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ABSTRACT

Liquefaction is one of the major causes of damage to civil engineering structures. The shear strength of sandy soil during strong earthquakes is reduced due to exerted energy that is related to increasing of pore water pressure. The absorbed energy calculated through hysteretic stress-strain loops as compared with the exerted earthquake energy is an alternative method to study the liquefaction susceptibility of saturated sandy soil. There are several numbers of remediation methods which reduce the excess pore water pressure such as gravel drains. In the current study seven precisely performed 1-g shaking table tests are conducted. Synthetic Firouzkooh sand was used as the reference soil. The effectiveness of the gravel drains in the model against liquefaction is investigated by energy method. Energy per unit volume absorbed by the soil for every test was calculated. Three gravel drain arrangements and two input motion levels are checked in this study. The results show that absorbed energy concept is an appropriate approach in this kind of complicated problems to study the gravel drains effectiveness. The consumed energy in the different soil elements has a good conformity with the generated pore pressure.

INTRODUCTION

The energy that is stored in the bedrock is released during an earthquake in the heat, friction, wave propagation and crushing circumstances. The elastic energy waves propagate through the soil medium in the form of plane P-waves and shear S-waves. This elastic energy is attenuated as the seismic wave travels through soil stratum because of wave scattering and geometry, inelastic soil behavior and the interaction of the saturated soil system. This interaction causes the nonlinear response of the surrounding soil which reduces the wave energy (Trifunac et al. 2001).

Liquefaction of soils during earthquakes has received a lot of attention among the geotechnical community, so researchers have attempted to determine the parameters that could better define the liquefaction potential of a soil deposit under random earthquake loading.

Nemat-Nasser and Shokooh (1979) introduced the energy concept for the analysis of densification and liquefaction of cohesionless soils. It is based on the idea that during deformation of these soils under dynamic loads part of the energy is dissipated into the soil. This dissipated energy is represented by the area of the hysteric shear strain-stress loop and could be determined experimentally. The accumulated dissipated energy per unit volume up to liquefaction considers both the amplitude of shear strain and the number of cycles, combining both the effects of stress and strain. Compared with other methods to evaluate liquefaction potential of soils, the energy approach is easy to deal with random loading because the amount of dissipated energy per unit volume for liquefaction is independent of loading form.

As compared with the cyclic stress-based and cyclic strain-based methods, the energy approach has the following advantages: (1) energy is a scalar quantity expressed by a single number; (2) it is not necessary to decompose the time history of shear stress to find an equivalent cycle number for selected average stress or strain level; (3) its use encompasses both stress and strain, as well as material properties (law et al. 1990); (4) energy method is related to the soil relative density and confining pressure but it is not dependent to loading form and stress path; (5) it can be related to the intrinsic motion and probable of quake.

PHYSICAL MODELING AND TEST EQUIPMENT

In the current study a series of shaking table tests were conducted on model gravel drains. The tests were performed using the shaking table and experimental facilities in the Civil Engineering Department of the University of Urmia.

Figure 1 and 2 show three dimensional view of the model and the arrangement of the gravel drains respectively. Diameter and center-to-center spacing of the drains has already designed and was 5cm and 20cm respectively. These gravel drains were sandwiched by geo-textile filters. Models were constructed in a transparent Plexiglas container of 200cm long, 50cm wide and 70cm high. The bottom of the container was covered by a fine screen mesh so that the saturation process could be performed by percolating water gradually and uniformly from the bottom of the soil box. Different types of transducers were employed to measure acceleration, pore water pressure and displacement at different positions as depicted in Figure 1. The pore pressure transducers were fixed in place to record the pore water pressures at the exact locations. However the acceleration transducers were free to move with the adjacent soil. The model foundation had dimensions of 24.5cm*45cm*5cm and was applying an overburden pressure of 4.26 kPa on the sand. A geometrical scaling factor of 1:25 can be assumed throughout these tests to model a prototype with a width of 6.1m.

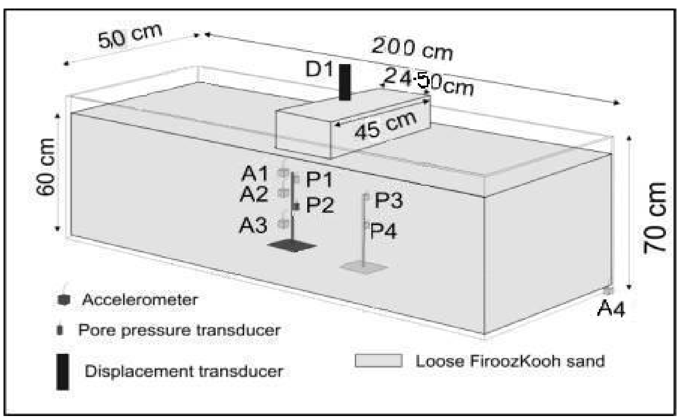

Fig.1. Three-dimensional view of the model apparatus

Table1. Physical properties of Firouzkooh sand

G_s	e_{max}	e_{min}	Cc	Cu	$D_{50(mm)}$	%FC
2.658	**0.874**	**0.548**	**0.97**	**2.58**	**0.30**	**1**

Table 1 shows the physical properties of the Firouzkooh sand. Moist tamping method, in which the sand was mixed with 5% moisture, was used to prepare a uniform soil profile. Wet Firouzkooh sand was poured inside the container and carefully tamped to a total unit weight of 14.41 kN/m^3, thus a target void ratio of 0.9 was gained for the liquefiable soil through the tests.

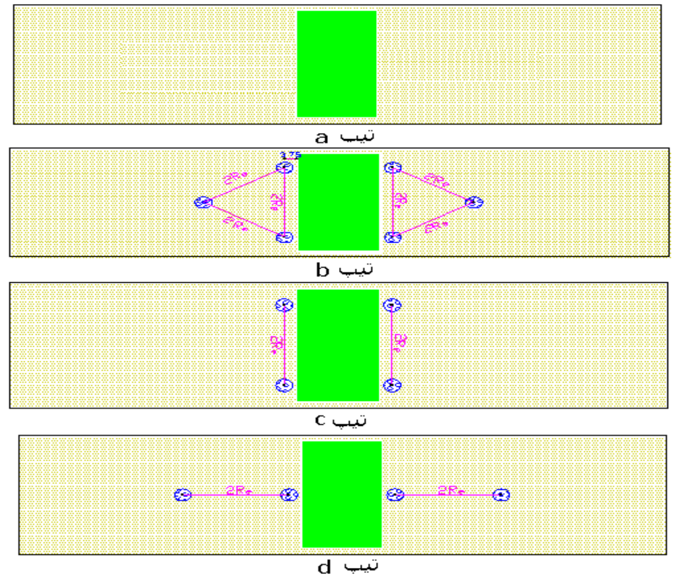

Fig.2. Schematic views of test arrangements

Dyed grid lines were created to make the behavior of model ground visible. The soil models were percolated with carbon dioxide to help dissolve the air in the void space, in order to facilitate full saturation by water. After that the model was saturated from bottom with a very slow steady flow of water in order to sustain the controlled density of the tamped sand. Shaking table of the current research was in one direction and input motion in all tests was random. Two input motion levels were exerted to the table; the first “D” series that were related to low input motions and the second “U” series that were stronger input motions. Also three different arrangement types of gravel drains were adopted from the Seed and Booker method (1977) named “b”, “c” and “d” types. The models with no reinforcement were call type “a”. Predominant frequency of shaking table was 2.7Hz and the maximum amplitudes of acceleration at the base of the table were 0.08g and 0.15g for different input motion levels.

DATA PROCESSING AND CALCULATION

The energy method is based on determination of the time history of the shear stress and strain at the location of the special measurements by the transducers for each layer. The hysteretic loops are formed and the amount of dissipated energy per unit volume can be determined for each layer up to the end of the earthquake. Numerical double integration of the acceleration time histories at the layers is lead to shear strain histories (Dief 2000, Idriss and Seed 1968).

A horizontal soil deposit is divided into N layers and N+ 1 node. Node 0 is at the bedrock and its displacement is known since the motion of the bedrock is given as input. Lumped masses are concentrated at the nodes and only have horizontal displacement. The notations used in Figure 3 and equations to calculate the masses are summarized as follows:

$$m_j = \frac{1}{2} h_j . \rho_j + \frac{1}{2} h_{j+1} . \rho_{j+1} \mapsto j = 1, N-1 \quad (1)$$

$$m_N = \frac{1}{2} h_N . \rho_N \quad (2)$$

Where: U_j = horizontal displacement of node j (j= 0,N),
h_j = thickness of the j^{th} layer (j= 1,N),
ρ_j = mass density of the j^{th} layer (j= 1,N),
m_j = mass per area on the j^{th} node

This lumped mass system, results in a group of equations which can be determined using the free body diagram shown in Figure 3, where a_j is the acceleration of the j^{th} node with mass m_j, defined by equation 3.

$$a_j = \ddot{U}_j \,(j = 1,2,\ldots,N) \quad (3)$$

Knowing the horizontal acceleration of the j^{th} node and the j^{th} mass m_j, the shear stress τ_j in the j^{th} layer can be calculated for each node from top to bottom by using the equations of motion in the form of the central difference method as follows:

$$m_N \ddot{U}_N = \tau_N \quad (4)$$

$$m_j \ddot{U}_j = \tau_j - \tau_{j+1} \quad (5)$$

Where τ_j is the shear stress in the j^{th} layer.

Also, knowing the horizontal displacements at the j^{th} node (U_j) and the thickness of the j^{th} layer (h_j), the shear strain in the j^{th} layer, γ_j can be determined (Zeghal and Elgamal, 1994):

$$\gamma_j = \frac{U_j - U_{j-1}}{h_j} \quad (6)$$

The accumulated energy per unit volume (δW) absorbed by the specimen, until it liquefies is given by Figueroa et al. (1994):

$$\delta W = \sum_{i=1}^{n-1} \frac{1}{2} (\tau_i + \tau_{i+1})(\gamma_{i+1} - \gamma_i) \quad (7)$$

Where, n is the number of points recorded to liquefaction.

Then from equations 4, 5, 6 and 7 the accumulated energy per unit volume (δW) absorbed by the specimen, until it liquefies can be determined. Because of the limitation in the instrumentation used in recording the seismic soil response, a linear interpolation of acceleration and displacement over the thickness of each layer was calculated based on the recorded motions at the top and bottom of this layer, as adopted from Zeghal and Elgamal (1994).

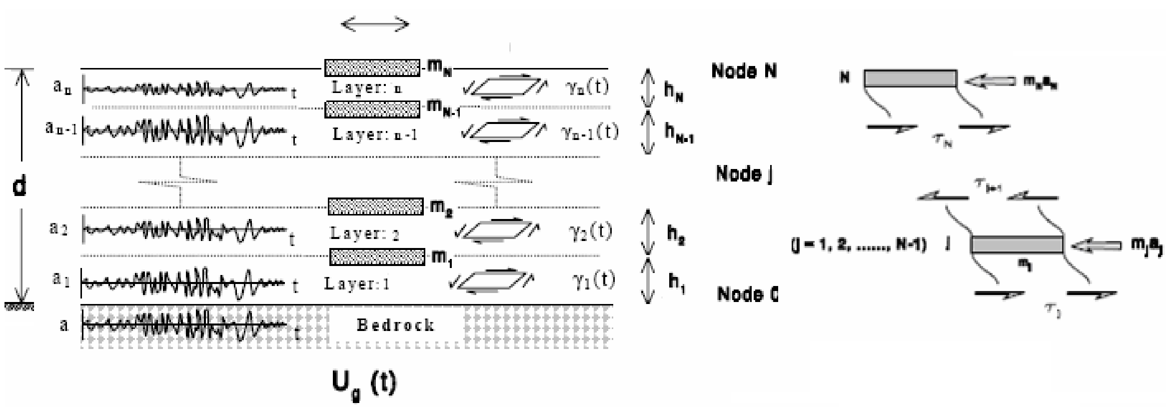

Fig.3. Free body diagram

Test Results and Analysis

A dynamic data acquisition system was utilized to record the behavior of the model during the test. In all of the conducted tests, data was recorded at a sampling rate of 1000 samples per second. Time histories of the input acceleration of the models are shown in figure 4.

These tests indicate the behavior of the soil improvement with gravel drains during the input shaking. Total unit weight and relative density of the models were 14.41 kN/m3 and %12.65 respectively. The excess pore pressure ratios were obtained from the records of the four pore pressure transducers.

In all of the tests, input motions were applied up to the start of the liquefaction and foundation settlement. The acceleration and the excess pore pressure responses of the test "**a**" are depicted in Figure 5 and 6 respectively. By comparing the acceleration records with the time series of excess pore pressure ratios, an agreement between the pore pressure spikes and the instantaneous drops in amplitude of acceleration was noticed. It implies that a very clear reduction of acceleration occurs after the second cycle in test "**a**" which means severe liquefaction and softening.

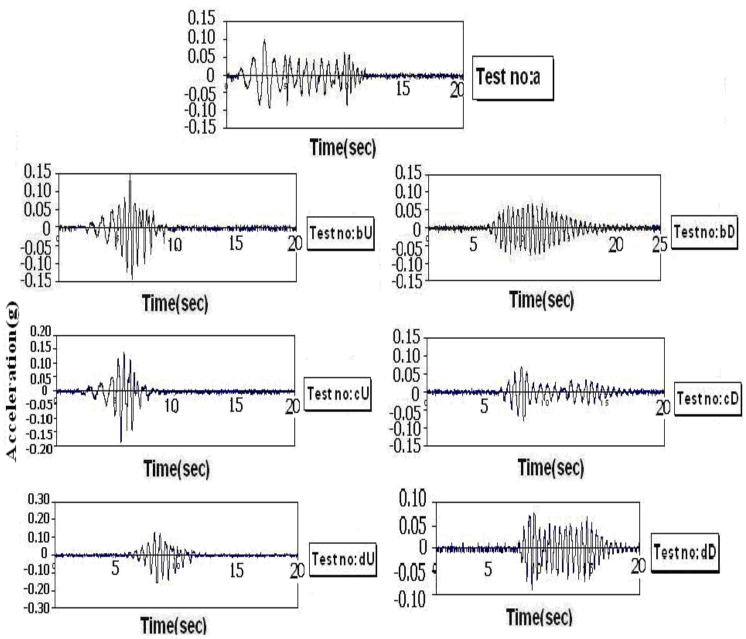

Fig.4. Time histories of input accelerations recorded in all of the 7 tests

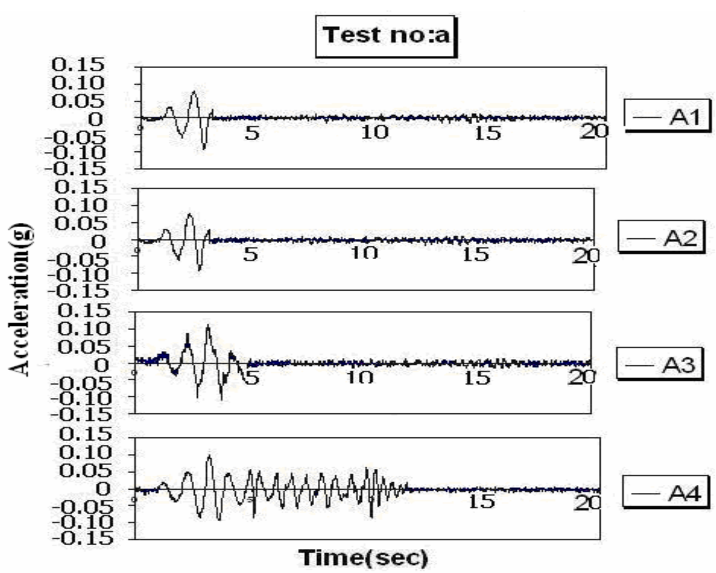

Fig.5. Time histories of accelerations recorded in Test A

In order to evaluating the soil behavior, stress–strain response is shown in Figure 7 at the location of the p1 pore pressure transducer in tests "a" and "bD". Without any soil improvement; i.e. test a, after the second cycle soil stiffness degrades rapidly, and the stress–strain curve becomes a horizontal line with infinite damping and zero stiffness. In the tests with gravel drains, soil strength is preserved to a desirable extent, which reveals their positive presence. In other words, without any improvement flow liquefaction occurs, but by using gravel drains, cyclic mobility dominates.

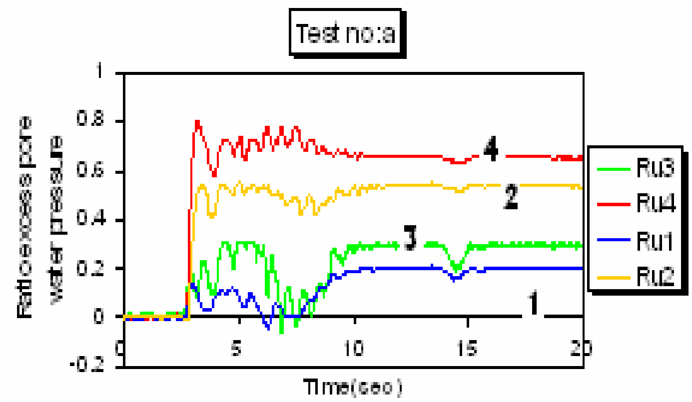

Fig.6. Time histories of excess pore pressure recorded in Test A

Liquefaction of loose, cohesionless, saturated soil deposits during earthquakes has been the subject of intensive research in geotechnical engineering. A significant amount of laboratory and field research has been focused on identifying the factors and mechanisms causing liquefaction. Soil liquefaction is a process involving structural collapse of the soil skeleton due to shear, with a concurrent loss of energy mainly by frictional mechanisms. The amount of frictional energy loss required to liquefy a soil depends on active intergrain contact density, confining stress, and frictional characteristics of the soil.

The cumulative energy loss up to liquefaction (W_L) has been identified as a useful index for liquefaction potential assessment of soils (Nemat-Nasser and Shokooh 1979; Figueroa et al. 1994, Thevanayagam et al. 2000; Trifunac 1995). Now by using the equations in previous section, accumulated energy per unit volume $(\delta W)_L$ absorbed by the specimen can be determined. This parameter is calculated in tests **a, bd** and **bU** at depths of 15cm and 35cm below the center of the foundation and the results are shown in Figure 8.

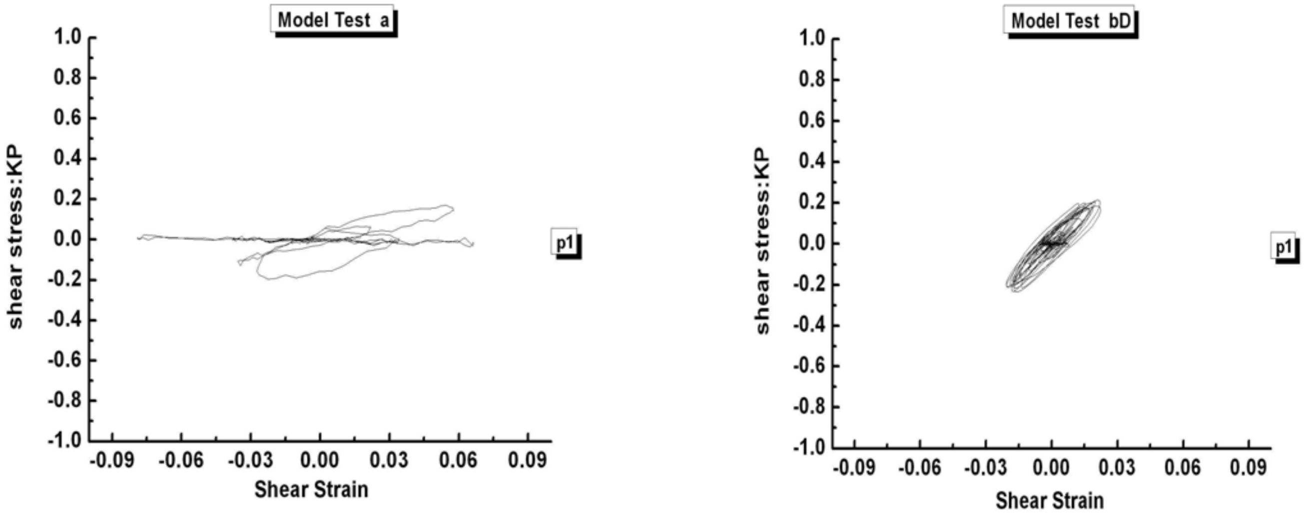

Fig.7. Stress–strain behavior of sand at location of P1 in tests a and bD

During soil deformation under dynamic loads the energy is dissipated into the soil. It is observed that the major contribution to the energy per unit volume occurs at the time of the high pore pressure build up and up to the start of the liquefaction. Law et al. (1990), Figueroa (1990), Figueroa et al. (1991, 1994 and 1998) and Liang (1995) established relationships between pore pressure development and the dissipated energy during dynamic motion that could be adopted in utilizing energy concept in the evaluation of the liquefaction potential. Propagation of seismic waves through the soil deposit induces shear strains, frictional energy loss, and gradual increase in excess pore pressures. The coupled effect of generation of earthquake-shear-strain-induced excess pore pressures and concurrent soil consolidation may lead to excessive and permanent shear strains and raise the excess pore pressures near the initial effective confining pressures. This phenomenon leads to liquefaction at various depths depending on the intensity and duration of shaking, soil density, soil compressibility, and permeability characteristics (Thevanayagam et al. 2000). It is observed that the major contribution to the energy per unit volume occurs at the time of the high pore pressure build up. This behavior has also been observed by Figueroa and Dahisaria (1991) and Ostadan et al. (1996).

After reaching the point of complete liquefaction, the specimen is not able to absorb any more energy because of the lack of shearing resistance, however a continuous increase in energy after reaching the point of complete liquefaction is observed because of the inherent residual friction in between the layers. Considering table 2 implies that pore water pressure in test **bD** has been much less than test **a**, thereupon shear strength and bearing capacity of test **bD** is higher that results in preventing excessive settlement. Then it is resulted that excess pore water pressure dissipation by using gravel drains increases the accumulated energy per unit volume (J/m^3) required for liquefaction. In Table 2 a summary of the 7 performed tests including the results of dissipated energy, is presented. Considering table 2 it is clear that the effect of gravel drains on the dissipated energy, dissipation of excess pore water pressure and shear stiffness, is dependent on the arrangement of the drains and input motion type. The other result is that the performance of gravel drains in strong earthquakes is weaker than that of weaker earthquakes. The latter statement can be understood from figure 8 too.

Table 2. Summary of the dissipated energy per unit volume for tests

Model Test NO.	Interval Time (s)	(Dr) %	Max(Ru)		Energy/Volume (J/m^3) W_L		Settlement(mm) Foundation
			z=15cm (p_1)	z=35cm (p_2)	d=15cm (p_1)	d=35cm (p_2)	
A	**4**	**12.65**	**0.21**	**0.55**	**15**	**42**	**Large**
bD	**14**	**12.65**	**0.05**	**0.08**	**38**	**109**	**3.08**
bU	**7**	**12.65**	**0.30**	**0.75**	**18**	**65**	**70**
cD	**10**	**12.65**	**0.50**	**0.62**	**18**	**36**	**44**
cU	**6**	**12.65**	**0.40**	**0.80**	**16**	**33**	**100< and <110**
dD	**10**	**12.65**	**0.42**	**0.60**	**17**	**43**	**61.7**
dU	**4.5**	**12.65**	**0.10**	**0.59**	**16**	**35**	**100< and <110**

During strong shaking, gravel drains were not able to dissipate the generated excess pore pressure considerably and their water. In all of the tests input shaking was applied up to the start of liquefaction so it can be concluded that the longer pore water pressure generation so the liquefaction and softening of the soil will be occurred later. Furthermore considering the results in figure 8 using gravel drains increases the accumulated energy per unit volume (J/m^3) required for liquefaction.

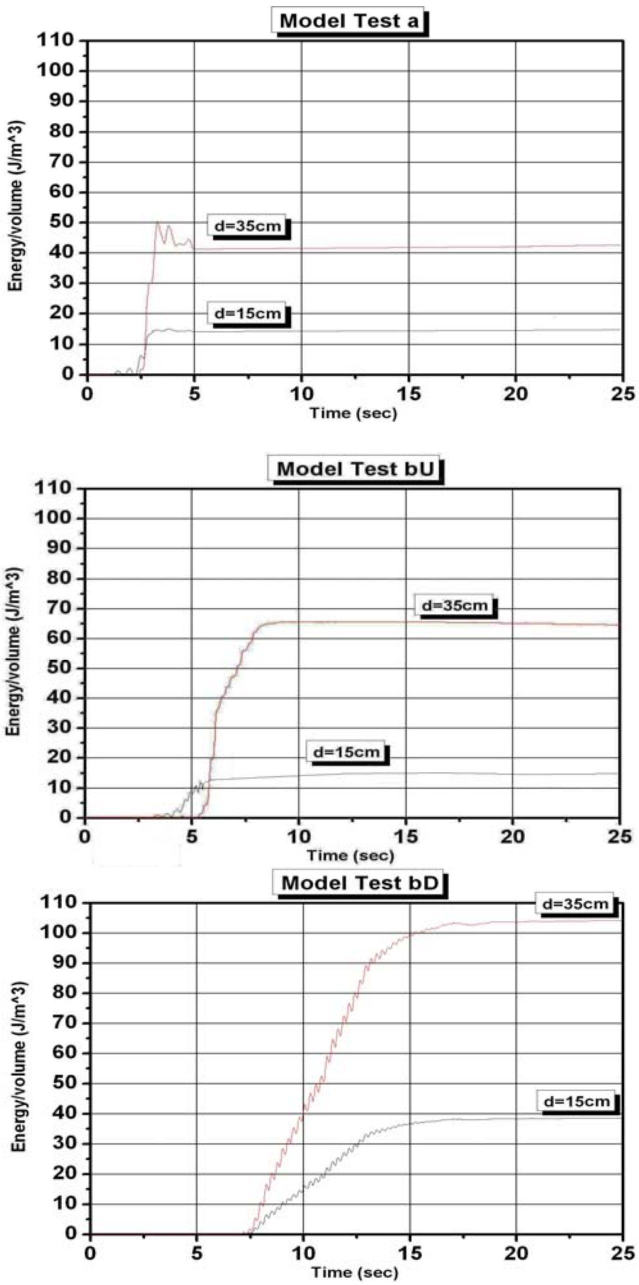

Fig.8. Time History of Accumulated Energy per unit Volume in tests "a", "bD" and, "bU"

The relation of pore water pressure generation and accumulated strain energy in saturated sand has been a topic

major effectiveness was evaluated in mitigating secondary effects of liquefaction such as upward flowing shaking periods, reveals the more suitable effect of the drains. In general the gravel drains can postpone excess

In test series **D** it took 10-14 seconds to apply shaking while in series **U** it took 4-6 seconds. Therefore in series **D** the performance of gravel drains was better than series **U** and liquefaction resistance as well as accumulated energy per unit volume required for liquefaction is much higher in series **D**. In series **U** the loss of shear stiffness of the soil is larger and quicker than that of series **D** and despite the larger shear stresses in series **U** the dissipated energy is lower as compared with series **D**. Furthermore the effect of drains arrangement is considerable. According to settlement values in table 2, triangular type arrangement (i.e. **b**) seems to be stronger than other arrangements and excess pore pressure dissipation as well as the amount of dissipated energy is considerably larger in this case.

Table 2 and figure 8 indicate that dissipated energy for all tests at depth of 35cm is larger than that of 15cm depth which is related to the presence of foundation. The foundation leads to pressure redistribution. In other words R_u values were lowest immediately below the foundation while at deeper depths the soil is more susceptible to liquefaction. This phenomenon could be completely reversed if there were no foundations placed on the soil. By considering table 2 it can be seen that the stress-strain values at depth of 35cm is larger than depth of 15cm therefore dissipated energy at deeper depth is considerably larger. The loss of shear stiffness and softening of soil in test **a** is larger and faster than that in test **bD**, so the accumulated energy per unit volume required for liquefaction in this test is lower than test **bD.** It can be concluded that utilizing the gravel drains raises the dissipated energy required for liquefaction in the soil.

of research for the last two decades and many researchers have introduced correlations between these parameters. The amount of normalized accumulated strain energy is correlated by the normalized pore water pressure. Normalized accumulated strain energy is accumulated strain energy i.e. Wh(t) divided by its maximum value i.e. Wh(max). On the other hand normalized pore water pressure is accumulated pore water pressure i.e. pwp(t) divided by its maximum value i.e. pwp(max). The correlation of these 2 normalized parameters in the form of time series is given in figure 9 in tests **a, bD, bU and cD** at depths of 35cm. When the normalized accumulated strain energy increases the normalized pore water pressure increases too and they are in good correspondence in the normal sandy models like test "a". But when drains are utilized the correlation does not exist anymore. In this case the energy consumption is continued while the pore pressure dissipates because of the drains. The comparison of these graphs can be lead to design guidelines of gravel drains.

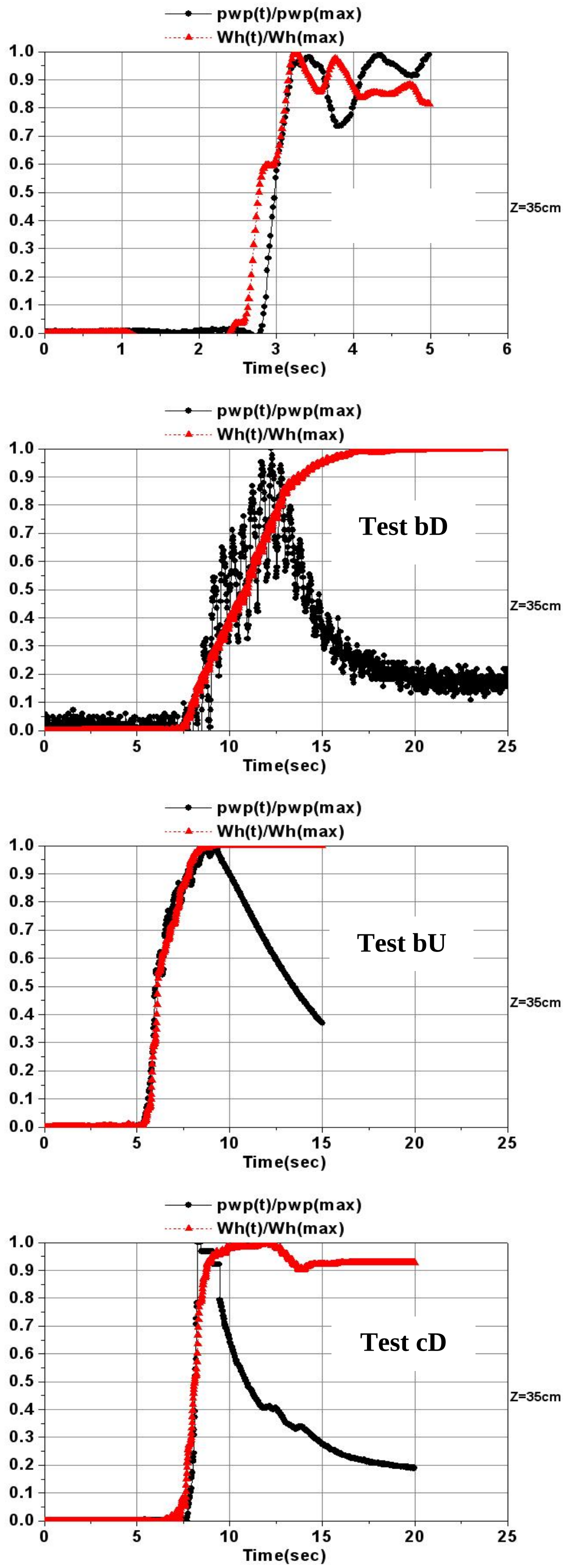

Fig.9. Comparison of Normalized Strain Energy and Normalized Pore Water Pressure

CONCLUSIONS

A series of 1g shaking table tests were carried out to evaluate the performance of gravel drains and dissipated energy method was adopted to explain the phenomenon. In order to comparing the results a test with no improvement was also performed. The followings are the important observations from the study.

(a) The energy absorption can be explained by the friction produced by the relative movement of the soil particles during loading.

(b) By using gravel drains the accumulated energy per unit volume (J/m^3) required for liquefaction is increased.

(c) The effectiveness of the gravel drains is related to the arrangement type of the drains and input motion level.

(d) Performance of gravel drains during strong earthquakes is weaker than minor earthquakes.

(e) Although the intensity of shaking was sufficient to produce complete liquefaction, the excess pore pressure ratio never reached 100% just under the centre and edge of the foundation because of the presence of initial shear.

(h) Soil improvement by means of gravel drains will transform the liquefaction mechanism from flow liquefaction to cyclic mobility.

(i) Comparison of the stress–strain behavior of the soil indicates that gravel drains postpone excess pore water pressure generation and liquefaction.

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