# Liquefaction Potential Analysis and Possible Remedial Measure for Existing Structure in Kathmandu Valley

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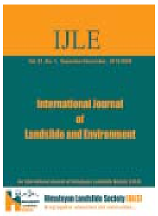


ORIGINAL ARTICLE
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## Abstract

Evaluation of liquefaction potential of soils using empirical relations based on field in-situ test data is a common practice in liquefaction study.In this study, Standard Penetration Test (SPT) datawere used to examine the liquefaction potential of the Kathmandu Valley. In total 66 SPT data among the collected 102 from 33 locations were used for the analysis. The factor of safety against liquefaction ($F_L$)was calculatedand found less than unity ($F_L$<1) in 48 locations, which indicates the high probability of occurrence of liquefaction during the predicted scenario earthquake of magnitude 8 with peak ground acceleration 300gal.

Similarly, mineralogical composition and physical index of field soil were compared to the commercially available sand and found more comparable with Toyoura sand.Two Centrifuge models in which one is saturated foundation soil model (Case1) and the other is desaturated foundation soil model by air injection (Case2) using the Toyoura sand were prepared in the laboratory. Both the models consisted of a metal plate at the top (representing the existing building structure in Kathmandu Valley) which is imparting the average contact pressure of 35kPa.The model was then set on the centrifuge. The air was injected in case2 through the injector at centrifuge acceleration 50g. The estimated residual degree of saturation at desaturated area was 85% in case2. Both the models were tested in the centrifuge at 50g acceleration with imparting a simulated sinusoidal wave of frequency 40Hz and typical acceleration amplitude of 190gal. The test results showed that excess pore pressure was significantly reduced from 65kPa in case1 to 7.5kPa in case2 at the same location of the model.Similarly, vertical settlement is reduced approximately 50% in the case2 as compared with the case1. Test results of this study show the desaturation by air injection technique can be a better solution to control the foundation soil liquefaction and save the thousands of structure standing over it in Kathmandu valley.

Keywords: Liquefaction potential, Liquefaction countermeasure, Centrifuge test, Building

## 1. INTRODUCTION

Kathmandu valley lies in the center of the seismically active Himalayan arc. The study carried out so far showed a big earthquake expected in the near future and this may cause huge damages in historically important structures existing in the valley. Kathmandu Valley mainly formed by the lake deposit consists of the saturated sand layer at shallow depth in different locations. Due to the loose sand deposit with shallow ground water table, occurrences of the liquefaction have been anticipated, but research efforts that have devoted to this topic in the past are limited.Continental collision of the Eurasian and Indianplates is the source of earthquake in Nepal, initiated about 40-55 Ma ago. The collision was followed by sub-duction of the Indian plate underneath Tibet, which is still continuing at an estimated rate of about 1.8 cm per year (Bilham, 2004). Thesub-duction results in tectonic stresses along the fault system parallel to the Himalayan arc.Numerous earthquakes have occurred in this region, including four major earthquakes of magnitude greater than M8 within the last 100 years (Seeber et al., 1981; Molnar, 1984; and Chandra, 1992). The seismic record of the region, which extends back to 1255 AD, suggests that earthquakes of thissize occur approximately every 75years, indicating that a devastating earthquake is inevitable in long term and likely in the near future (Dixit et al., 2000). The study carried out in the Kathmandu valley in different time shows that there areliquefaction susceptible are as where historic buildings are exist (JICA, 2002; Piya et al., 2004; Shrestha et al., 2004, Dixit et al., 2013). The explained evidences after the 1934 earthquake in the book entitled "Nepal KoMahabhukampa" (Thegreat earthquake of Nepal, 1935) indicates that wide spread liquefaction had occurred in the loose sand deposited are as of the Valley (Rana, 1935). All this study and past evidence show that there is a high chance of occurrences of liquefaction induced damages to existing structures in future earthquakes. Therefore, strengthening of predicted liquefiable soil under the existing structures is prime need to save its traditional histories and architects of Kathmandu Valley. This study puts an effort to make liquefaction hazard analysis by using the empirical relations based on the field in-situ test data in Kathmandu Valley. Similarly, desaturation by air injection technique is introduced to strengthen the liquefiable foundation soil layer of existing residential building. Desaturation by air injection is a technique in which properties of soil are changed by injecting the air artificially into the liquefiable layer. Existence of air in the pores in soil is considered to enhance a liquefaction resistance. Air in the pore plays a role of absorbing generated excess pore pressures by reducing its volume. The bulk modulus and change in volume of the pore fluid, that is, air water mixture, may be the factors dominating this mechanism. Considering a soil mass of its pore filled with air and water, for a small change in the pore pressure, ΔP, we obtain the volumetric strain of the soil using the Boyle's law as,

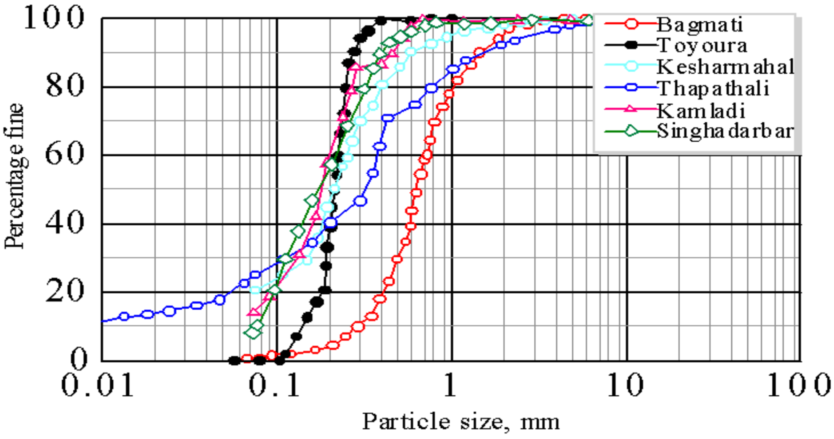
$$\varepsilon v = \frac{\Delta p}{p_0 + \Delta p}\left(1 - Sr\right)\frac{e}{1+e} \le \frac{\sigma_c'}{p_0 + \sigma_c'}\left(1 - Sr\right)\frac{e}{1+e} = \varepsilon v^{*} \qquad (1)$$

Where, equation and e denote the initial effective confining pressure, the initial absolute pressure of the fluid, and the void ratio of the soil mass, respectively. Okamura and Soga (2006) derived this influential factor of the liquefaction resistance of the partially saturated sand from theoreticalapproach and verified through a series of triaxial tests on a clean sand. A unique relationship between liquefaction resistance ratio and the potential volumetricstrain $\varepsilon v^{*}$ is found.The study carried out so far shows the effectiveness of the air injection technique to desaturate the sandy soil and make it unsaturated for a long time (Okamura et al. 2011). Past test results show that the capacity of the desaturated sand of liquefaction resistance was quite high as compared to the saturated sand. In this study, physical modeling in centrifuge was performed simultaneously with hazard analysis to observe the performance of desaturation technique by air injection as a countermeasure technique to the foundation soil of residential building. The centrifuge models were prepared based on the findings of the quantitative analysis carried out by using the collected SPT value (N) and residential building details from the study area (Kathamndu Valley of Nepal). Factor of safety ($F_L$) against liquefaction was calculated by using the SPT value (N) with scenario earthquake of magnitude 8.0 and peak ground acceleration 300gal in some part of the Kathmandu Valley (JICA, 2002). The centrifuge model in the laboratory also represents the typical ground section of the Kathmandu Valley. The index properties, grain size distribution as shown in Fig. 1 and mineralogical contents of the field, sand were compared to the commercially available sand to prepare the representative physical model in the laboratory. It was found that Toyoura sand commonly used for research work has a quite comparable properties with the field sand.Toyoura sand was used to prepare the physical model in this study.

## 2. QUANTITATIVE ANALYSIS

Empirical methods based on field exploration data are simple, easy and less time consuming for evaluating the liquefaction potential of soil deposit quantitatively. These methods are commonly based on field penetration tests that can be correlated to the cyclic shear resistance of the in situ soil. In situ penetration tests are also preferred because field measurements provide an economical indication of deposit variability (Seed and De Alba 1986). For this reason, empirical methods based on in situ penetration test are always preferred for assessment of liquefaction potential.

There are several empirical relations which are proposed by different researchers based on in situ test data. The empirical relation recommended by Youd et al. (2001) for assessment of liquefaction potential based on the in situ test are one of the most reliable ones. This recommendation came after thoroughly reviewed and made consensus on existing empirical relations among 20 liquefaction experts through the 1996 and 1998 workshop sponsored by the National Center for Earthquake Engineering Research (NCEER). They reviewed and developed the recommendation on empirical relations based on; (1) Standard Penetration Tests (SPT) blow count, (2) Cone Penetration Tests (CPT) tip resistance, (3) in situ shear wave velocity and (4) Becker Penetration Test (BPT) blow count. The workshop also reviewed and made recommendations on the magnitude scalingfactors, correction factors for overburden pressures and sloping ground and input values for earthquake magnitude and peak acceleration.In this study, empirical relation based on SPT blow count recommended by Youd et al. (2001) is used to calculate the factor of safety ($F_L$) against liquefaction at different soil layer deposit in a specific site location. For seismic motion Mid Nepal Earthquake was taken which is the scenario earthquake model developed by JICA in their study on "Earthquake




| SN | Soil type | Grain size, mm | | | Remarks |
|---|---|---|---|---|---|
| | | $D_{60}$ | $D_{30}$ | $D_{10}$ | |
| 1 | Toyoura sand | 0.24 | .19 | .14 | |
| 2 | Bagmati sand | 0.7 | 0.5 | 0.28 | |
| 3 | Kesharmahal | 0.25 | 0.15 | .075 | |
| 4 | Singhadarbar | 0.48 | 0.12 | .01 | |
| 5 | Kamladi | 0.2 | .14 | .065 | |
| 6 | Thapathali | 0.23 | .12 | .075 | |

Fig. 1: Grain size distribution curves of toyoura sand and field sand

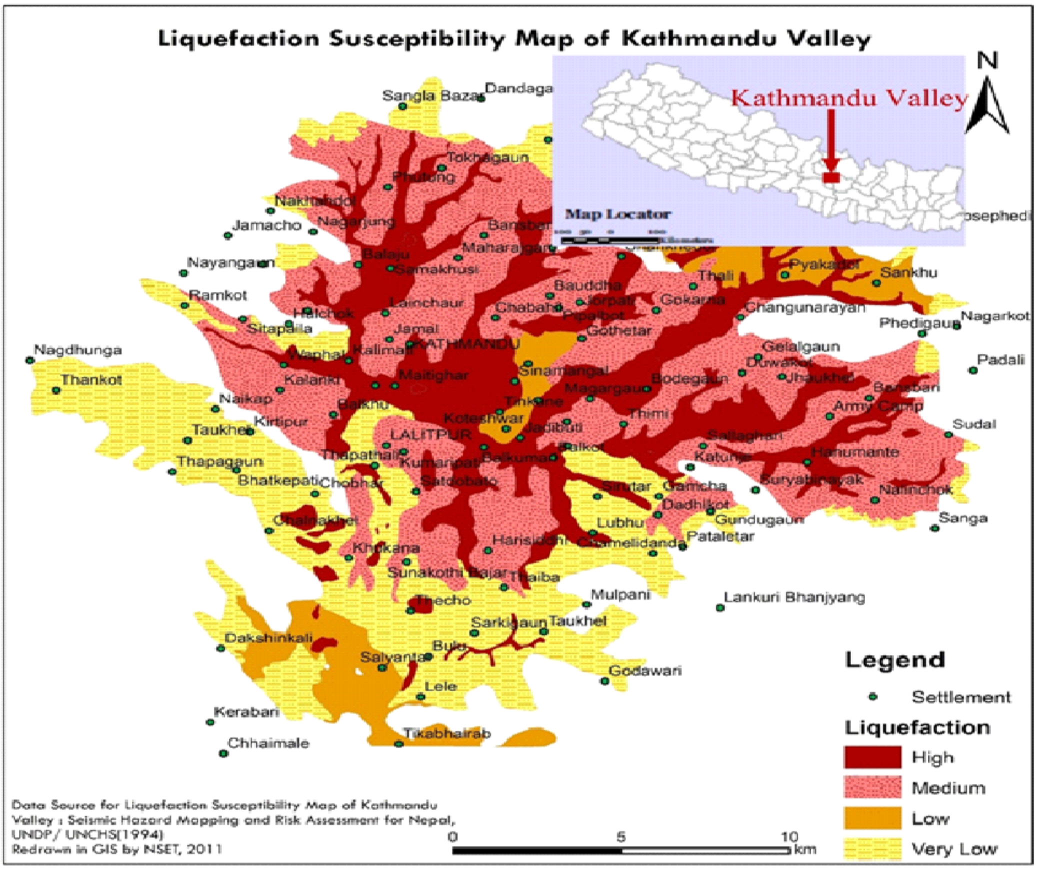



Fig. 2: Liquefaction hazard map of Kathmandu Valley (UNDP/UNCHS, 1994)

Disaster Mitigation in Kathmandu Valley" in 2000-2002. The magnitude of this scenario earthquake is M8 and peak ground acceleration is 300gal in some part of the Kathmandu Valley (JICA, 2002).

### 2.1 Data collection

Standard Penetration Test (SPT) was carried out in different locations of Kathmandu Valley in the past. The purpose of the most of the SPT was to obtain the bearing capacity of soil for infrastructure design and construction in specific locations. The available SPT details have a different bore-hole depth which is mainly depends on the types of structure planned to construct over it. To assess the liquefaction potential of any specific site quantitatively, SPT details of shallow depth (less than 30m) is used in practice. So, in this study available SPT detail in the range of 9m-30m depth were collected and used in the analysis.

In total 102 boreholes SPT data from 33 locations of Kathmandu Valley were collected and used in the analysis. The liquefaction hazard map prepared by UNDP/UNCHS in 1994 was used as a base map to screen the collected SPT borehole data for quantitative analysis. Those borehole locations which are in the high and medium liquefaction susceptible zone in the map were considered for the calculation as depicted in Fig.2. So, altogether 66 borehole locations were selected for quantitative analysis on the basis of this first screening process. The collected borehole locations are as depicted in Fig.3.

### 2.2 Quantitative Analysis Based on SPT blow count

After disastrous earthquake in Alaska and in Niigata, in 1964, Seed and Idriss (1971) developed and published empirical methodology based on field data, termed the "simplified procedure" for evaluating the liquefaction resistance of soils. This is the most common procedure used in practice for liquefaction potential analysis throughout the world. This procedure has been modified and improved periodically by different researchers since that time. In 1996 and 1998 National Center for Earthquake Engineering Research (NCEER) organized a workshop within 20 liquefaction export to gain consensus on updates and expansion that should be made to standard procedures that have evolved over the past 30 years. The outcome of the workshops came as a summary report on evaluation of liquefaction resistance of soils in 2001 with Youd and Idriss as authors and all other participants named as co-authors. The summary report includes the recommendation for the empirical analysis based on Standard Penetration Test (SPT). In this study quantitative analysis through the empirical relations based on SPT data suggested by Youd et al. (2001) is used for the calculation.

The simplified procedure updated and recommended by Youd et al. (2001) can be comprises the following four steps.

#### 2.2.1 Identify the potentially liquefiable layers to be analyzed

The soil layer above the ground water table does not need to include in the liquefaction potential analysis. Similarly, Soil layer which has an SPT N value greater than 30 normally do not liquefy during earthquake. These are the basis to identify the potential liquefiable layers to be analyzed.

#### 2.2.2 Calculation of seismic demand expressed as CSR (Cyclic Stress Ratio)

Cyclic Stress Ratio (CSR) was calculated using the relation formulated by Seed and Idriss (1971) and Suggested by Youd et al. (2001)

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \qquad (2)$$

Where, $a_{max}$ = peak horizontal acceleration at the ground surface generated by the earthquake, g = acceleration due to gravity, $\sigma_{vo}$ and $\sigma'_{vo}$ are total and effective vertical overburden stress, respectively; and $r_d$ = stress reduction coefficient $a_{max}$ depends on the magnitude of the earthquake, epicentral distance from the rupture zone and soil type. The earthquake magnitude of 8.0 and 300gal acceleration as reported on JICA (2002) Mid Nepal Earthquake Model was used for the analysis. $r_d$ stress reduction coefficient in Eq. (2) describe the flexibility of the soil profile. To calculate the, Seed and Idriss (1971) developed world wise use curve of verses depth. As reported by Youd et al. (2001), Liao and Whiteman (1986) developed a new relationship to calculate the $r_d$ which latter on approximate by Blake (1996) for ease of computation with the following equation.

$$r_d = \frac{\left(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{2}\right)}{\left(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^{2}\right)} \qquad (3)$$

Where, z is the depth below ground surface in meters.

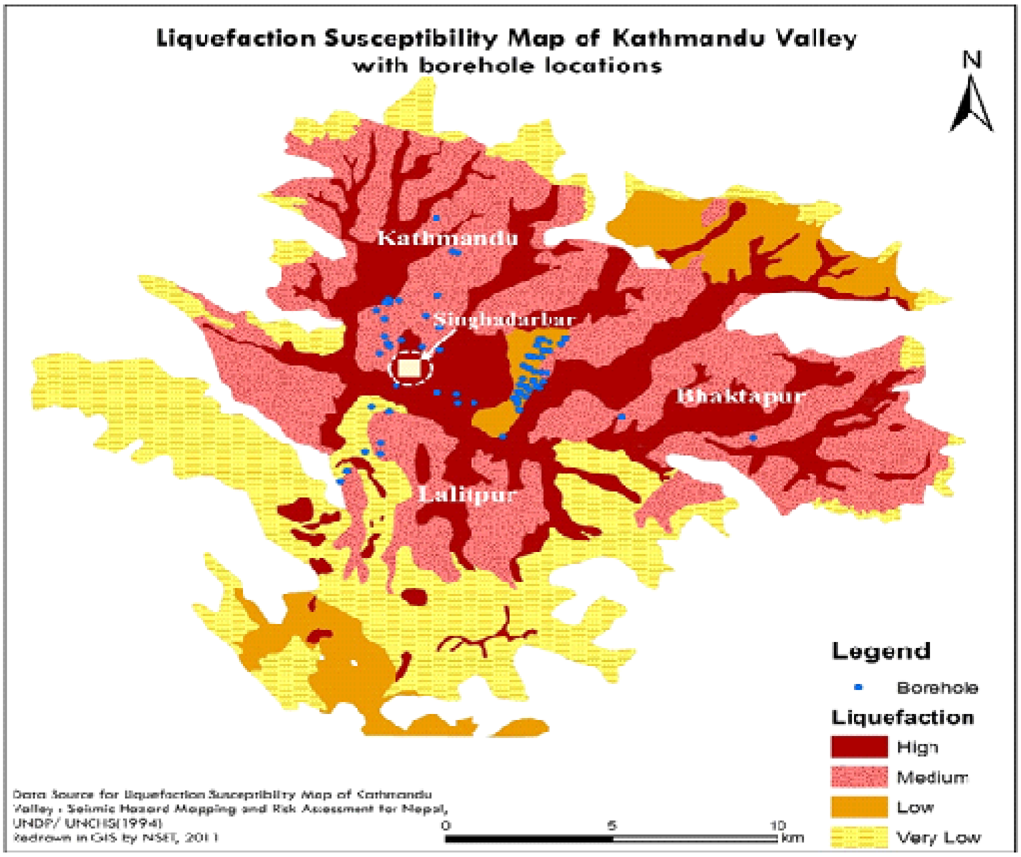



Fig. 3: Borehole location map (UNDP/UNCHS, 1994)

### 2.2.3 Calculation of soil layer capacity expressed as CRR (Cyclic Resistance Ratio)

Based on the characteristics of the potentially liquefiable soil layer (eg. density, fine contents, measured SPT N value), the cyclic resistance ratio (CRR) can be determined by using the empirical relation developed by different researchers and recommended by Youd et al. (2001).

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_s \tag{4}$$

The measured SPT N value of the field test was corrected by the following equation which was modified from Skempton (1986) and as listed by Robertson and wide (1998) and recommended by Youd et al. (2001).Where $(N_1)_{60}$ = Corrected or normalized SPTN value, $N_m$ = measured SPTN value, $C_N$ = Correction factor for overburden stress,$C_E$ = Correction for hammer energy ratio (ER),$C_B$ = Correction factor for borehole diameter,$C_R$ = Correction factor for rod length and $C_S$ = correction for samplers with or without liners. SPT N- values increase with increasing effective overburden stress, an overburden stress correction factor was estimated by the equation modified from Skempton 1986 listed by Robertson and Wride (1998) and recommended by Youd et al. (2001).

$$C_N = \left( \frac{P_a}{\sigma'_{v0}} \right)^{0.5} \tag{5}$$

Where, $P_a$ is atmospheric pressure equals to approximately 100 kPa (1 atm) and the value of$C_N$ is not exceeded and limited to 1.7. Values of other correction factors included in Eq. (4) such as $C_E$ = Energy ratio for safety hammer is 0.7-1.2, Correction factor for energy ratio is taken as 1, $C_B$ = 1 for bore hole diameter 100 mm, $C_R$ = 0.85 for rod length 4 m to 6 m and $C_S$ = 1 for sampler without liners listed in Robertson and Wride (1998) were used in this study. Cyclic resistance ratio (CRR) is influenced by fine contents in the soil layer. As noted by Seed et al. (1985) CRR increases with the increase of fine contents in the soil layer. The equation developed by Idriss with the assistance of Seed (Youd and Idriss, 1997) for correction of $(N_1)_{60}$ to an equivalent clean sand value considering the fine contents as cited by Youd et al. (2001) was used in the analysis.

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60} \tag{6}$$

Where, $\alpha$ and $\beta$ =coefficients determined from the following relationship

$$\alpha = 0 \; for FC \le 5\% \tag{6.1}$$

$$\alpha = \exp\left[1.76 - \left(\frac{190}{FC^2}\right)\right] \quad \textit{For } 5\% < FC < 35\% \tag{6.2}$$

$$\alpha = 5 \; for FC \;\; \geq 35\,\% \tag{6.3}$$

$$\beta = 1 \; for FC \;\; \leq 5\% \tag{6.4}$$

$$\beta = \left[0.99 + \left(\frac{FC^{1.5}}{1000.00}\right)\right] \quad \textit{For } 5\% < FC < 35\% \tag{6.5}$$

$$\beta = 1.2 \; for FC \geq 35\% \tag{6.6}$$

Cyclic Resistance Ratio (CRR) for clean sand derived for 7.5 magnitude earthquakes was calculated by the following equation (Rauch, 1998).

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10.(N_1)_{60cs}+45]^2} - \frac{1}{200} \tag{7}$$

The eq. (7) is valid for$(N_1)_{60cs}$<30.For $(N_1)_{60cs} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable (Youd et al. 2001).

### 2.2.4 Calculation of the factor of safety against liquefaction ($F_L$) (resisting force divided by driving force).

The clean sand based equation (7) is only applies to magnitude 7.5 earthquakes. To apply this equation to magnitudes smaller or larger than 7.5, Seed (1983) introduced correction factors termed "magnitude scaling factors (MSFs). Similarly, Youd et al. (2001) recommended the equation to calculate the factor of safety ($F_L$) against liquefaction including magnitude scaling factors and correction factors for larger overburden pressure with static shear stress conditions in sloping ground.

$$\textit{Factor of Safety } (F_L) = \frac{CRR_{7.5}}{CSR} \cdot MSF \cdot K_\sigma \cdot K_\alpha \tag{8}$$

Where, MSF is the magnitude scaling factor. For earthquake magnitude others than 7.5 we need to modify factor of safety by multiplying MSF. The value of MSF for lower bound (M<7.5) and upper bound (M>7.5) as recommended by Youd et al. (2001) was used in the analysis.

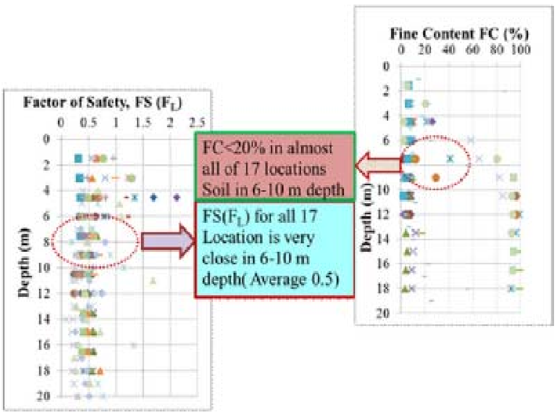
$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad \textit{for } M < 7.5 \tag{9}$$

$$MSF = \left(\frac{M_w}{7.5}\right)^{-2.56} \quad \textit{for } M > 7.5 \tag{10}$$

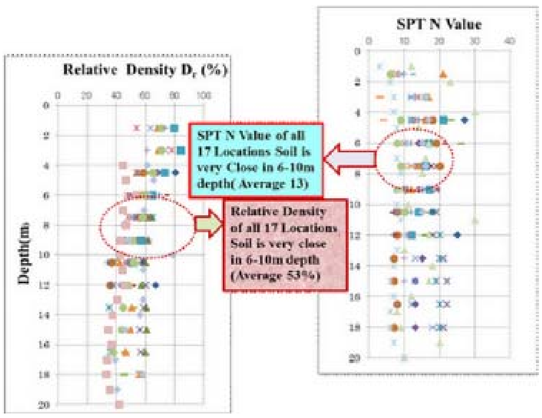
$K\sigma$ is the correction factor for large effective overburden and $K\alpha$ is correction factors for sloping ground. As mentioned by Youd et al. (2001) the application of  and in a simplified procedure beyond routine practice and require specialized expertise therefore these two factors are not considered in this study.

### 2.3 Quantitative Result analysis and Discussion

In total, 66 boreholes of collected 102 from 33 locations of the Kathmandu Valley were considered for analysis. Factor of safety ($F_L$) against liquefaction at different soil layer deposit was calculated and analyzed for each borehole. Factor of Safety was calculated for the soil layer deposit below the ground water table. Among 66 boreholes, the factor of safety ($F_L$) of the 18 boreholes found greater than 1 which means safe from liquefaction. The rest of the 48 borehole locations have a factor of safety less than 1 in almost all layers of soil deposit so these borehole locations considered as unsafe from the liquefaction point of view.




(a) $F_L$ and FC with depth




(b) Dr and SPT N Value with Depth

Fig. 4: Quantitative analysis findings

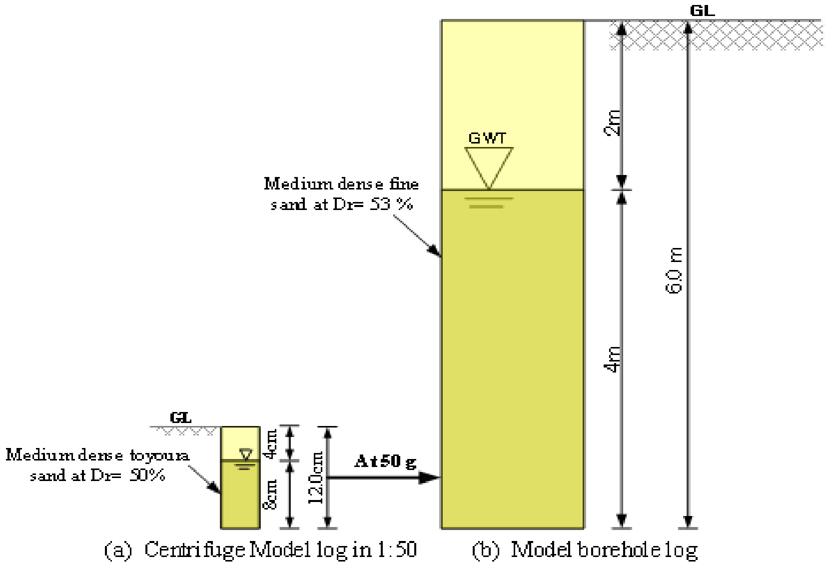



Fig.5:Ground section for centrifuge modelling

Factor of safety ($F_L$) verses depth, Relative Density ($D_r$) Verses depth, Fine content verses depth and SPT N value verses depth relations were plotted and analyzed for all 48 boreholes where liquefaction potential seen from the quantitative analysis. The plotted relation between the factor of safety ($F_L$) and depth shows that there is more or less same factor of safety (average 0.5) for 6 to 10m depth in 17 locations of the Singhdarbar area. The plotted relation between SPTN value and depth showed the measured SPTN value is more or less similar (average 13) between 6-10 m depth in 17 locations of the Singhdarbar area. Similarly, the plotted relation between relative density (Dr) and depth showed that 17 locations in Singhdarbar area have a quite similar relative density at 6-10 m depth (average 53%). The findings of the quantitative analysis area as depicted in Fig.4(a) and (b). It showed that the soil deposit exists in 6-10m depth of Singhdarbar area can be a representative soil to carry out the further study in Kathmandu Valley. Fig.3 shows the location of the Singhadarbar in the liquefaction hazard map of the Kathmandu valley. So, in this study the properties of this layer soil wastaken as a reference soil to reproduce the model in the laboratory for centrifuge test.

## 3 PHYSICAL MODELLING IN A CENTRIFUGE

The centrifugal model tests enable considerable cost saving in terms of total quantity of materials, labor and time spent in model preparation compared to large-scale shaking table test (Ling et al. 2003).

In this study models were prepared in 1: 50 scale in the laboratory as depicted in Fig. 5 and tested on centrifuge by imparting the predicted seismic event at 50g centrifuge acceleration. The test findings came out through the saturated and desaturated (by air injection) model tests were compared and evaluated the desaturation by the air injection technique as a liquefaction countermeasure technique for foundation soil of existing structure.

### 3.1 Model Preparation in the Laboratory

Two models were prepared as shown in Fig.6 (a) and 3(b) in which one is saturated foundation soil model (Case 1) and the other is desaturated foundation soil model by air injection (case 2). The model structure consisted of a metal plate at the top (representing the building structure) which is imparting the average contact pressure of 35kPa to the 6m (in prototype scale) deep loose liquefiable sand bed.

Rigid container (430mm long x 120mm wide x 230mm deep) was used to make model in this study. The front face of the container was transparent so that the sand layers are distinctly observed.

In model, 6 meters (in prototype scale) foundation soil layer was produced in 1:50 scale with planning to test the model in 50g centrifuge acceleration. The bottom layer (2cm in model scale) of the model is prepared for relative density 90% as a dense layer and considered not to liquefy during the test. The densification by surface tamping method was used to make a dense layer of relative density 90%. In case 2, two dimensional air injector with a 1mm wide orifice at both sides was placed on this dense sand layer. The dry sand was then rained into the container to a 12cm (in model scale) to provide uniform sand deposit with the relative density Dr =50%.

As per the reference soil layer of field ground water table is in 2 meters depth, which means top 2 meter soil layer is an unsaturated layer and do not liquefy during earthquake shaking. Accordingly, in model, four layers(2cm each in model scale) considered as liquefied layer and put the accelerometer and pore pressure sensors as positioned in the conceptualized model as shown in Fig.6(a) and (b).Since the top two layers (2cm in each in model scale) are unsaturated layer, only accelerometers were placed.

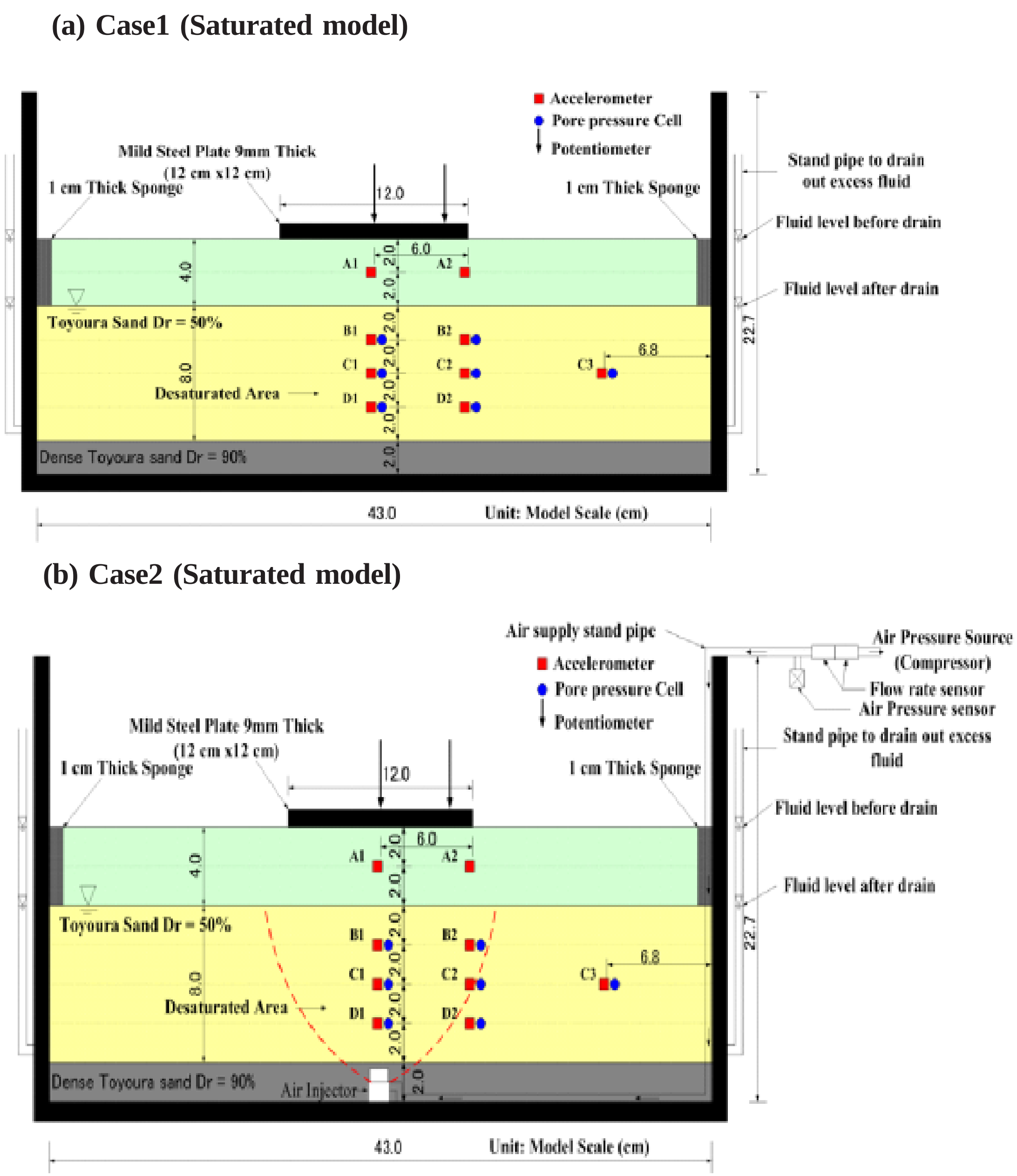



Fig.6: Centrifuge Models tested at 50g

### 3.2 Model Saturation and Measurement of Degree of Saturation (Sr)

The degree of saturation is a critical parameter for liquefaction study of the granular soil. Little variation in the degree of saturation will have the significant effects on the liquefaction resistance capacity of the soil. In fact, naturally deposited saturated sand have almost fully saturated condition, so it is necessary to apply the most reliable method in the saturation process so that the fully saturated condition can be ensured in the laboratory and can make the model identicalto the field condition. In this study the technique suggested by Okamura and Inoue (2010) was used to make the model fully saturated.

The prepared model was transferred and put into the pressure chamber to start the saturation process. At first, -100kPa vacuum pressure was applied to the pressure chamber and then carbon dioxide ($CO_2$) gas was imparted to replace the air bubbles remains in the chamber and model. The $CO_2$ replacement technique used to enhance the degree of saturation because of its fast dissolved characteristic in a fluid. Again the chamber was vacuumed at -100kPa and then de-aired viscous fluid (50 cst) was poured into the model. The viscous fluid was a mixture of water and hydroxypropyl methylcellulose called metolose. This metolose solution pore fluid was prepared by dissolving 2% metolose by weight in water, to achieve a viscosity of about 50 times the viscosity of water.

During the earthquakes and liquefaction there are two physical phenomena happening at the same time, one is the vibration of the model and the other is the pore pressure dissipation controlled by the seepage velocity. In a centrifuge, it is desirable that there is a constant time scaling law for the movement and seepage but the times laws are not the same in movement ($t_p/t_m = N$) and seepage ($t_p/t_m = N^2$). In centrifuge modeling, it is normal practice to change the viscosity of the pore fluid by the same value as the level of g so that the same time scale law can be maintained in both the phenomena.

The degree of saturation (Sr) was measured with the method developed by Okamura and Inoue (2010). In this method, the model was considered as a submerged model ground with a level surface in a pressure chamber. Assuming air in the void as an ideal gas, the change in the air volume in the model can be related to the change in the air pressure in the chamber. It means if the pressure in the chamber is increased from P1 to P2 relation can be expressed as;

$$\frac{P_1}{P_2} = \frac{V_1}{V_2} = \frac{V_1 + \Delta V}{V_1} = 1 + \frac{\Delta h}{V_1} A \tag{11}$$

Where V1 and V2 indicate the volume of air at chamber for absolute pressure P1 and P2 respectively, A is the area of the model container and Dh is the change of the water table. The volume of the air in the model can be obtained by knowing the change in the pressure and water table and can be expressed as;

$$V_1 = \frac{\Delta h}{\left(\frac{P_1}{P_2}\right) - 1} A \tag{12}$$

In this study Light-emitting diode (LED) sensor was used to measure the displacement in the model during the pressure change from P1 to P2 in the pressure chamber. LED sensors have a very high resolution which can measure displacement up to 10mm. The set-up for measurement of degree of saturation was as shown in Fig. 7.

In this test, the displacement transducer (LED) and pressure transducer were set in the centrifuge to record the displacement duringthe pressure change. At first -10kPa vacuumpressure was applied to the pressure chamber and keeps some time to stabilizing it. After that, the pressure was increased from -10kPa to -16kPa in a three minute.Change in the water level resulted from thecompression of air bubbles was recorded in the computer with the pressure change in the chamber. The degree of saturation was then calculated from the relation as;

$$Sr = \left(1 - \frac{Va}{Vv}\right) * 100\% \tag{13}$$

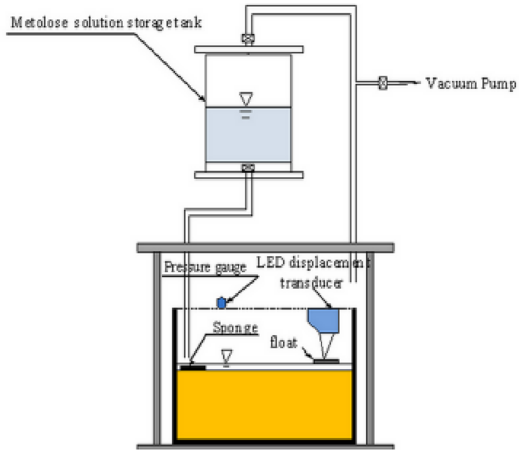



Fig. 7: Set-up for measurement of degree of saturation ($S_r$)

Where, Va is the volume of air equal to V1 at equation 12 and Vv is the volume of void which can obtained by knowing the relative density, weight, specific gravity and total volume of the soil. The measured initial degree of the satu-

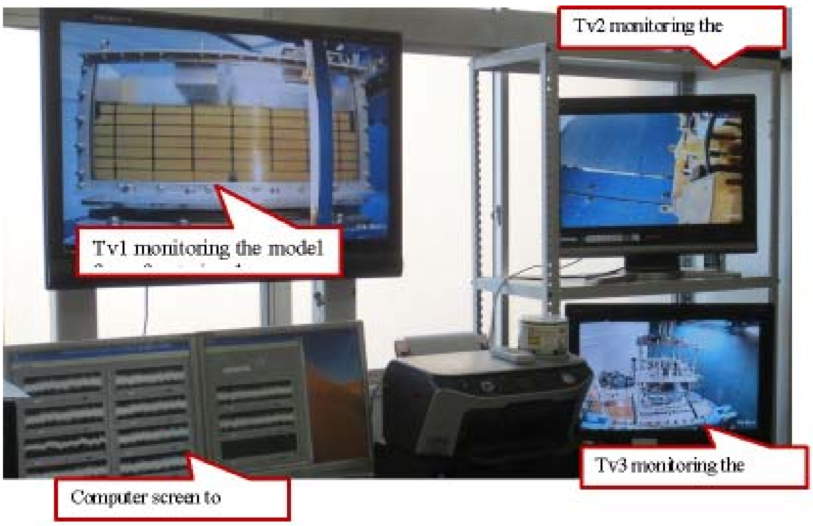



Fig. 8: Centrifuge Model setup in the laboratory

ration in this study was 99.7 % in saturated model and 99.8 % in desaturated model.

### 3.3 Model Setup and Test Conditions in Centrifuge

The saturated model was then set on the centrifuge. On the top of the soil layer 35kPa surcharge weight representing building,weight by mild steel plate was placed. Centrifuge acceleration was slowly increased bykeeping, monitoring the sensor responseson the computer screen and model condition with the help of on-board video camera. After the centrifuge acceleration was reached at 40g, the acceleration was kept constant for ample time to drain outthe excess pore fluid until the height of water table coincided to the level 4cm (in model scale) below the ground surface. The centrifuge acceleration was then again increased and run in a 50g after the pore fluid comes with a target level.Centrifugelaboratorysetup is as depicted in Fig.8.

#### 3.3.1 Air injection

Air was injected in case2 at 50g through the injector set at the bottom of the dense sand layer as shown in Fig.6 (b). Air pressure was supplied through the compressor connected with a centrifuge. The air entry value (AEV) ofToyoura sand is 2.5kPa. The air injection pressure increase rate was slow.so that the time of air flow beginning in the model can be well-detected and proper control on the air pressure supply can be made. The special attention was made on the air injection pressure so that the soil grain will not be disturbed during the air flow around the injector. The maximum injection pressure that can disturb the soil grain is $(P_{inj})_{max} = P_{hyd} + 0.5s_v'$ (Ogata and Okamura 2006) where, $P_{hyd}$ and $s_v'$ denote the hydrostatic pressure and vertical effective stress just above the air injector placed at the base of the model container. Fig.9indicates the time histories of air pressure supplied to the injector, airflow rate and change in pore pressure measured at location C3 of the model in case2. The flow rate and water level started rising at t= 1150sec., indicating that air began to flow into the soil. This timing coincided when the air pressure reached $(P_{inj})_{min} = P_{hyd} + AEV$ as shown in Fig9, where $P_{hyd}$ and AEV denote the hydrostatic pressure at the depth of the injector and the air entry value of the soil, respectively. The water level rose as the injection pressure was increased. When the air injection was halted at t= 1740sec. The pore pressure settles to a residual pressure 1.75kPa higher than that before the air injection.The rise in the water level is equivalent to a volume of air in the soil. It was observed by an on-board video camera during the air injection that the color of the sand in the desaturated area was changed. This area, shown in Fig.6 (b) was consistentwith that obtained by detailed eye observation after the centrifuge was stopped after the shaking test. The residual degree of saturation in the desaturated areas was approximately 85% in case 2.

#### 3.3.2 Shaking test

Hereafter in this paper, all the test results are presented in prototype scale, otherwise mentioned. On completion of excess pore fluid drain and get the fluid height at target level in case1and the desaturation process in case2, one dimensional lateral shaking was imported along the model long axis using a mechanical shaker while the centrifuge was running at 50g acceleration. A simulated sinusoidal wave of a frequency of 40Hz and typical equivalent acceleration of 190 gal was imparted in both the models. The input acceleration time history is as shown in Fig.10.

### 3.4 Result and Discussion

The test condition between case1 and case2 were same except the degree of saturation in case2 as here, it was lowered by air injection. The observed acceleration time histories at location C1 in case1 and case2 is as depicted in Fig.11(a) and (b). The response acceleration amplitude in case1 is much higher than in case2 as shown in Fig.11(a) and (b). Also in case1 phase difference was observed between the input and response acceleration, but in case2 no such difference observed. It shows that soil at C1 significantly softened in case1.

Similarly, the excess pore pressure time histories in case1 and case2 at location C1 are shown in Fig.12. It shows that in case2 developed pore pressure at location C1 is just 7.5kPa which is quite lower than the

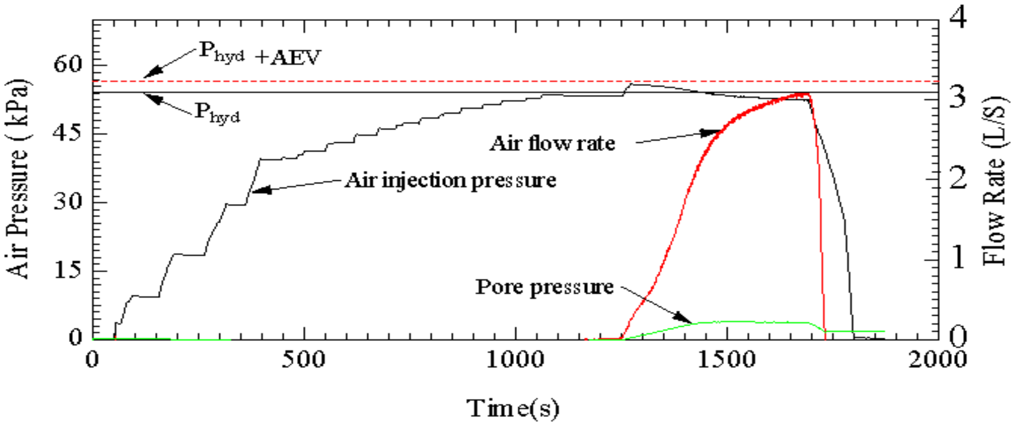



Fig. 9: Time histories of air pressure and flow rate

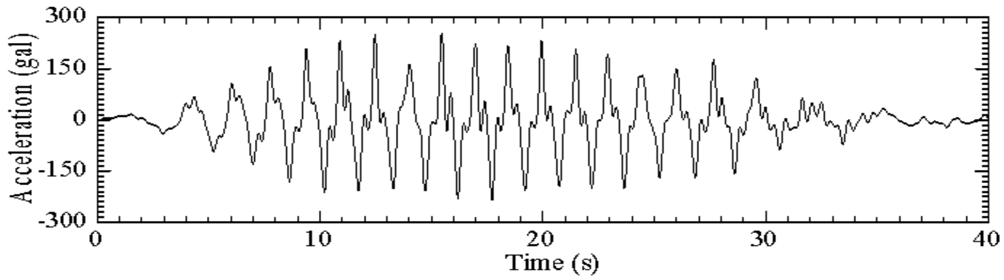



Fig. 10: Input acceleration at 40Hz-50g shaking

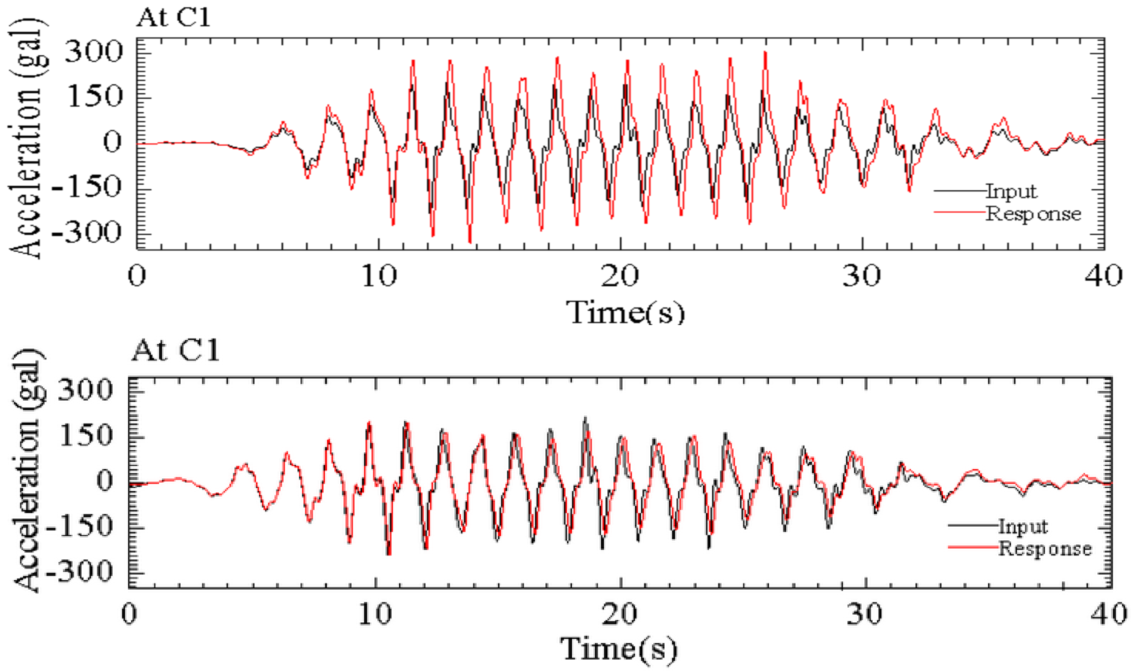



Fig. 11: Acceleration time histories at C1 in case 1 and case2

effective overburden pressure, but in the case1 excess pore pressure at C1 is 62.5kPa which is in-between the effective overburden stress with and without the structural weight as depicted in Fig.12. It showed that excess pore pressure was remarkably reduced in the area where the degree of saturation was lowered by the air injection. The distribution of the maximum excess pore pressure developed in case1 and case2 at the center ofthe structure (B1, C1 and D1)isas depicted in Fig.13. The developed excess pore pressure is much lower than the effective overburden stress ($s_v$¢) in the case2 but in-between the effective overburden stress ($s_v$¢) with and without structure in case1 as shown in Fig.13.In case2 B1, C1, and D1 located in the desaturated area as shown in Fig.6(b).

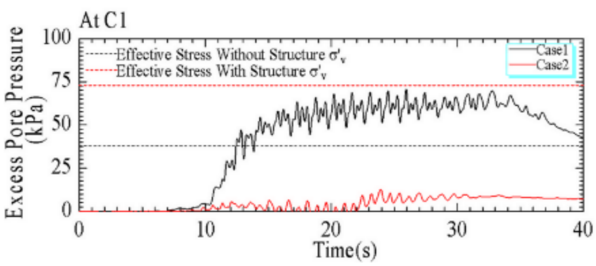



Fig. 12: Excess pore pressure at C1 in Case1 and Case2

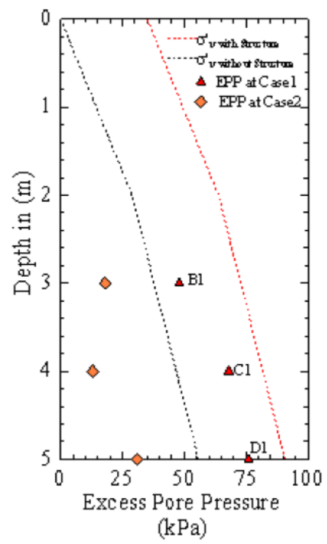



Fig. 13: Excess pore pressure distribution

The measured structure settlements in case1 and case2 are shown in Fig.14. The structure settlement at building center in case1 was 25cm, where as in case2 it is 13cm. Similarly, at the building edge measured settlement was 29cm in case1 and 16cm in case2 as shown in Fig.14.The settle-

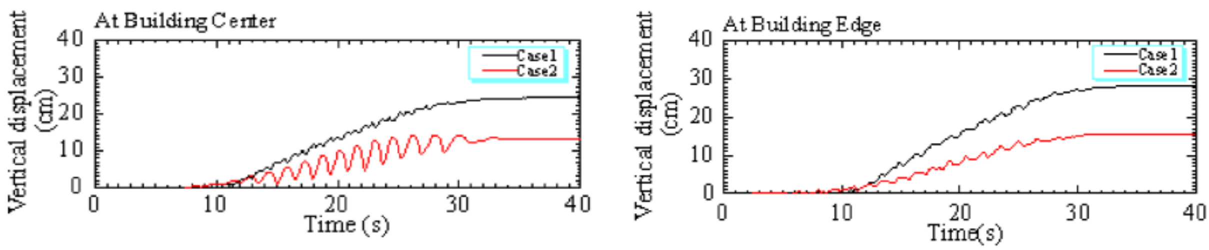



Fig.14: Vertical displacements in case 1 and case2

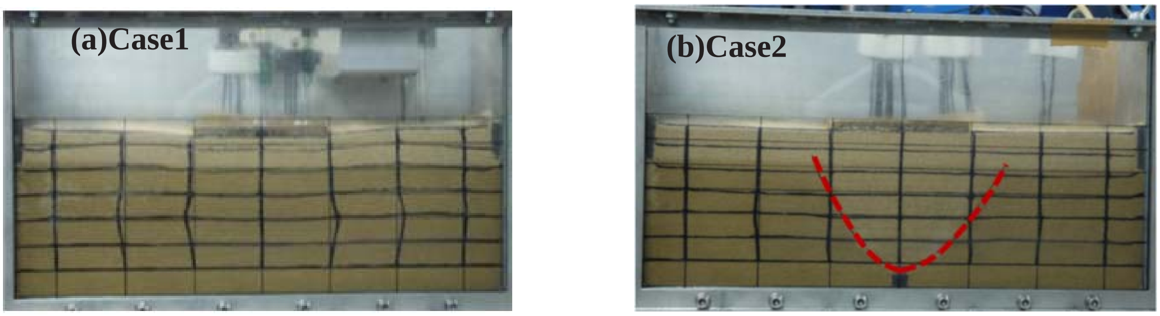



Fig. 15: Model after the test in case1 and case2

ments of structure in case2 were reduced approximately by 50% as compared to the case1. Fig.15 shows the case1 and case2 model after the shaking, where we can see the difference of distortion in the models at the same level of shaking. In case2, color sand in the desaturated area did not distort in both the horizontal and vertical axis, but in the case1 significant distortion on the color sand was observed in both horizontal and vertical axis due to the liquefaction.

## 4 CONCLUSIONS

Liquefaction hazard analysis based on the field in-situ data were carried out in this study. The factor of safety (FL) against liquefaction was calculated for every soil layer deposit of the each borehole location. Most of the selected borehole locations have $F_L$<1 indicates the potential hazard for liquefaction.

Similarly, the physical modeling carried out in the centrifuge shows the effectiveness of the desaturation technique by air injection in the foundation soil of existing structure. In this study the excess pore water was significantly reduced from 65kPa in case1 to 7.5kPa in the case2 at the same location of the model. Also the vertical settlement is reduced remarkably in the case2 as compared with the case1. It is 25cm in the structural center in case1 and reduced to 13cm in case2. The curvature in the vertical color strip shows the horizontal deformation due to liquefaction in case1 but in case2 there is no curvature seen in vertical color strip means no liquefaction occurred in this area.

The findings of this study showed the effectiveness of the air injection technique to strengthen the liquefiable soil below the building structure. It shows that the desaturation by the air injection technique can be a better solution to control the foundation soil liquefaction and save the millions of structure standing over it in Kathmandu valley.

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