

# LABORATORY STUDIES ON THE EFFECT OF VERTICAL GRAVEL COLUMN DRAINS ON LIQUEFACTION POTENTIAL

A. Hamedi

Department of Civil Engineering, Shahid Bahonar University, Kerman, Iran  
alireza.hamedy@gmail.com

S. M. Marandi\*

Department of Civil Engineering, Shahid Bahonar University, Kerman, Iran  
marandi@uk.ac.ir

\*Corresponding Author

(Received: March 1, 2011 – Accepted in Revised Form: September 15, 2011)

doi: 10.5829/idosi.ije.2011.24.03b.02

**Abstract** In recent years, deadly earthquakes in seismic areas have made researchers and specialists to pay special attention to liquefaction phenomenon. Excess pore water pressure in loose sediments may cause phenomena such as boiling, shearing strength and dynamic stiffness reduction and lateral movements with associated difficulties. So far many remedial methods such as soil replacement with proper materials, in situ compaction of the soil, soil improvement using in situ grouting, and column drains have been designed and used to overcome liquefaction phenomenon. For areas with liquefaction potentials and high depths, soil replacement method is generally impossible, and vertical drain method which is both economical and easy in execution can be used as an alternative way. In this article, three series of 1-g shaking table laboratory tests were carried out, and vertical column gravel drains were modeled. In the models, various importing acceleration with variation in column gravel drains distances were studied. For modeling, Anzali shore sand situated in the north of Iran was used. Results showed that the behavior of gravel column drains varies with variation in imported seismic acceleration, and the output of this method for reduction of liquefaction potential is affected considerably. The results also showed that the conventional design method criteria to determine distances between the drain columns are very conservative.

**Keywords** Column Drains, Liquefaction, Shaking Table, Modeling

**چکیده** زلزله های مرگبار سالهای اخیر در مناطق فعال لرزه ای باعث گردیده تا پدیده روانگرایی مورد توجه خاص محققین و دست اندرکاران امر قرار گیرد. ایجاد فشار آب حفره ای اضافی در نهشته های شل در اثر نیروی زلزله، میتواند منجر به پدیده هایی همچون جوشش، از دست رفتن مقاومت برشی و سختی دینامیکی، و گسترش جانبی همراه با مشکلات ناشی از آنها شود. تاکنون راهکارهای متعددی برای مقابله با افزایش اضافه فشار آب حفره ای مطرح و بکار گرفته شده اند. برخی از این روشها شامل جایگزینی خاک نامطلوب، متراکم کردن مصالح در محل، بهسازی خاک با تزریق در محل و استفاده از چاه های زهکش بوده است. برای مناطقی که از خاکهای با پتانسیل روانگرایی و با عمق زیاد تشکیل شده است امکان جایگزینی خاک وجود ندارد. یکی از راههای مؤثر که هم از لحاظ اقتصادی مقرون به صرفه بوده و از نظر اجرایی نیز ساده است، استفاده از زهکش های قائم میباشد. در این مقاله در یک مطالعه آزمایشگاهی، با انجام سه سری آزمایشهای میز لرزه 1-g، زهکش های قائم شنی مدل سازی شده است. در این مدلسازی تاثیر شتاب های ورودی مختلف و همچنین فاصله زهکش ها مورد مطالعه دقیق قرار گرفته است. جهت مدلسازی از ماسه ساحل انزلی واقع در شمال ایران استفاده گردیده است. نتایج این تحقیق نشان داده است که زهکش های قائم تحت شتاب های ورودی مختلف، رفتارهای کاملاً متفاوتی از خود نشان میدهند که بر میزان بازدهی این روش در کاهش پتانسیل روانگرایی بسیار تاثیرگذار است. همچنین داده های آزمایشگاهی نشان داده است که معیارهای طراحی سنتی برای تعیین فاصله زهکش ها تا حدود زیادی محافظه کارانه است.

## 1. INTRODUCTION

One of the main matters of concern for geotechnical engineers is liquefaction in loose sandy soils. When sandy saturated soil is exposed to vibrations caused by earthquake, soil static equilibrium is lost because of irregular dynamic forces, and soil liquid behavior is observed due to liquefaction phenomenon. This phenomenon causes excess pore water pressure in surrounding soil and may lead to the spread of damage to structures. A change in pore water pressure follows a change in effective stress, and in a critical state in which pore water pressure equals the total stress in soil particles, soil strength is suddenly lost and liquefaction occurs.

Recent earthquakes showed that liquefaction causes serious problems to structures and leads to settlement or shear failure in soil beneath foundation [1]. The earthquake of the year 1995 Hyogoken-nanbu (kobe), 7.2 Richter scale, caused widespread liquefaction in artificial fills located along Osaka coastline. This earthquake also made the failures in relatively modern fills located on Rokko and Port islands [2]. There have also been some reports concerning liquefaction and its destructive effects: floatation of buried underground structures in loose sand deposits including sewage pipes, manholes in 1964 earthquake in Nigata [3] and 1983 earthquake in Nihonkai-Chubu [4].

Since Nigata and Alaska earthquake, 5 decades ago, there have been outstanding developments in reducing liquefaction effects. Numerous solutions have been suggested to countermeasure liquefaction and its associated damages: removing or replacing undesirable soil, compacting in situ materials, improving soil via grouting, chemical stabilization, and drain wells. Among the stated solutions, drain well is generally thought as one of the most effective methods in reducing liquefaction potential [5]. All the techniques used for reducing liquefaction potential are based on A) reducing excess pore water pressure through fast draining during and immediately after earthquake, B) improving soil skeleton flexibility to prevent extensive cyclic deformations during earthquake, C) reinforcing soil skeleton, increasing soil strength and shear strain, and decreasing generated excess pore water pressure.

One of the most widely used methods to reduce liquefaction hazards is applying gravel drain columns. Benefits can be counted as the compaction of non-cohesive soils surrounding column, excess pore water pressure dissipation, and distribution of induced stresses by earthquake or remained stresses (due to constructing stiffer piles) [1]. Murali Krishna and Madhav stated that one of the chief benefits of ground treatment with granular piles is the densification of in situ ground by which the in-situ properties of the ground are modified to mitigate liquefaction potential [6]. High internal friction angle of gravel materials gives important frictional components to consequential composite materials which can increase the strength and flexibility of the materials. They also declared that the very high deformation modulus and stiffness of the granular pile material provide reinforcement for the in situ soil and offer another mechanism to mitigate liquefaction.

Investigations to improve ground by gravel columns replacing a part of in situ soil began by the end of decade 60. Applying gravel columns as a solution to stabilize soils with liquefaction potential was first considered by Seed and Booker's works [7]. They stated that the generated pore water pressure due to cyclic loading dissipates as fast as generation using the gravel drains system. The gravel drains system was considered by researchers thereafter: Ishihara and Yamazaki, 1980 [8], Tokimatsu and Yoshima, 1980 [9], Baez and Martin, 1995 [10], and Boulanger et al, 1998 [11]. Most of these follow up works highlighted the effectiveness of permeability of drains as the main lack of Seed and Booker's charts. They pointed that Seed and Booker's first guess stating that a drain is infinitely permeable is overly simplistic. Gravel columns technique won the Technical Development award of Civil Engineering Society of Japan [12]. Gravel or crushed stones, or mixtures of them are generally used as fill materials in the drains. In Japan, where gravel columns have been employed in many cases, the most commonly used material was crushed stones between sieves no. 5-7 [13]. The effects of the installation method of stone columns, e.g. cased and uncased wells, number of lifts and magnitude of energy per lift to compact gravel

columns, and drains spacing, have been discussed by Madhav and Thiruselvam [14].

The installation of gravel columns keeps the excess pore water pressure ratio values ( $r_u$ ) low by decreasing the tendency to generate excess pore water pressure and increasing the rate of pore pressure dissipation:  $r_u = u_e / \sigma'_v$  where  $u_e$  and  $\sigma'_v$  are excess pore water pressure and vertical effective stress, respectively.

The major benefits of keeping  $r_u$  low: A) Remaining of most of soil strength which lets soil retain its role as a vertical support for the top structure. Maintaining strength excessively reduces ground lateral deformation caused by dynamic excitation. Ground slope may seriously increase this deformation even if it is negligible: lateral spreading of 3m has been observed in ground with 3% slopes [15]. B) Large and/or unbalanced settlements which often occur by reaching  $r_u$  to 0.5-0.6 are prevented [16]. The reason is that high volume compressibility of liquefied soil is seen under low confined stress at high values of  $r_u$ . Scott, Adalier and Elgamal essentially attribute this to soil sedimentation components as well as the above factor [17, 18]. Relative variations are often small at low  $r_u$  values. C) High hydraulic gradient which may lead to the penetrating of fine materials into gravel drains and destructing drainage ability.

In order to evaluate gravel drains impact on liquefaction process, many shaking table tests have been performed by different researchers. Tokimatsu and Yoshimi performed modeling tests to investigate the effects of gravel drains. Their results showed that the drainage effects of gravel drains are means to stabilize sandy soils with liquefaction potential under structures [19]. Through their large scale laboratory studies Sasaki and Taniguchi noticed the development of excess pore water pressure, during the shake and near the drains, mitigated and the rate of excess pore pressure dissipation after shaking could be accelerated by drains [20]. They also realized uplift induced forces caused by liquefaction under structure were lowered by gravel drains installation. The effects of gravel drains system on buried pipes during an earthquake were considered by Miyajima et al. [21]. They used shaking table

tests to model gravel walls and rows of gravel piles parallel to the pipes axis. They concluded that drains system reduces the maximum excess pore water pressure and duration of liquefaction, but its effectiveness was sensitive to the piles spacing. Orense et al. employed recycled concrete crushed stones to construct wall-type gravel drains in their follow up studies [13]. They installed these kinds of drains around underground structures to investigate the effectiveness of them and concluded that choosing an appropriate grain size for drains will effectively reduce excess pore water pressure under the structures and consequently reduce uplift forces. After that, they stated there is a critical width of gravel drain for any specified structure, ground and earthquake condition in which no uplift occurs, using finite element analysis. In another research, Brennan and Madabhushi used centrifuge tests to realize vertical gravel drains behavior: "the pore water from a radially expanding zone of soil contributing to drainage through the drains is developed [22]." Sadre Karimi and Ghalandarzadeh performed two different methods to mitigate liquefaction [1]. They conducted shaking table tests on model gravel column drains and compacted sand piles with pre-determined dimensions and spacing and applied the same input acceleration to all the tests. They declared that compacted sand piles are more effective with respect to liquefaction resistance and soil settlement under structure during shake period but, after the end of shaking, the effectiveness of drains will increase with acceleration in pore water pressure dissipation.

Although numerous studies about liquefaction remediation have been performed by different researchers, due to its large damages, it is thought that more laboratory information is needed to countermeasure this destructive phenomenon. The effects of vertical gravel drains on reducing generation of excess pore water pressure because of three different dynamic input motions are considered using shaking table tests.

## 2. SHAKING TABLE MODELING TESTS

Shaking table tests on small scale models are one of the most important study methods in geotechnical earthquake engineering. Many tests

have been conducted on scaled models to investigate structures which are related to soil. Constructing small models and testing them is simple in comparison with the real models. The small ones are repeatable easily, and controlling and changing test conditions for a specified case are possible. Scaling is an interesting problem in geotechnical engineering. Although there are some valuable researches e.g. Rocha [23], Roscoe and Poorooshasb [24], Kagawa [25], and Iai [26], some problems are still unsolved. Due to small confined stresses, the scaling is more complicated in normal conditions (i.e. normal gravity or 1-g). As the soil modulus of elasticity and shear modulus depends on confined stresses, the generated dynamic stresses and settlement of structures may be affected in 1-g shaking table tests. Another concern in using shaking tables is to avoid the side effects of the test container. This can be remedied by placing sensors at a reasonable distance from the sides of the container. However, these limitations do not underestimate the value of small scale tests, and they are still important sources of information in geotechnical engineering.

In this research accurate studies have been performed under cyclic loading using 1-g shaking table on sandy materials especially Anzali shore sand located in the north part of Iran. By changing various parameters like soil compaction, frequency and dynamic loading acceleration, and drains spacing, the effects of them on excess pore water pressure fluctuations, which are important criteria to study liquefaction, have been investigated. Since the gravel column drains were installed during the specimen preparation process, no accompanying densification occurred during installation. Therefore, the effect of drainage alone was evaluated. There are strong similarities between these studies and those with large scale shaking tables, and it is possible to observe grain materials behavior under cyclic loading clearly.

### 3. LABORATORY PROGRAMS

**3.1. Material Properties** Evaluation of several regions of Anzali port soil and drilled bore holes showed that the majority of soil is poorly graded sands or gravelly sands with no fines (SP), and also in a few parts of depths, poorly graded sand with some fine grained silt (SP-SM). The soil materials

used in the present study were (SP) provided by Iran north shore and were sieved according to ASTM D422-63 [27] standard. Grain-size distribution curve of used materials is shown in Fig. 1. The uniformity coefficient and the coefficient of gradation are 2.35 and 1.13 respectively according to ASTM D2487-10 [28]. The specific gravity of soil solids was measured 2.67 according to ASTM D854-02 [29]. Relative density was defined in terms of the loosest and the most compacted state according to ASTM D 4253-00 [30] and ASTM D 4254-00 [31]. Maximum and minimum values of void ratio were derived 0.76 and 0.49 respectively according to the above mentioned standards. Table 1 shows the employed material properties.

Relative density ( $D_R$ ) is a parameter to investigate relative compaction when fine grains are less than 15% and is defined as equation (1). Calculating relative compaction by laboratory tests is not possible because of the impossibility to obtain undisturbed soil specimens in sands and grain soils in any depths. Thus researchers made a relation between in situ tests and relative compaction [32]. Terzaghi and Peck [33] established equation (2) between standard penetration and relative density ( $D_R$ ) in sandy soils, and it has been subsequently reconsidered and verified by Skempton [34].

$$D_R = \frac{e_{\max} - e_0}{e_{\max} - e_{\min}} \quad (1)$$

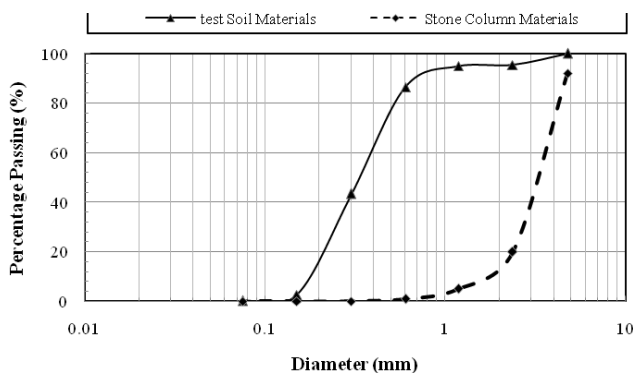
$$D_R = 100 \cdot \sqrt{\frac{(N_1)_{60}}{60}} \quad (2)$$

Where,  $e_{\max}$ ,  $e_{\min}$ ,  $e_0$ , and  $(N_1)_{60}$  are maximum possible void ratio, minimum possible void ratio, void ratio in natural state of soil, and SPT corrected values, respectively.

To evaluate the field relative compaction of used sand, SPT results of several bore logs, from Anzali up to 16m depth, were taken into consideration. Average approximation of the parameter was calculated by normalizing  $N$  and  $(N_1)_{60}$  values of the mentioned tests (Fig. 2). Using these results and equation (2) made it possible to compute average relative density of Anzali sand in different depths (Fig. 3). Fig. 3 shows that the average relative compaction did not change considerably by depth and was about 50%.

**TABLE 1.** Employed material properties

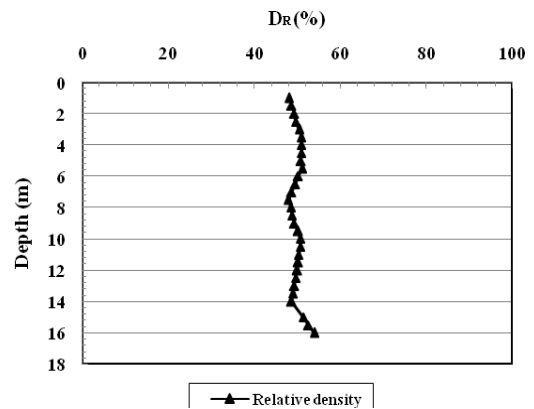
The test soil properties	
Specific gravity, $G_s$	2.67
Coefficient of curvature ( $C_c$ )	1.13
Uniformity coefficient ( $C_u$ )	2.35
Grain size of 50 percent passing, $D_{50}$ (mm)	0.33
Minimum void ratio, $e_{min}$	0.49
Maximum void ratio, $e_{max}$	0.76
Permeability, $k$ (cm/s)	$1.29 \times 10^{-2}$
$m_v$ ( $m^2 / kN$ )	$5 \times 10^{-5}$



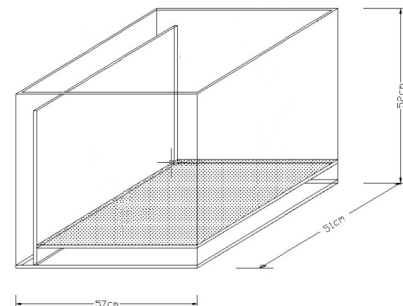
**Figure 1.** Grain size distribution of used materials and vertical drains

### 3.2. Laboratory Equipments

**3.2.1. Transparent tank** Transparent tank is an uncovered Plexiglas cube with dimensions of 57cm × 51cm × 52cm and 1cm thickness. Plexiglas planes are connected and sealed together by pins and special glue (Fig. 4).



**Figure 3.** Average relative density ( $D_R$ ) by depth in Anzali region



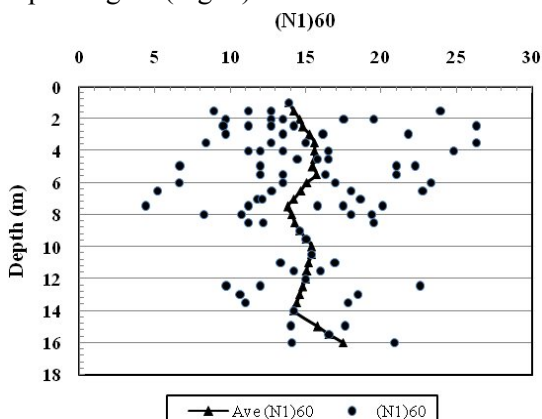
**Figure 4.** Three dimensional view of the container

**3.2.2. Shaking table** Shaking table used in the laboratory included: A) Electromotor and a gearbox with power of 2.2kW and a 30mm shaft with 200 rpm constant speed. B) SIMENS G 110 inverter with the capacity of 2.2kW which may change electromotor speed from 0 to 1 time of its normal speed or 0 to 200rpm (this change was applied by a volume set to the machine).

### 3.3. Model Preparation

**Sand pluviation device:** To ensure making uniform, homogeneous specimens and also to perform repeatable tests with a specified compaction, sand pluviation machine was designed and set on the main machine. Adjusting different pluviation heights was also possible. This technique was used by Mir Mohammad Hosseini and MoghaddasTafreshi [35] to prepare uniform and repeatable soil specimens with a specified compaction. This machine consists of the following parts:

**Main frame:** This frame was made in order to



**Figure 2.** Average  $(N_1)^{60}$  by depth in Anzali region

keep pluviation container over the tank and is connected to the main frame of shaking table. There are some brackets at different heights on the frame to keep the container to adjust sand pluviation heights.

**Peripheral curtains:** The perimeter of the container was surrounded by plastic curtain to stop scattering of sands during pluviation.

**Pluviation container:** Required sand was poured in pluviation container with dimensions of 50cm × 50 cm × 30cm in a few steps. There are two grooves under the container to set the sieve and drawer plate to pour sand from the container to the tank. The drawer plate is closed at the beginning. After filling the container, sand starts to sprinkle by pulling the drawer plate out. Obtained compaction of soil depends on pluviation height, pluviation rate and grain distribution of used sand. An increase in height leads to more compaction of the obtained soil profile while an increase in pluviation rate leads to less compaction. The rate can change by replacing different sieves under the container, e.g. using finer sieves will decrease the rate. Fig. 5 shows the pluviation container from the front and top view.

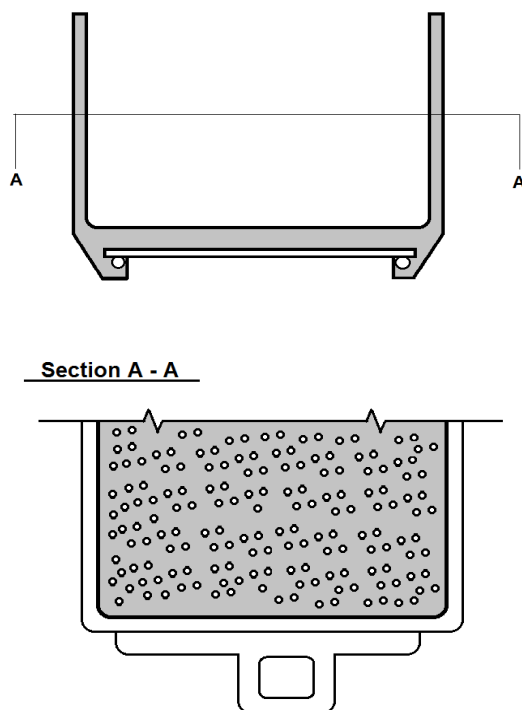


Figure 5. Sand pluviation device

**Tank displacement sensor:** It is possible to calculate input acceleration considering displacements of the tank during cyclic motion. A digital displacement gauge (LVDT) was used in order to record accurate and continuous displacements during motion with amplitude of 37mm and accuracy of 0.01mm which was able to produce outputs under fast cyclic changes. The sensor was fixed on the main frame of the shaking table, which was stable during cyclic motion, by a sheath vertically on the tank wall. The location of the sensor can be adjusted horizontally by three bolts. The central core of the sensor moved with the machine during motion. A -4.11 to 3.12 voltages was sent to data logger according to amplitudes of 0.0 to 37mm respectively based on the location of the tank and quantity of central core compression.

**Pore pressure transducer sensors:** Variation of pore pressure due to cyclic loading was very fast during the tests. Three pressure transducers were employed to record accurate and continuous pore water pressure with the capacity of 0.0 to 0.1bar (about 0 to 100cm of water). These transducers were set at 7.5, 17.5, and 27.5cm from filter on the tank floor. To minimize errors due to boundary conditions, transducers recorded pore pressures in the middle of the specimen, which was located in the farthest place from tank walls. They were of model PSCH00.1BCIA from SENSYS which produced an electric current from 4 to 20mA according to input pressure. This output current resulted from the pressure equal to 0 to 1bar based on the calibration defined by the manufacturer with a 0.041 percent of full scale nonlinearity error. Fig. 6 shows the positions of transducers and the cross section of the model.

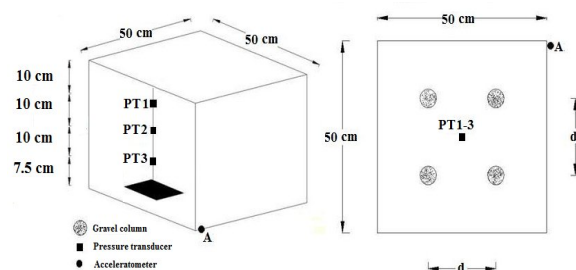


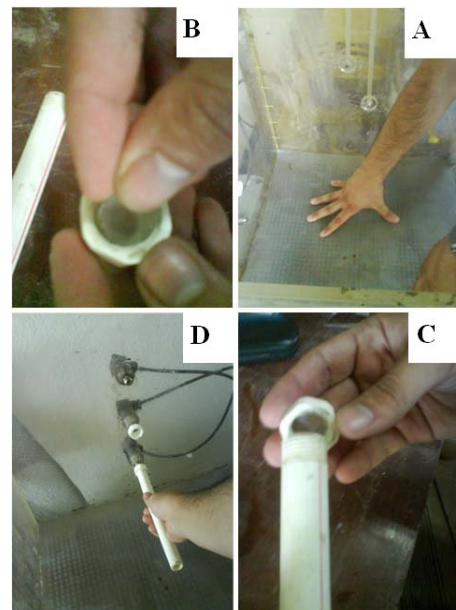
Figure 6. Sensors position and cross section of the model

**Data logger:** This card includes four parts: electricity supply board, sensors driver board, analog to digital convertor board, and a transformer converting 220V alternating current to 5 and 12 direct current. The main part was analog to digital convertor board which read analog data from sensors and converted them to digital so that they could be received by a data record system like a computer. To make this part of data logger, a convertor board of model ADC16-10016 produced in TNM Company was used. The board was 12 bit with a sampling rate of 100 kHz which could receive data from 16 sockets. When all the sockets were in use, the sampling rate was 6.25 kHz. To run the data logger, the pressure and displacement sensors, which sent an electrical current in every mentioned time gaps, should be connected to it. Then it converted analog received data to digital ones in its internal system, and whether it was in one or bipolar state, it transferred the received data value to computer through USB.

**Software Used to record data:** Required software was coded in VC++ software in order to record data in computer by recalling DLL files. The software, called TEXT RECORD, recorded input data as a text file for every input channel separately so that every single number was in a row. So the recorded numbers were simply transferable to other soft wares, e.g. Excel and MATLAB to be analyzed and to draw related curves.

To prepare specimens, first the sieve number 400 was set at the bottom of the tank container to avoid fine grains from being washed and removed. Then filters were set on pipes which transmitted water from middle of the soil profile to the tip of pressure transducers. Pluviation container was placed on the desired height after equipping tank accessories, and the drawer plate was fixed in a closed state. Peripheral curtains were hanged from the perimeter of pluviation container and were stuck to the interior walls of the container of the tank. Dry sand was poured into the upper container to reach the desired elevation (about 25cm) and sand was poured into the container of the tank by pulling drawer plate out. The pluviation container was placed in a higher step (25cm) and the process was repeated again so that pluviation height was equal to the former step. This process was

continued until the container became full. Preparation steps are shown in Figs. 7 and 8.



**Figure 7.** Container preparation steps: A) Installing sieve No. 400 on the bottom of container, B) Installing a filter on transferring pipes, C) pipes preparation, D) Installing water pressure transferring pipes

**3.3.1. Installing gravel columns** In order to install gravel columns, socks which are used as a Geotextile filters were placed in pipes, and gravel materials with the mentioned distribution were poured in the socks to fill them. Based on Orense et al. [13], who conducted laboratory studies on the changing of the permeability of drains with compaction energy and grain size distribution, no compaction energy was applied to the gravel materials in order to have the maximum permeability. A small quantity of the sand was poured in the bottom container to make about 3cm of soil profile. Pipes then were placed in the soil with predetermined distances, and soil profile was completed using pluviation technique. Finally pipes were pulled out of the soil profile gently and gravel columns remained there (Fig. 9).

**3.3.2. Soil saturating** Saturation of soil was performed using pluviation technique. The container was filled with about 10cm of water and the sand was pluviated in water so that the specimen was saturated from the beginning.



Figure 8. Installing tank accessories



Figure 9. Drains set up in soil profile

**3.3.3. Preparation of reading and recording instruments** After connecting sensors to data logger and running Text Recorder software, the model was ready to perform tests. Input shaking in all tests was a harmonic wave. The frequency of input shaking was adjusted by the inverter which had been connected to the electromotor. The inverter changed the frequency of input electric current to electromotor which changed the speed of motor rotation. The frequency of electric current was shown on a monitor that was variable from 0 to 50. It means for maximum frequency of electric current, the speed of rotation was equal to its maximum value (200rpm) which gave a 3.33 Hz of input shaking frequency. Varying the number on the monitor from 0 to 50 made a linear change in input shaking frequency from 0 to 3.3 Hz. After adjusting the desired frequency and clicking on the "Record All in Text" button in the software,

recording began and the tank started shaking after 5 seconds of recording data by pressing the button on the inverter. Shaking was stopped after 60 seconds, but recording data continued for 120 seconds and was stopped by clicking again on "Record All in Text" button. Fig 10 is the summary of laboratory process.

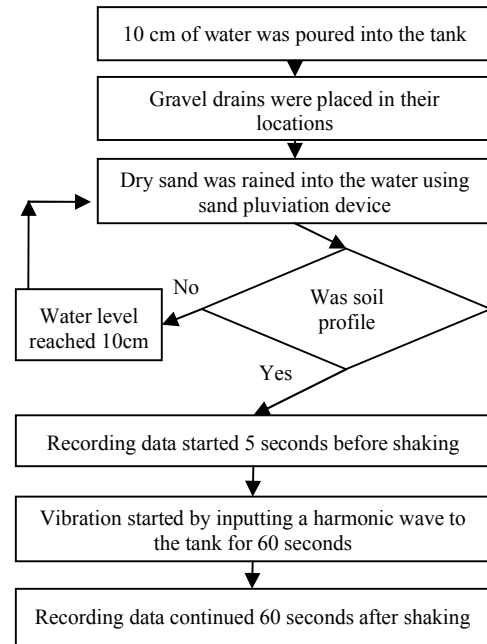


Figure 10. Model preparation process

#### 4. DESIGNING GRAVEL DRAINS

Seed and Booker charts were used in order to design gravel drains as a liquefaction countermeasure in non-cohesive soils [7].  $T_{ad}$ , a parameter relating the duration of the earthquake to the consolidation properties of the sand, and  $R_u$  values should be determined at first, and by assuming a value for drains diameter, center to center spacing for drains could be calculated. For example, in our study, considering  $T_{ad} = 112.2$  (Geometrical scale set to 1/30),  $R_u = 0.2$  and  $N/N_L = 4$ ,  $a/b$  would be 0.23 using Seed and Booker charts.  $N/N_L$  is a ratio characterizing the severity of earthquake shaking in relation to liquefaction characteristics of the sand, and  $a/b$  is a ratio indicating the geometric configuration of the sand drains. In this study for  $a=2.5cm$ ,  $b=2.5/0.23=11cm$ . By placing  $a$  and  $b$ , center to center spacing and diameter of drains were obtained as Table 2:



**TABLE 2.** Performing relations for gravel drains

Drains diameter	Center to center spacing between gravel columns	Performing model
$2a$	$d=1.77 b$	Square installation
5cm	$d = 1.77 \times 11 = 19.5\text{cm}$	Square installation

Considering the above calculations, the diameter of drains was set 5 cm, and the maximum theoretic center to center spacing was 20cm which was compared to the values from laboratory results.

## 5. SHAKING TABLE TESTS RESULTS

**5.1. Drainage Effects** In order to investigate drainage effects of gravel columns on the variation of pore water pressure, a series of 10 shaking table tests was conducted in 3 groups. Input motion type was different in these groups and the initial soil compaction was the same in all the tests. An unreinforced test using no countermeasure (by gravel drains) was performed in each group to be compared with tests with countermeasures. Table 3 shows tests program in this test series. The relative densities calculated from equation (1) in terms of the maximum and minimum possible void ratio ( $e_{max}$ ,  $e_{min}$ ) are presented in Table 1. Void ratio of the model soil ( $e$ ) was calculated based on the dry weight of the employed soil and the volume of saturated soil in the test container. Tests results show that input motion conditions considerably

influenced the effectiveness of drains to remediate excess pore water pressures.

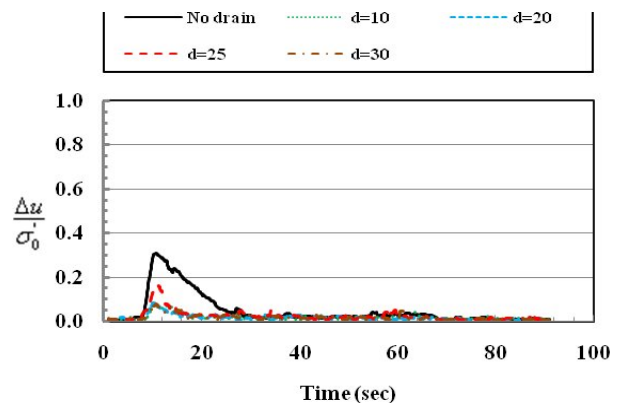
$(r_u)_{max}$  values for group A tests and each pressure transducer are presented in Table 4. Figures 11(a), 11(b), and 11(c) show the variation of the parameter with time for this group and for sensors 1 to 3. These results indicate that the drains effectively dissipated excess pore water pressure in group A where maximum input motion acceleration ( $a_{max}$ ) was lower than the other groups during dynamic shaking. A surprising point in this group was observed in test no. 4 in which the average of maximum cyclic input accelerations was about 0.01g more than the other tests in the group. But this slight difference increased the generated  $r_u$ , even more than that of test no. 5, where the drains spacing was 5cm more. This means that the behavior of drains was deeply affected by input acceleration. In other words the effectiveness of drains was under the influence of maximum input acceleration rather than the drains spacing. In the other tests of the group, except for location of pressure transducer No. 1, excess pore water pressure ratio, which was also considered in our calculations, was limited to the desired value (less than 0.2). This can indicate that the fluid from deeper strata is drained first, reducing the effectiveness of drains for near-surface layers. In other words, deeper fluid uses the full capacity of drain, and overlying deposits must wait for a way to be discharged.

**TABLE 3.** Tests program in groups A, B, and C

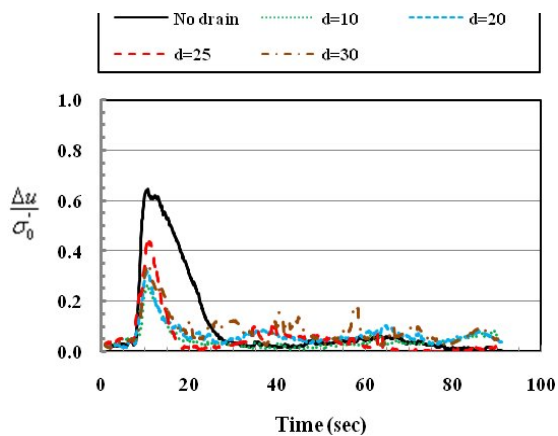
Group	Test No.	Relative density (%)	$a_{max(ave)}$	surcharge	$a_{max}$	Drains spacing (cm)
A	1	50	0.07g	none	0.07g	none
	2	50	0.07g	none	0.07g	10
	3	50	0.07g	none	0.07g	20
	4	50	0.08g	none	0.08g	25
	5	50	0.07g	none	0.07g	30
B	6	50	0.25g	none	0.27g	none
	7	50	0.23g	none	0.26g	20
C	8	50	0.12g	none	0.13g	none
	9	50	0.14g	none	0.15g	20
	10	50	0.14g	none	0.16g	10

**TABLE 4.** Maximum  $r_u$  values in group A for three pressure transducers

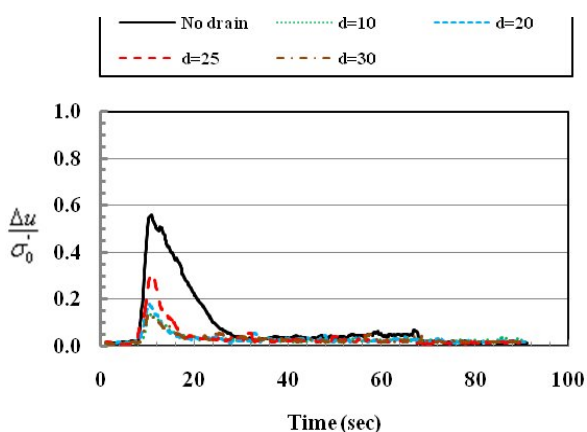
Drains relative spacing (cm)	Without gravel column	10	20	25	30
Test No.	1	2	3	4	5
Average of maximum input accelerations	0.07g	0.07g	0.07g	0.08g	0.07g
$r_u$ ( $PT_1$ )	0.65	0.28	0.32	0.46	0.37
$r_u$ ( $PT_2$ )	0.56	0.12	0.17	0.30	0.16
$r_u$ ( $PT_3$ )	0.31	0.07	0.07	0.16	0.06



**Figure 11(c).** Time histories of pore pressure ratio for group A (Pressure transducer No. 3)



**Figure 11(a).** Time histories of pore pressure ratio for group A (Pressure transducer No. 1)

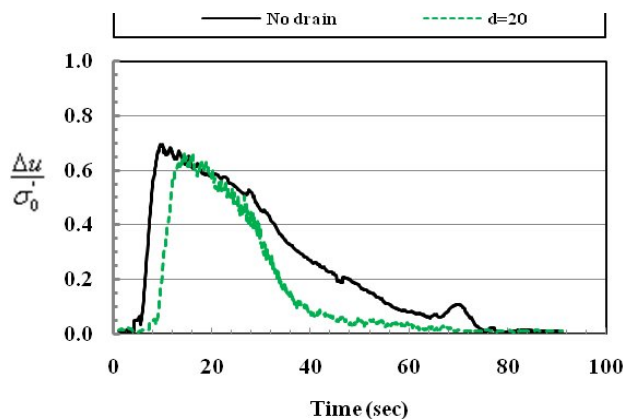


**Figure 11(b).** Time histories of pore pressure ratio for group A (Pressure transducer No. 2)

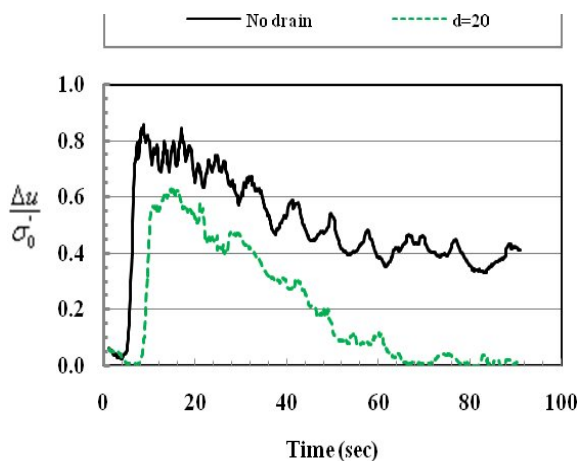
Table 5 shows  $(r_u)_{max}$  values for group B and time histories of  $r_u$  are shown in Figures 12(a), 12(b), and 12(c) for sensors 1 to 3. In this group, drains were unable to limit  $(r_u)_{max}$  to the desired amount. Except for sensor No. 1, the other sensors indicated that  $(r_u)_{max}$  value was equal to the case without drains, but this did not mean the ineffectiveness of drains. For the model test without drains (test No. 6), pressure transducer No. 1 recorded excess pore water pressure long time after strong shaking. Although (in test No. 6) in deeper strata ( $PT_2$  and  $PT_3$ ) the pore water pressure started to dissipate after cessation of shaking, it did not dissipate in shallower deposits ( $PT_1$ ) owing to the upward movement of water from the lower deposits. Such a trend was also observed during the 1995 Kobe earthquake where upward seepage was evident in Rokko and Port Island about an hour after the main event [36]. This migration of water may reduce the strength of surface soil or may generate 'secondary' (or seepage-induced) liquefaction causing large deformations or loss of bearing capacity [37]. Gravel drains reduced this excess pore pressure development obviously. So in the test with gravel drains (test No. 7), the excess pore pressure after shaking was completely eliminated. Furthermore, the excess pore pressure was dissipated with a higher rate after reaching the maximum value due to the presence of drains. Another important point was reaching of  $(r_u)$  to its maximum value which occurred later in test No. 7 in comparison with test No. 6, and drains delayed liquefaction process.

**TABLE 5.** Maximum  $r_u$  values in group B for three pressure transducers

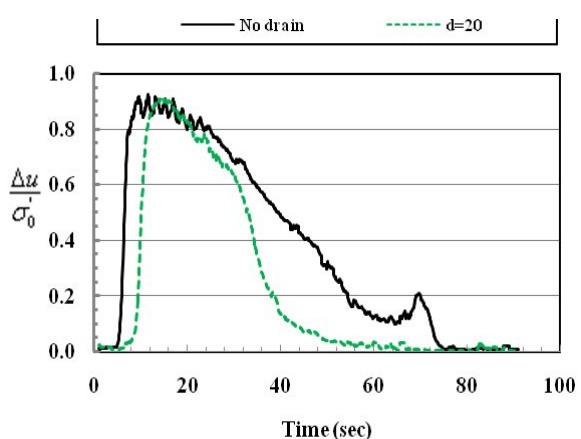
Drains relative spacing (cm)	Without gravel column	20
Test No.	6	7
Average of maximum input accelerations	0.25g	0.23g
$r_u (PT_1)$	0.85	0.62
$r_u (PT_2)$	0.93	0.91
$r_u (PT_3)$	0.70	0.66



**Figure 12(c).** Time histories of pore pressure ratio for group B (Pressure transducer No. 3)



**Figure 12(a).** Time histories of pore pressure ratio for group B (Pressure transducer No. 1)



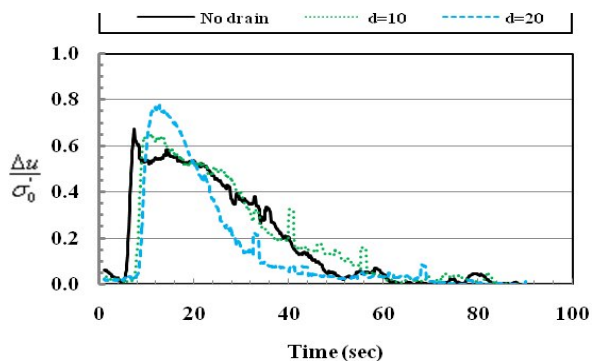
**Figure 12(b).** Time histories of pore pressure ratio for group B (Pressure transducer No. 2)

After group B model tests, group C tests were performed with an input motion of type 2. This type is between groups A and B, considering the magnitude of input acceleration. Figures 13(a), 13(b), 13(c), and Table 6 show time histories of pore water pressure and obtained  $(r_u)_{max}$  values in this group, respectively. Drains behavior was considerable here. Pressure transducer No.1 almost registered equal values of  $(r_u)_{max}$  in the reinforced and unreinforced model tests. It could indicate that, in this depth, drainage was completely vertical thorough soil surface rather than drains. Moving down in the soil depth, the effectiveness of drains on reducing  $(r_u)_{max}$  values increased. Noticing the results of test 9, the drains mitigated  $(r_u)_{max}$  values in comparison with the test 10, yet the values were still high and more than the designed expected values. On the other hand, the results of test No. 10 indicated a behavior against the expected one. Although the drains spacing were less than test 9, transducers No. 2 and 3 showed a higher  $(r_u)_{max}$  values. This problem was the result of the limitation to produce a completely uniform input motion by the used shaking table as seen in Table 3. In other words, the maximum input accelerations values were a little different for any tests in the group. In this group, similar improvements by drains with group B were also clear. Pore pressure showed delayed response compared to unimproved test No. 8 as well as fast dissipation after reaching the maximum value. Another important point in the group was the similarity in pore pressure values between tests

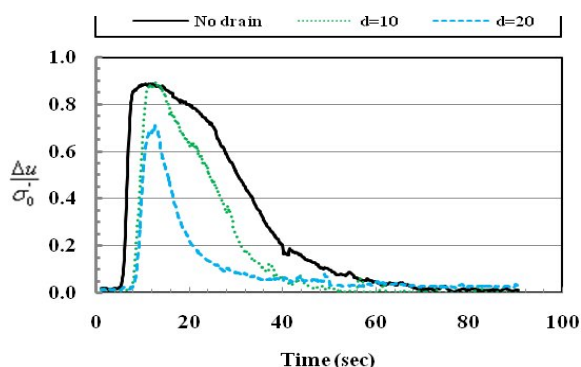
with and without drains after shaking. The excess pore water pressure had been dissipated before the end of shaking in this group. Excess pore water pressure could not be stable in the unreinforced test during dynamic motion due to vertical drainage under this type of motion. In fact, it was not confronted with uplift problem at the end of shaking.

**TABLE 6.** Maximum  $r_u$  values in group C for three pressure transducers

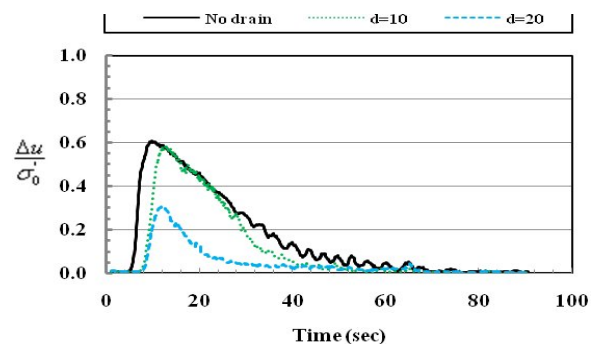
Drains relative spacing (cm)	Without gravel column	10	20
Test No.	8	10	9
Average of maximum input accelerations	0.12g	0.14g	0.14g
$r_u (PT_1)$	0.68	0.65	0.77
$r_u (PT_2)$	0.89	0.89	0.71
$r_u (PT_3)$	0.59	0.58	0.30



**Figure 13(a).** Time histories of pore pressure ratio for group C (Pressure transducer No. 1)



**Figure 13(b).** Time histories of pore pressure ratio for group C (Pressure transducer No. 2)



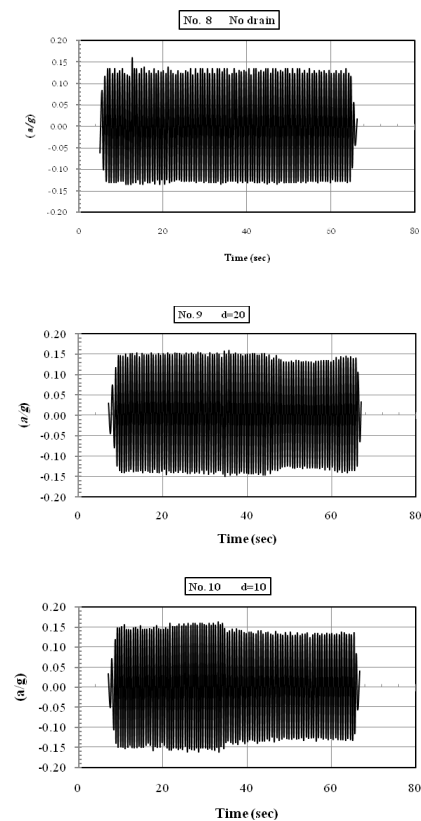
**Figure 13(c).** Time histories of pore pressure ratio for group C (Pressure transducer No. 3)

In addition, in all tests of the 3 mentioned groups, pore pressure No. 3 showed less ( $r_u$ ) values compared to No. 2, which was located in the shallower depth. It refers to an increase in confined stress by depth. Pore pressure No.1 also followed the mentioned rule for the tests of group A where input motion was of type 1. However for the other two groups (groups B and C), pressure No. 1 apparently showed a contradictory behavior. The mentioned ratios derived from pressure transducer No.1 were less than those of No. 2, located in the deeper depth. The explanation is because of the small scale of the model soil and vertical drainage from the soil surface. Also, situating the transducer ( $PT_1$ ) near the surface (depth of 10cm from the soil surface), the generated excess pore pressure dissipated during dynamic shaking due to the short path of drainage, and pore pressure did not reach the potentially expected value. So, the pore pressure was drained vertically before reaching the maximum expected amount. However, in group A, due to lower input acceleration, ( $r_u$ )<sub>max</sub> generated values were also low so that the pore pressure did not dissipate before reaching the expected values.

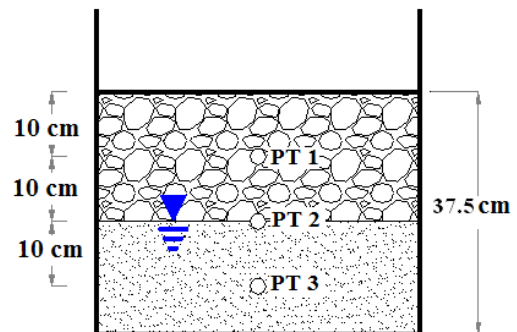
For input motions of type 3 (Group B), there was not radial drainage at the beginning of the tests. However, after reaching of  $r_u$  to the maximum value, radial drainage started, and drains increased the rate of excess pore water dissipation. In test No. 6 (without drains),  $PT_1$  recorded more value for ( $r_u$ )<sub>max</sub> in comparison with test No. 7 (with drains). This was due to 0.02g excess input acceleration. To some extent, this further ( $r_u$ )<sub>max</sub> can be also observed for other transducers ( $PT_2$  and  $PT_3$ ). For input motion of type 2 (group C), similar radial drainage was recorded for near surface strata

(where  $PT_1$  was located). At the beginning of the tests, vertical drainage was observed from the soil surface. Fig.14 shows time histories of input accelerations for group C. Although  $a_{max(ave.)}$  (average of maximum input accelerations) was the same for the tests No. 10 and 9, different  $a_{max}$  (maximum input accelerations) were applied to the tank during the tests. Input  $a_{max}$  for the test No. 10 was further at the beginning of the test and became less at the middle. This fluctuation caused following outcomes: for  $PT_1$ , the rate of dissipation of  $r_u$  in test No. 10 was almost equal to that of test No. 8 and, the rate of test No. 9 was the highest (due to less input acceleration in comparison with test No. 10). By moving down in the soil depth, the effectiveness of drains increased. For  $PT_2$  and  $PT_3$ , the rate of dissipation of  $r_u$  increased in test No. 10 and became faster than that of test No. 8.  $(r_u)_{max}$  value did not reduce due to the higher input acceleration. However, for the test No. 9 both the rate of dissipation and  $(r_u)_{max}$  value decreased considerably.

**5.2. Surcharge and Compaction Effects** To investigate the effects of the presence of a top layer soil as a surcharge with no liquefaction potential placed on liquefiable soil and also soil compaction, as soil improvement methods, on the pore water pressure variations, 5 tests were performed in two groups. These groups were different considering the type of input motion. 20cm of grain materials, without liquefaction potential, were employed in the tests with surcharge, so just one of the pressure transducers (No. 3) remained in the saturated sandy soil (Fig. 15). Recorded  $(r_u)$  values from this transducer were compared to those of transducer No. 1 in the test without soil improvement because of the similarity in the drainage path. Table 7 shows tests program in the groups D and E.



**Figure 14.** Time histories of input acceleration for group C



**Figure 15.** Cross section of the overburden layer

**TABLE 7.** Tests program in groups D and E

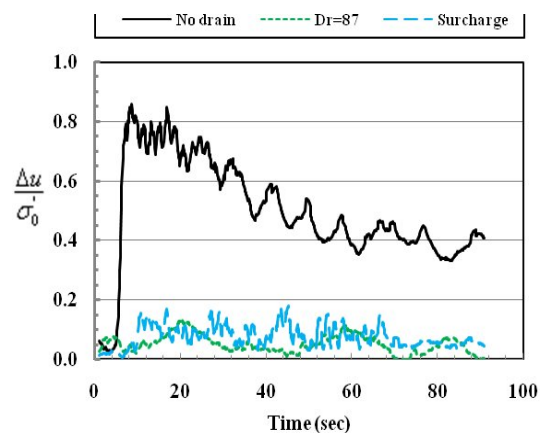
Group	Test No.	Relative density (%)	$a_{max(ave)}$	Surcharge (kPa)	$a_{max}$	Drains spacing (cm)
D	6	50	0.25g	none	0.27g	none
	11	87	0.25g	none	0.27g	none
	13	50	0.23g	3.2	0.27g	none
E	8	50	0.12g	none	0.13g	none
	12	50	0.13g	3.2	0.17g	none

The results in group *D*, presented in Table 8 and Figs. 16(a)-(c) show that the risk of liquefaction was completely removed by using the recent methods during the dynamic motion and after that. As observed earlier in group *B*, where the input motion was of type 3 like group *D*, the drains could not reduce the magnitude of  $(r_u)_{\max}$  during dynamic motion. However the ratio considerably decreased in test No. 11, where the soil was in the compacted state ( $D_r=87\%$ ). It should be noticed that using compaction method is associated with its own practical difficulties and economical costs, but it can be considered as an appropriate method against liquefaction in strong earthquakes when needed. The other method, using a soil without liquefaction potential as a surcharge on liquefiable soil was also totally effective in reducing  $(r_u)$  ratio during dynamic motion and after that in test No. 13. Since 20cm of sandy soil was replaced with grain materials in the test, only one pressure transducer (No. 3) remained in saturated soil to record excess pressures. Comparing the recorded values of tests 12 and 13 with those of No. 1 in test 6 showed an 80% reduction in  $(r_u)_{\max}$  value, reaching a certain amount. The problem of the excess pore water pressure after shaking and secondary liquefaction was also removed in tests 11 and 13.

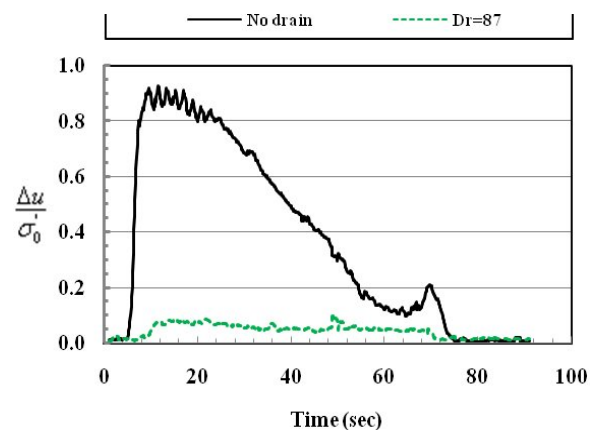
Considering the results of group *D*, using surcharge was employed for the model with input motion of type 2 in test No. 12 to investigate the method under a different input motion. The presented results in Table 9 and Fig. 17 indicate the excess pore water pressure decreased again during dynamic motion. Therefore using of unliquefiable surcharge can also be considered as a choice to remediate liquefaction in strong earthquakes.

**TABLE 8.** Maximum  $r_u$  values in group *D* for three pressure transducers

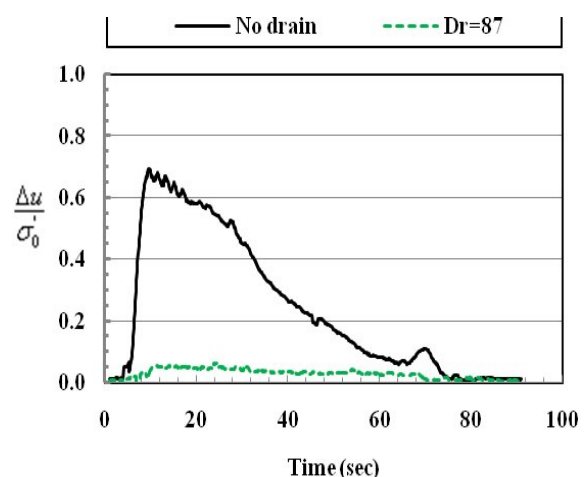
Test No.	11	6	13
Average of maximum input accelerations	0.25g	0.25g	0.23g
Relative density	87	50	50
Surcharge (kPa)	None	none	3.2
$r_{u-}$ ( $PT_1$ )	0.15	0.85	0.18
$r_{u-}$ ( $PT_2$ )	0.10	0.93	-
$r_{u-}$ ( $PT_3$ )	0.07	0.70	-



**Figure 16(a).** Time histories of pore pressure ratio for group *D* (Pressure transducer No. 1)



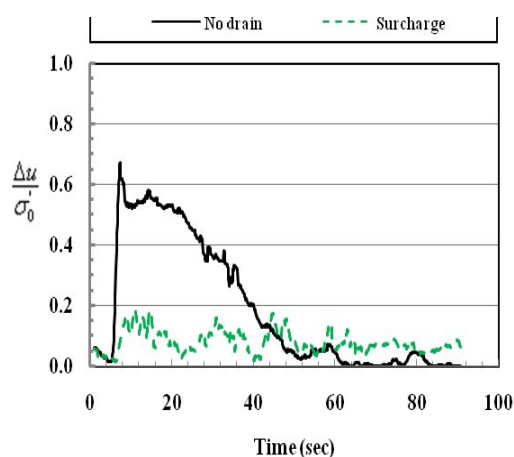
**Figure 16(b).** Time histories of pore pressure ratio for group *D* (Pressure transducer No. 2)



**Figure 16(c).** Time histories of pore pressure ratio for group *D* (Pressure transducer No. 3)

**TABLE 9.** Maximum  $r_u$  values in group E for three pressure transducers

Test No.	8	12
Average of maximum input accelerations	0.12g	0.13g
Surcharge (kPa)	none	3.2
$r_{u - \max} (PT_1)$	0.68	0.20
$r_{u - \max} (PT_2)$	0.89	-
$r_{u - \max} (PT_3)$	0.59	-



**Figure 17.** Time histories of pore pressure ratio for group E

Briefly talking, the study laboratory results, in weak shakings for example, were somehow in agreement with theoretic opinions while in some other cases, e.g. strong shakings, there was not much agreement. The results obtained coincide with the findings of the study carried out by Sadre Karimi and ghalandarzadeh who applied a strong motion to the model soil [1]. However, Brennan and Madabhushi who carried out the centrifuge modeling test suggested that the real advantages of drains may lie not in preventing liquefaction but, is effective in reducing the time that deposits spent in a liquefied state [22]. In the present study it is believed that simplistic assumptions used to solve soil-water equations in former studies, e.g. assuming laminar flow for water, were not ignorable. An increase in the effectiveness of drains after stopping the shake was because of the water flow that changed to laminar state. Meanwhile, the other two methods used, soil

compaction and using a surcharge on liquefiable soil, were successful during strong shakings.

## 6. CONCLUSION

Based on the study presented on the effectiveness of vertical gravel drains to remediate liquefaction effects using laboratory shaking table tests, the followings are the major conclusions:

1. Excess pore water pressure ratio ( $r_u$ ) values decreases by depth.
2. Excess pore water pressure dissipates in deeper strata after strong shaking, but in shallower depths the dissipation gradually increases.
3. Using gravel columns accelerates the dissipation of excess pore water pressure after stopping shaking.
4. Gravel columns show less effectiveness in shallower (near surface) depths.
5. For not-strong shakings, where drains show the most effective behavior, the obtained ratio ( $a/b$ ) from Seed and Booker's charts is conservative. A ratio of 1.5 times more than the theoretic ratio was adequate for the studied soil.
6. The effectiveness of gravel columns during dynamic motion seriously depends on the type of input motion.
7. Although gravel columns cannot be counted as complete countermeasures, they delay the time of increasing excess pore water pressure and soil liquefaction.
8. Gravel columns are more effective to reinforce soil in the earthquakes with low durations.
9. To use gravel columns as liquefaction countermeasures, the severity of earthquake in the region and determination of the optimum spacing between drains should be noticed.
10. Soil compaction is an appropriate method against liquefaction in strong earthquakes.
11. Putting surcharge by top soil layers on liquefiable soils also reduces liquefaction potential in strong motions.

## 7. ACKNOWLEDGMENTS

The authors would like to express their thanks and appreciations to reviewers of this paper for their valuable comments. We are also grateful to Ehsan Mehrabi Kermani for editing the English text.

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