator displacement of shake table was 3 T and ±160 mm respectively. The operating frequency and acceleration range was 0.01 to 50 Hz and 0.001g to 1g respectively. A specially fabricated perspex glass tank of dimension 1.7 m ×1 m×0.75 m was used in this study. The table was mounted on the shake table for sample preparation and experimental testing. Polyethylene foam of 50 mm thick was pasted on both sides of the tank perpendicular to shaking direction to minimize the rigid boundary effects.

2.2 Soil selected for the study

In this research work, sand collected from solani river bed was used for experiments. The soil was characterized as poorly graded sand based on the preliminary tests on sand conducted as per IS 2720 Part III and IV respectively. The maximum and minimum void ratio of the sand was 0.80 and 0.56 respectively. Table 1 presents the other properties of sand used in the experimental study. The other details are listed in Table 1.

Table 1. Properties of soil used in the experimental study

Soil Characteristics	Value
Specific Gravity	2.67
Uniformity Coefficient (C_u)	2.6
Coefficient of curvature (C_C)	1.14
Bulk unit weight of sand	17.65 KN/m^3
Soil Type	Poorly graded
Maximum density of sand	16.7 KN/m^3
Minimum density of sand	14.5 KN/m^3

2.3 Sample preparation

In this study, a ground having 900 mm height with 60 % relative density was prepared for experimental testing. The ground was prepared with 50% saturation representing partially saturated conditions as a result of ground water table fluctuations experienced during dynamic events. The required water content for achieving 50% saturation was calculated prior using the relationship $S \times e = w \times G$. and the bed was prepared using sand pluviation technique. For achieving target density, relative density tests were conducted to determine the maximum and minimum void ratio as per IS 2720 Part VI (2006). The natural void ratio was calculated using the equation, $e = e_{max} - D_r(e_{max} - e_{min})$.By estimating the unit weight of solani sand, the quantity of sand and water to be filled was calculated prior and sample preparation was done by pouring sand using a conical hopper. The height of fall from conical hopper was fixed as 190 mm which based on the relative density tests for achieving 60% relative density. The total quantity of soil and water was divided in to 4 layers for achieving maximum uniformity in ground preparation. Initially the quantity of water required for the first 200 mm layer was poured into the tank and followed by sand pouring through conical hopper at the selected height. After 200 mm depth was prepared, tunnel was placed above the prepared sand bed. To arrest the water ingress into the tunnel model, silicon sealant was used around the tunnel model. The sample preparation was continued until the desired height of 900 mm was achieved.

2.4 Scaling laws and selected input motion

For understanding the prototype behavior in laboratory, it is essential to develop a scaled down model such that response of various parameters obtained in laboratory testing can represent the response of prototype structure. In order to achieve this similarity, scaling laws were used. The scaling factors for various parameters are derived based on the dimensional analysis method using Buckingham Pi Theorem. Table 2 shows the scaling factor for various parameters.

For scaling down a prototype tunnel, a geometric scaling ratio of 1:10 was adopted in this study. The tunnel having dimensions 280 mm×280 mm×740 mm was prepared using gypsum with mix ratio of 1:0.7. The compressive strength of the mix based on the cube tests was 3.2MPa.

As the primary objective of the current experimental investigation is to assess the behavior of tunnel embedded partially saturated sand bed under repeated dynamic conditions; sinusoidal input motion of 0.1g and 0.2g with 5 Hz frequency was adopted and applied to the ground. The selected input motion simulates low to medium shaking events in the actual conditions. After 0.1g input motion, the partially saturated sand bed embedded with tunnel was allowed for complete dissipation of generated excess pore water pressure and subsequently 0.2g input motion was given as input.

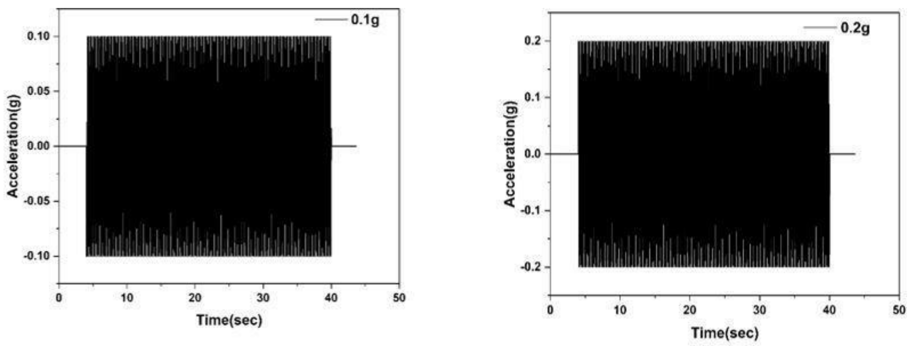

Fig 1. Selected input motion for the experimental study

Table 2 Scaling laws adopted in the experimental investigation

Variable	Scale Factor	For λ=10
Length	λ	10
Density	1	1
Force	λ^3	1000
Stress	λ	10
Strain	1	1
Elastic Modulus	λ	10
Acceleration	1	1
Time	$\sqrt{\lambda}$	$\sqrt{10}$
Frequency	$1/\sqrt{\lambda}$	$1/\sqrt{10}$

2.5 Instrumentation

For monitoring the response of tunnel embedded ground under repeated shaking events contact based instruments such as accelerometers, displacement transducers and pore pressure transducers were used. From the installed instruments, the tunnel soil interaction due to repeated shaking events were assessed by measuring the acceleration response, settlement of soil at ground surface and developed pore water pressures at different depths. For measuring the displacement and strains developed on the surface of the tunnel, non-contact based 2-Dimensional digital image correlation was adopted. The detailed instrumentation scheme is shown in Fig. 2.

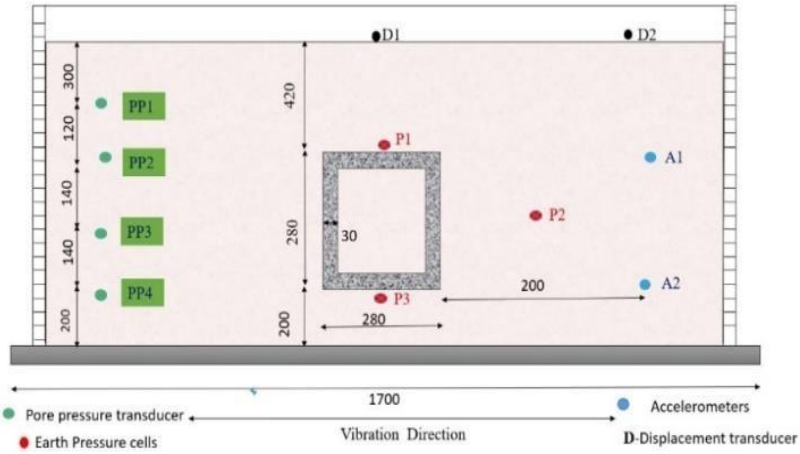

Fig.2. Sensor arrangement for experimental test program

2.5.1 Digital Image correlation

Digital image correlation (DIC) is a non-contact, subset-based technique for measuring the full field displacement and strains by capturing series of images. DIC involves a camera, a set of LED lights and VIC 2D software for processing the captured images. DIC generally involves tracking the corresponding position between matching subsets in reference and deformed images. Zhao et al (2021). Before conducting the experimental test, following procedure is followed in preparing the specimen for capturing the images. a) Initially the surface to which displacement and strains need to be obtained (Area of Interest AOI) is sprayed with white paint and allowed for drying. b) After drying, the AOI is applied with black speckles. c) Before conducting the experimental test, a reference image was captured and subsequently series of images were captured during the test until the end of shaking duration. The subsequent images captured during the test are called as deformed images. The reference image was then calibrated by specifying the width of the tunnel and subsequently the reference image and deformed images are correlated by specifying the suitable number of subsets and step size in such a way that, a single subset should contain a minimum of three to five speckles. The reference image and deformed images are processed using VIC 2D software to obtain the displacement and strains developed on the tunnel. The experimental test set up and measurement of strain at different locations was shown in Fig.3 and 4.

Fig.3. Experimental set up for tunnel soil interaction test

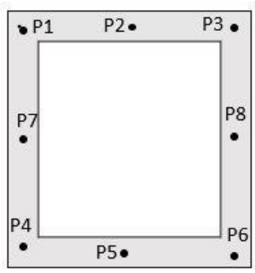

Fig.4. Different locations selected on the tunnel for obtaining displacement and strains

3 Results and Discussions

3.1 Displacement of Soil

The displacement of soil during 0.1g and 0.2g input motion was measured using a displacement transducer placed at ground surface above the tunnel location (D1) and adjacent to the tunnel location (D2) as presented in the Fig.2. The observed displacement values at 0.1g and 0.2g loading conditions is shown in Fig. 5. It can be observed that displacement of soil found to increase with increase in input motion. As seen in Fig. 4 (b), adjacent soil (D2) showed increment in displacement compared to soil above the tunnel system (D1). The above observations inferred that, (i) application of repeated shaking induces soil displacement and (ii) comparatively tunnel embedded soil portion showed lesser displacement due to combined soil densification and tunnel embedment. During 0.1g input motion, the displacement of soil at D1 was 1.1 mm where-as during 0.2g input motion, the displacement of soil found to be 2.25 mm. Similarly, the displacement of soil at D2 was 1.8 mm and 3.65 mm corresponding to 0.2g input motion. About 1.62 times increment in soil displacement was observed in adjacent soil portion compared to middle portion which evidenced the occurrence of soil densification due to repeated shaking events. Further, the time of shaking for both given selected input motion was 40 seconds which also additionally contributed in inducing soil densification in the partially saturated ground. The influence of soil densification in pore pressure response is discussed in the following section.

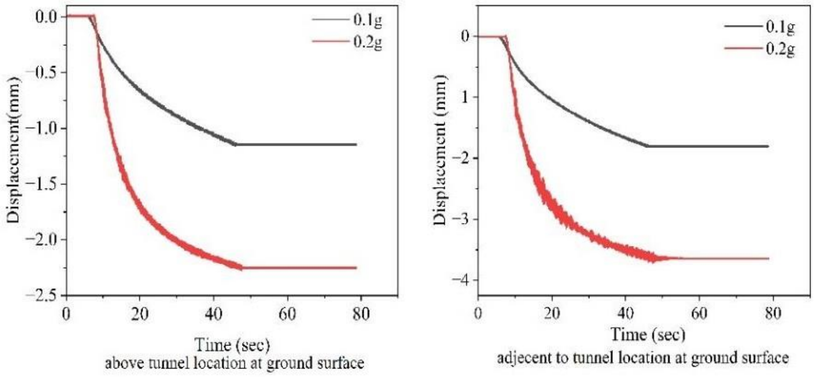

Fig.5. Displacement of soil during 0.1g and 0.2g input motion

3.2 Development of Pore water pressure and pore water pressure ratio in the soil

For measuring the pore water pressure developed at different depths, pore pressure transducers were installed at 300 mm (PP1),420 mm (PP2) ,560 mm (PP3) and 700 mm (PP4) depth inside the soil respectively. Fig.6 illustrates the developed pore water pressures and corresponding estimated pore pressure ratio for 0.1g and 0.2g loading conditions. It can be observed that no significant generation of pore water pressures during 0.1g shaking conditions. This may be due to the selected water content in sample preparation which experiences compaction during initial shaking condition. Further, the tunnel embedment improves soil densification inside the ground which also minimizes generation of pore water pressures. The estimated pore pressure ratio also evidenced the occurrence of soil densification during 0.1g shaking condition. The maximum pore water pressure generation was at deeper depths, i.e., 700 mm and 600 mm respectively which mainly due to overburden depth which generates excess pore water pressures. However, at 0.2g input considerable development of pore water pressure at different depths was observed. This was mainly due to the influence of repeated incremental shaking which disturbs the compacted soil due to which generation of pore water pressure from bottom to top was observed. Similar to 0.1g observations, pore pressure transducers located at deeper depth showed higher generation of pore water pressures which also evidenced the disturbances in the compacted ground. Similarly, the estimated pore pressure ratio was about 1.5 to 3.5 times more during subsequent repeated shaking event of 0.2g input motion when compared to previous 0.1g loading. Though, the estimated pore pressure ratio did not show any significant observations on soil liquefaction, the generated pore water pressures evidenced the disturbances induced by the repeated incremental shaking events. In the absence of proper drainage member, the development of pore water pressures during repeated shaking events may results in soil liquefaction and associated soil deformation effects. Thus, the observations highlighted the need for developing proper design of control measures for the tunnel embedded ground under repeated shaking events.

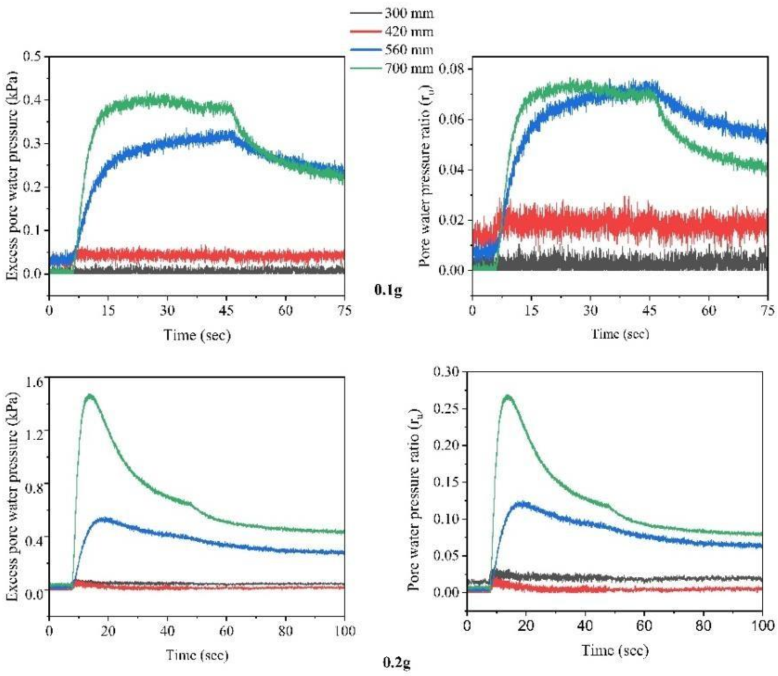

Fig.6. Developed excess pore water pressure and pore pressure ratio during 0.1g and 0.2g input motion

3.3 Displacement and Strains developed on the tunnel

For measuring the displacement of the tunnel model due to repeated shaking events, non-contact based 2D DIC set up was used. Displacement was obtained by adopting the DIC procedure which was explained in section 2.7. The obtained tunnel displacement for 0.1g and 0.2g shaking condition is shown in Fig. 7. It can be observed from the Fig 7 that displacement of tunnel found to increase with increase in input motion. During 0.1g input motion, the maximum displacement of tunnel was found to be 2.2 mm whereas 5.14 mm tunnel displacement was observed during subsequent repeated shaking respectively. The maximum displacement during repeated shaking event of 0.2g input motion was about more than 2 times when compared to 0.1g input motion. The higher displacement of tunnel during subsequent repeated shaking highlights the influence of longer shaking events and associated soil compaction experienced during 0.1g testing. Further, the increment in tunnel displacement was mainly due to the disturbances induced by the repeated incremental acceleration loading on the compacted ground which reduces the soil confinement around the tunnel system and causing increment in displacement.

For estimating the strains developed on the tunnel, different locations such as P1, P2, P3 were selected at the top of the tunnel, P4, P5, P6 selected on the bottom of the tunnel and P7, P8 were selected at the left and right-side wall of the tunnel as shown in the Fig.4. It can be observed from the Fig 8 that strain found to be maximum at P4 i.e., at the bottom right corner of the tunnel and mid side wall of the tunnel P7 during 0.1g input motion. The maximum strain at those locations can be attributed to the high over-burden pressure from the surrounding soil which induces high stresses to the tunnel system during continuous shaking. Similar to the case of 0.1g input motion, strains were also estimated during subsequent 0.2g input motion. It can be observed that the side wall of the tunnel, P7 and P8 developed maximum incremental tension and compressive strain which is attributed to the loss of soil confinement around the tunnel due to longer shaking duration with higher acceleration loading which induces higher strains at P1 and P8 respectively.

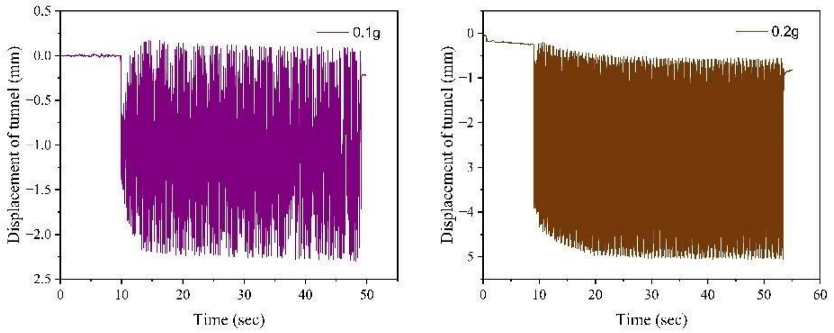

Fig.7. Displacement of tunnel obtained from DIC during 0.1g and 0.2g input motion

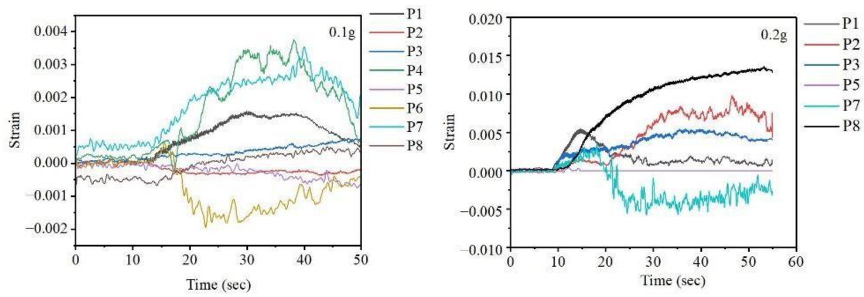

Fig.8. Strain developed at different locations of the tunnel

4 Conclusions

In this study, tunnel embedded partially saturated ground was subjected to repeated incremental shaking events. Based on the observations, the following conclusions were made.

1. When the partially saturated ground subjected to dynamic loading with longer shaking duration, soil compaction was observed which minimized generation

of pore water pressure during initial 0.1g condition. In case of tunnel embedded ground, the soil above tunnel system experiencing lesser displacement compared to adjacent soil which is away from tunnel location which is mainly due to influence of tunnel which minimize soil displacement. About 2 times increment in soil displacement was observed during 0.2g input motion at adjacent soil location which also highlights the influence of repeated shaking events and tunnel embedment in soil displacement.

2. The generated pore water pressures during repeated shaking event also highlight the need for drainage measures in the ground to improve the stability of tunnel system during repeated shaking events. About 1.5 to 3.5 times increment in generation of pore water pressures was observed during repeated shaking events. Due to this, soil confinement around the tunnel reduces which induces tunnel displacement during repeated shaking events
3. Both tunnel displacement and strain development in tunnel increases during repeated incremental loading. As mentioned, the loss of soil confinement around the tunnel during repeated shaking result in variation in stress transfer between the tunnel-soil system causing tunnel displacement and strain development.

Thus, from the above observations, it can be concluded that dynamic soil tunnel interaction under repeated loading need to be incorporated during tunnel design which improve the seismic performance of the tunnel system and also minimize damages incurred to the tunnel system in case of dynamic loading events.

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