RESPONSE OF GEOSYNTHETIC-REINFORCED SOIL WALLS UNDER SEISMIC CONDITION BY SHAKING TABLE TESTS

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ABSTRACT: Recent earthquakes have permitted to compare the different failure modes between traditional retaining structures and most recent reinforced soil walls. All these events show that the performances of the retaining structures in the same seismic area are very different. During Kobe earthquake, for example, a great number of retaining structures were damaged, but reinforced soil structures show a very limited damage and permanent displacements. The paper presents the results of the tests performed on the shaking table of University of Catania, Italy. The tests are conducted with a model of geosynthetic-reinforced soil wall 0.35m high. A polypropylene biaxially oriented geogrid was used as soil reinforcement. The models were subjected to a sinusoidal input and to the E-O component of the 1990 Catania earthquake. The models were instrumented to measure lateral facing displacements and acceleration response along the height of the wall face and within the backfill. The results of the experimental program shows that reinforced soil wall recorded a reduction of the permanent displacements of the facing, comparing with the gravity retaining wall, and a consequently reduction of the backfill deformation. The failure surface observed does not agree with the one predicted by M&O method. The reinforced soil wall shows a dynamic response as a function of its stiffness; the influence of frequency motion is also evident.

1 INTRODUCTION

The evaluation of the dynamic behaviour of the geosyntetic-reinforced retaining structures is a relatively recent field of study due to the observation of the retaining-structure damages during the recent earthquakes, showing a good performance during the motion, with a reduction of permanent deformations and displacements. The theories enveloped in the recent years are based on the pseudo-static limit equilibrium of the Mononobe&Okabe analysis (1929), that cannot take into account the effects of soil deformation and the consequent degradation of internal friction angle to residual state, that occur in the denser soil during an earthquake. The method is based on three assumption: the wall has already deformed outwards sufficiently to generate the minimum active earth pressure; a planar failure surfaces were formed when lateral soil deformation becomes large enough to fully mobilize the shear strength of the soil; the soil wedge formed is considered as a rigid body so that the acceleration is maintained constant with depth and amplification effect is not observed.
All the tests performed by the authors, had shows that only the first assumption is satisfied. Many authors have study the design problem, with various solutions. Yet there is not enough experimental analysis to validate these theories and especially to evaluate the effects of the distribution of the reinforcements inside the wall.

Juran and Christopher (1989) describes the results of a laboratory model on the performance, behaviour and failure mechanisms of reinforced soil-retaining walls using different materials, showing that the confinement of the reinforcement has a major effect on the structures performance. Law, Ko, Goddery and Tohda (1992) have performed a certain number of centrifuge tests to predict the response of the full scale geosyntetic-reinforced wall. All the test performed shown that the wall stability was depending by the strength of geotextile, density of soil, loading history and stress level in the ground. Some other full-scale test were performed to evaluate the performance of reinforced soil-wall with rigid facing, showing a good response of the wall against severe earthquake, including foundation liquefaction (Murata, Tateyama & Tatsuoka, 1994). Shaking table tests were performed by Sugimoto, Ogawa & Moriyama (1994), basing the tests on the newly developed similarity rule. The tests results shown a good stability of reinforced structures and the dynamic failure was depending by the external stability condition.

Michalowski (1997), using a kinematic approach of limit analysis for the stability analysis of reinforced slope, have evaluated the reinforcement necessary to prevent collapse of slopes due to reinforcement rupture, pullout or direct sliding, obtaining a design charts for the required reinforcement strength and its length. To this aim was defined a dimensionless parameter:

$$k_t = \frac{nT_t}{H} \tag{1}$$

where n = number of reinforcement layers, H = height of the slope and T_t = strength of a single layer.
Leshchinsky & Perry (1997) have proposed a seismic design procedure for geosyntetic-reinforced soil structures, basing the assumption on the method on a pseudo-static limit equilibrium analysis and considering a permanent displacement limit. Through a parametric study conducted the authors have performed a method to a best design for reinforced structures. The method introduce a tolerable permanent displacement when the required length is extremely long and it can be used also to design the soil cover of waste containment. This method was extending by Ling and Leshchinsky (1998) considering the effects of the vertical acceleration, showing that for horizontal acceleration value grater than 0.2, the vertical component is significant for the structure stability.
The stability method based on the kinematic theorem of limit analisys was also used by Ausilio, Conte and Dente (2000), considering two different failure modes. A procedure based on the assessment of a earthquake-induced permanent displacement is proposed for the design of reinforced slopes in seismically active areas.

In recent years further experimental and numerical analysis were conducted on reinforced wall model by El-Emam, Bathurst, Hatami & Mashhour (2001), Sofronie, Taylor and Iosif (2001), Watanabe, Tatteyama, Kojima and Koseki (2001), Hatami & Bathurst (2001).
This paper gives an experimental contribution to all these approaches showing that: the vertical distribution of reinforcement, under seismic condition, play an important role to the global stability of the reinforced structure; the distribution of pressure in dynamic condition is non-linear; the point of application of total thrust is strongly dependent by the wall movements during the motion; the distribution is also depending by the degradation of soil-reinforcement friction angle. The dynamic approach was computed using the pseudo-dynamic approach of the method of Steedman and Zeng, (1990) where the sinusoidal acceleration used as base shaking is given by

$$\ddot{u}(z,t) = k_h g\,\mathrm{sen}\,\omega\left(t - \frac{H-z}{V_s}\right) \qquad (2)$$

where Vs is the shear wave velocity, ω is the pulsation of the lateral shaking, H is the height of the wall and z the current height. The final expression of the distribution of the lateral pressure can be expressed as:

$$p_{AE} = \frac{\cos(\alpha-\phi)k_h\gamma z}{\cos(\delta-\alpha+\phi)\tan\alpha}\mathrm{sen}\omega\left(t-\frac{z}{V_s}\right)+\frac{\mathrm{sen}(\alpha-\phi)\gamma z}{\cos(\delta-\alpha+\phi)\tan\alpha} \qquad (3)$$

which is clearly non linear.

2 EXPERIMENTAL PROCEDURE

The shaking table available at the laboratory of the University of Catania consists of a steel frame and a steel plate bolted on the frame, it is 2 m long, 1 m wide and 80mm thick and is supported by four rollers constrained to move on rails, in order to restrict the motion only to one direction. The test box is 1 m long, 0.70 m wide and 0.40 m high. The motion is provided to the table by a loading unit consisting of an hydraulic system, with a capacity to transfer a static load of 3000 daN and a dynamic load equal to 1000 daN when the acceleration value is equal to 2g. The system is able to transfer a maximum displacement equal to $\pm$25mm. The sides of the test box are made of transparent glass with the purpose to observe the model deformation and the failure surface during the test. The thickness of the glass sides was chosen equal to 10mm in order to ensure a plane-strain deformation.

The geosyntetic-reinforced soil wall used in the test was designed with height H=35 cm. The soil used in all tests is a dry silica sand from the Sicily east coast which characteristics are D_{60}/D_{10} = 2.407, D_{50} = 0.42mm, maximum and minimum unit weight γ_{max} = 18.27 KN/m^3 and γmin = 15.04 KN/m^3 respectively, and peak value of the shear strength φ = 37°, obtained at the same relative density used in the tests as a result of a certain number of direct shear tests. Backfill were prepared by dry pluviation in the test box, in which the deposition height is maintained constant respect the backfill, obtaining a final relative density $D_r \cong$ 75%.

In each test the model was instrumented with two accelerometers and two LVDT displacement transducers, with the purpose to record accelerations, facing displacements and deformation, at the top and at the base. One other accelerometer was placed into the backfill outside the reinforced zone to measure the soil acceleration.

A polypropylene biaxially oriented geogrid was used as soil reinforcement. The reinforcements were uniformly distributed over the height of the wall with a step of 0.05m. With the purpose to investigate on the spacing effects some other tests were conducted with a non-uniform distribution of reinforcements. The step used for these tests is of 0.05m from the base to the height of 0.25cm and of 0.035m until the top op the wall.

This distribution was choose from the observations of the tests with uniform distribution that had showed a lateral facing deformation more evident in the upper zone of the wall due to the inertial force concentration in this zone.

The model facing was made by a certain number of aluminium L-shaped sections, connected by a metal hinge and placed into the test box along a vertical guide to support the construction of the model.

The reinforcements calculated for the model, were anchored to the vertical facing in the internal face of each aluminium element.

Figure 1 and figure 2 show the model during the construction and ready to the test respectively.

The required length of the geogrid was calculated with the application of Mononobe and Okabe method. The models were subjected to a sinusoidal input, where the frequency and amplitude was alternatively varying, and to the E-O component of the 1990 Catania earthquake.

A data acquisition system and software for data processing were employed to record and analyze the data obtained during dynamic testing. To observe the formation of the failure surface through the glass side of the test box, vertical coloured sand marchers were introduced into the backfill.

The geosyntetic-reinforced soil-wall systems were subjected to a sinusoidal input acceleration whose amplitude was strongly increased with time until to a maximum value; the frequency is maintained constant.

A certain number of models were investigated; in this paper were reported the more representative tests.

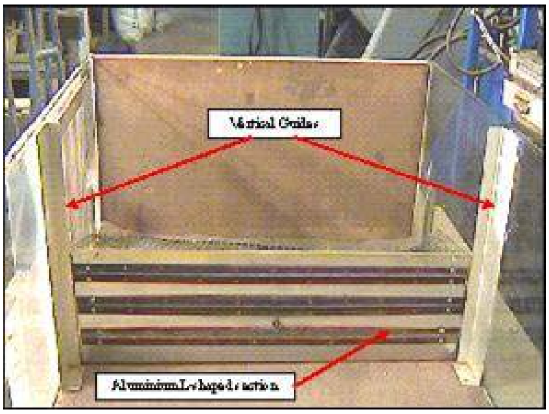

Figure 1 The model during construction

Figure 2 The model ready to the test

3 EXPERIMENTALE RESULTS

3.1 *Model with uniform distribution of the reinforcements*

In figure 3 are reported both the accelerations and displacements time-history for the test named TH6 in which the input motion is fixed to a frequency of 7Hz with maximum amplitude equal to 2mm, able to transfer a maximum acceleration of 0.6g.

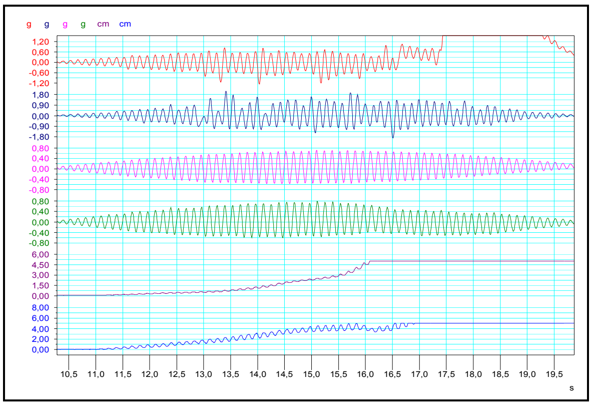

Figure 3 Accelerations (Top and base wall, input and backfill) and displacements (base and top) time-history for the test named TH6

The accelerometric data shows that prior to threshold acceleration the wall acceleration is similar to the acceleration in the backfill; when input acceleration overcome such threshold, a cut-off acceleration for the wall is clearly evident, indicating that a relative acceleration has developed in the system and a series of amplification phenomena appeared.

It is apparent that permanent displacements build up in the outward direction when the table is moving backward. After 12.5 seconds, when the input acceleration is equal to 0.35g the wall top acceleration showed amplification phenomena until to 0.80g.

This behaviour is more evident for the accelerations in the backward direction, that is when the wall move outward.

Moreover to the amplification phenomena recorded are associate a difference of phase between the accelerometric records of input and of wall top. This phase change in horizontal acceleration plays an important role on the distribution of the dynamic increment of the earth pressure and on the stability of the wall.

In figure 4 are reported the input and wall top accelerations, and top displacements.

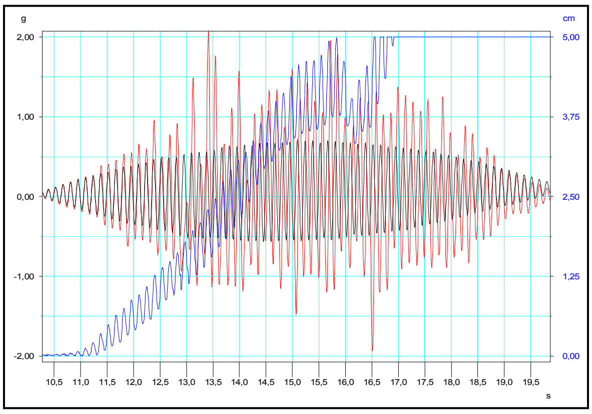

Figure 4 Accelerations (Top and input and top displacements) time-history

It's possible to observe that the accumulation of permanent displacements was more gradual, but large oscillation appeared at the top of the wall from the beginning of the shaking test and increases when the wall and the table accelerations are negative, that is, directed backward.

It can be observed a first stage (until 13 sec.) in which the top of the wall, accumulating permanent displacements, has the tendency to rotate, maintaining the large oscillation. A second stage (after 13 sec), in which, is reached the threshold acceleration value and a sudden increase of the permanent wall base displacement was observed. Large oscillations produced large deformations of lateral facing, with a consequent collapse of the structure that recorded very large final permanent displacements. In figure 5 are reported the photos of the system before and after the test named TH6.

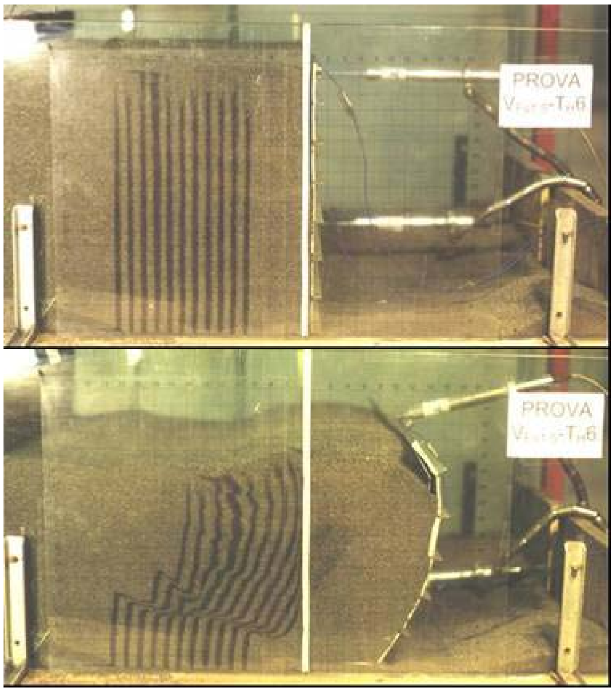

Figure 5 Test TH6 before and after the test

The photos of figure 5, show a clear formation of two distinct failure surfaces with an angle respect to the horizontal of 16° and 26° respectively.

In figure 6 is reported the test named TH2 in which the input motion is fixed to a frequency of 4Hz with maximum amplitude equal to 5mm able to transfer a maximum acceleration of 0.4g.

In figure 7 is reported the test named TH4 in which the input motion is fixed to a frequency of 6Hz with maximum amplitude equal to 2mm able to transfer a maximum acceleration of 0.4g.

The recorded data and the deformation of vertical lateral facing for tests TH2 and TH4, shows that the frequency of the input motion influenced the oscillation especially near the top of the wall with a similar final value of the permanent displacement.

In figure 8 is reported a sketch in which were drawing the recorded deformations of lateral facing for the previous three tests; it can be observed the similar deformation for the test TH2 and TH4, whereas a large deformation, when the input frequency was increased to 1 Hz, is evident.

All the tests performed with a vertical uniform distribution of the reinforcement, show a tendency of the systems to accumulate large deformation near the top of the wall, due to the incremental of inertial forces that appear when the threshold acceleration was reached.

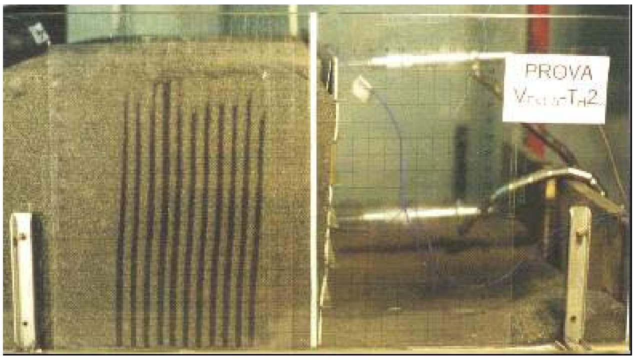

Figure 6 Test TH2 after the test

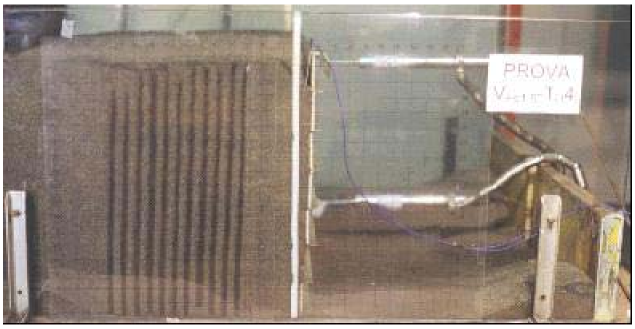

Figure 7 Test TH4 after the test

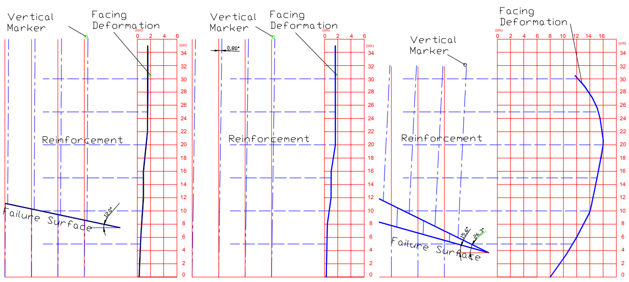

Figure 8 a sketch in which were drawing the recorded deformations of lateral facing for the tests TH2, TH4 and TH6.

With the purpose to reduce these deformations and, at the same time, to evaluate the effects of the distribution of the vertical reinforcements the same tests were investigated with a different distribution of the reinforcements as described in the previous experimental procedure.

3.2 *Model with non-uniform distribution of the reinforcements*

In figure 9 is reported the photo of the test, named TEST8, which input characteristics are the same of the test TH6. The reduction of permanent deformation clearly appeared. It can be observed that the lateral deformation maintaining his value constant with the height of the wall, due to a reduction of the deformation near the top of the wall where the reinforcements were increased.

Also for this test amplification phenomena of the top acceleration was clearly appeared and large oscillation were recorded by the displacement transducers. In figure 10 are reported the input and wall top acceleration time-history and wall top displacements.

The effects of the reduction of the deformation of the lateral facing of the geosyntetic-reinforced soil-wall is further on evident in the test named TEST5 in which the input characteristics are the same of the test TH4, that is able to transfer an horizontal acceleration equal to 0.4g. In this test the system shown all the amplification phenomena near the top of the wall, but a little lateral deformation was

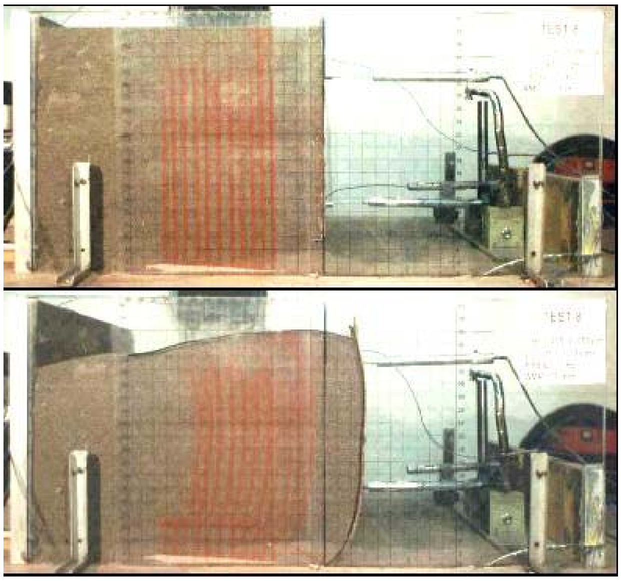

Figure 9 TEST8 before and after the test

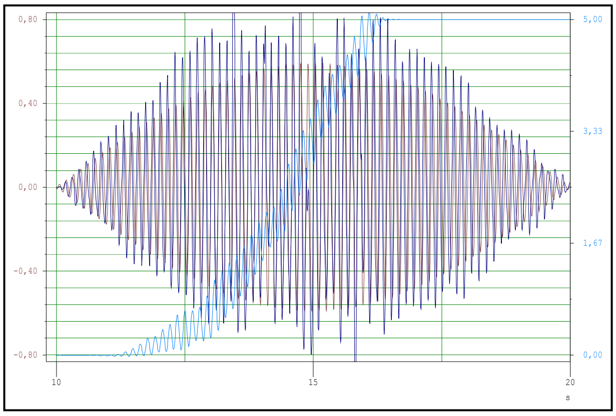

Figure 10 Acceleration and displacement time-history for the wall top of the TEST8

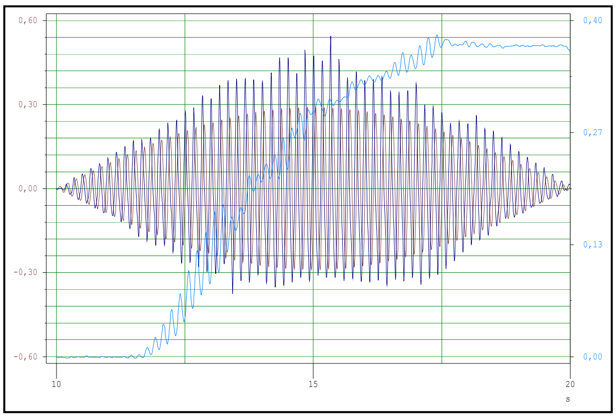

Figure 11 Acceleration and displacement time-history for the wall top of the TEST5

recorded with a final permanent displacement equal to 0.3cm, reducing the previous value by a scale factor equal to 6.

In figure 11 are reported the input and wall top acceleration time-history and wall top displacements. In figure 12 was reported the photo of TEST5 before and after the test and no evident deformation of vertical lateral facing appeared. Some other tests were performed using, as a input motion, the accelerometric signal recorded during the Catania earthquake in 1990, E-W component.

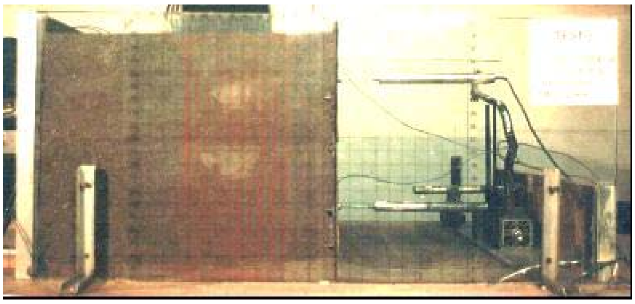

Figure 12 TEST5 before and after the test

The tests, named CT90, shows a good performance of the model with the no-costant distribution of the reinforcements and a reduction of permanent vertical lateral facing deformation was appeared when the peak value of acceleration reached the value of 0.8g. In figure 13 are reported the accelerometric data at the top of the wall and input together with top displacements.

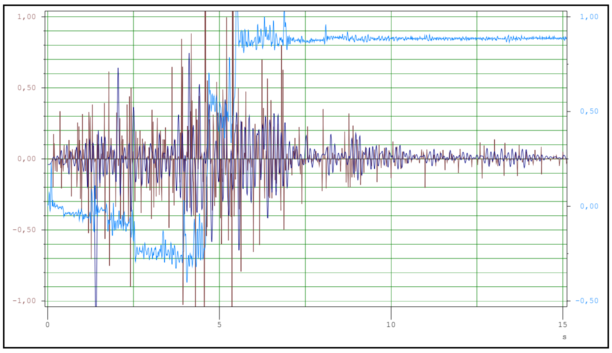

Figure 13 Test CT90: input, top wall acceleration and displacement time-history.

Figure 14 shows a photo of this system after the test and a detail of the failure surface formed during the motion.

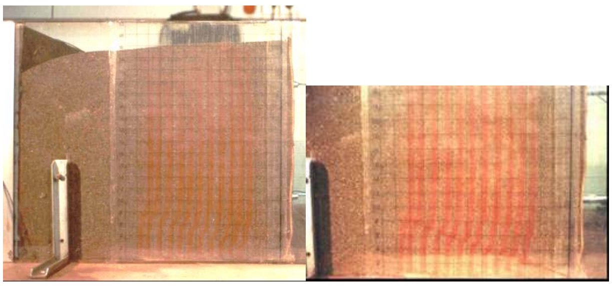

Figure 14 Test CT90 after the test and (on the right) a detail of failure surface for Test CT90.

3.3 *Discussion of recorded data*

Comparing the tests results it is possible to observe that an increment of the reinforcements near the top of the wall produce a good response to the dynamic load, because the inertial forces effects were reduced. An explanation for this behaviour can be taken considering that the soil-reinforcement interaction is fully mobilized when the geogrid were placed uniformly and when the threshold acceleration was reached; at this instant the reinforcement layers begin to take a larger portion of the load producing pull-out effect into the model. The new load distribution with the new non-uniform reinforcements disposition produce a reduction of the total force transfer to the layers and the interaction between soil and geogrid is not fully mobilized.

Moreover, analyzing the recoded data can be underlined the very large increment of acceleration level of wall top. At this behaviour are associate two different types of displacements: a recoverable and an irrecoverable that occurs for every cycle input load; the first, defined as “elastic displacement” can be calculated and his effects on acceleration level can be expressed by the follow equation

$$\ddot{s}'_w = \frac{\partial^2(s+x_t)}{\partial t^2} = \frac{\partial^2 s}{\partial t^2} + \frac{\partial^2 x_t}{\partial t^2} \qquad (4)$$

where “s” is the displacement imposed by the input motion and “x_t” is the relative displacement between the soil-table system and the top wall.

The tests permormed have shows that, the “elastic” displacement of the wall top had an important influence on the distribution of dynamic pressure.

The first consequence is a very high point of application of the total thrust. Moreover, as showed in the tests, during the motion a difference of phase between accelerometric records was appeared, especially near the top where large oscillation were recorded. These states of stress are due to the different stiffness that occurs between the soil and the wall, especially during the motion when the soil degradation and the effects of strain localization and post-peak reduction in shear resistance occur in the backfill soil during an earthquake generating a reduction in the natural frequency of the soil-structure system. In table 1 are reported, for all the tests performed, the values of input and top wall accelerometric peak value, and the amplification factor expressed in percent. In the second column of the table are reported the input characteristics of the base load.

Table 1 Results of accelerometric data

Test	Fr/Am	Input+	Top+	I>T	T>I
	(Hz/mm)	(a/g)	(a/g)	%	%
1	4/4	0,25	0,35	-	40
2	4/5	0,32	0,40	-	25
9	5/2	0,20	0,30	-	50
10	5/3	0,30	0,42	-	40
3	5/4	0,40	0,66	-	65
4	5/5	0,50	0,64	-	28
5	6/2	0,28	0,46	-	64
6	6/3	0,43	0,78	-	81
7	7/2	0,4	0,77	-	93
8	7/3	0,6	0,8	-	33
ct1	90EW	0,80	0,64	25	-

Test	Fr/Am	Input-	Top-	I>T	T>I
	(Hz/mm)	(a/g)	(a/g)	%	%
1	4/4	-0,25	-0,35	-	40
2	4/5	-0,32	-0,40	-	25
9	5/2	-0,20	-0,30	-	50
10	5/3	-0,30	-0,38	-	27
3	5/4	-0,40	-0,45	-	13
4	5/5	-0,50	-0,45	11	-
5	6/2	-0,28	-0,36	-	29
6	6/3	-0,43	-0,54	-	26
7	7/2	-0,40	-0,56	-	40
8	7/3	-0,60	-0,72	-	20
ct1	90EW	-0,80	-0,64	25	-

The experimental results compare well with the general assumption that the acceleration amplification factors are larger for the model with a hinged toe. With the aim of the lateral glass side of test box, for each test the failure surface envelopment was observed. A similar behaviour was observed for each test. A planar failure surface was formed when lateral displacement was high enough to fully mobilize the shear resistance of the soil-reinforcement system.

The failure surfaces compare well with those computed with Mononobe&Okabe theory as a function of the horizontal acceleration. Moreover, for all tests with different distribution of reinforcements, a secondary failure surface was observed, as a consequence of “direct sliding” behaviour. This failure surface appeared at 11cm from the base of the wall, corresponding to the second layer of geogrid.

This failure surface can be associated to a different stiffness between the two reinforced zones inside the wall. In figures 16, are reported the failure surfaces details for the test performed named TEST3 and TEST7. For these tests the peak acceleration value was 0.4g with an amplification factor of 65% and 93% respectively.

In figures 17, are reported the failure surfaces details for the test named TEST8, where the peak acceleration value was 0.6g with an amplification factor of 33%.

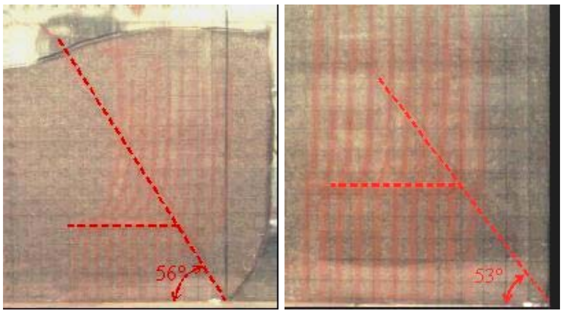

Figure 16 A detail of failure surface for TEST3 and TEST7

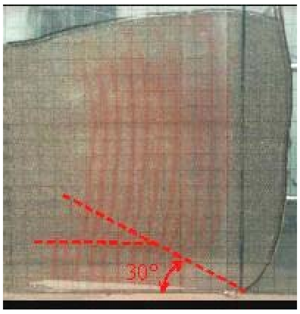

Figure 17 A detail of failure surface for TEST8

4 CONCLUSION

The following conclusions were made based on the experimental results obtained and discussed in this paper:

- The vertical distribution of reinforcements influences the permanent final deformation of the system;
- An “elastic” displacement is exerted by the wall during the initial stage of the motion producing large increment of acceleration level at the wall top;
- an increment of the reinforcements near the top of the wall produce a good response to the dynamic load, because the inertial forces effects were reduced
- Large amplification phenomena of accelerations near the top of the wall were recorded and that is associated to a difference of phase between the accelerometric records of input and of wall top. This phase change in lateral acceleration plays an important role on the distribution of the dynamic increment of the earth pressure and on the stability of the wall
- Failure surfaces compare well with those computed with Mononobe&Okabe theory as a function of the horizontal acceleration;
- No breakages were observed into the reinforcements;
- The tests result underline that the design of the geosynthetic-reinforced soil wall must take into account the strain compatibility between the soil and the reinforcement;
- For the systems with the increment of the reinforcements near the top of the wall is necessary an higher threshold acceleration, able to produce permanent deformation, respect the same system with uniform distribution of reinforcement.
- A kinematics criteria to design reinforced structures under seismic condition is necessary to predict the permanent displacement that occur and an accurate disposition of the reinforcements must be observed to mitigate their value

5 REFERENCE

Mononobe N., Matsuo R. 1929 - On the Determínation of Earth Pressure during Earthquake, Paper No. 388, Proc. of World Engineering Congress, Vol. 9, 1929

Juran, I., Christopher, B. 1989: Laboratory model study on geosynthetic reinforced soil retaining walls., Journal of Geotechnical Engineering, ASCE, 115 (7), pp. 905-927

Law, H., Ko, H. Y., Goddery, T. & Tohda, J. 1992: Prediction of the performance of a geosynthetic-reinforced retaining wall by centrifuge experiments, Geosynthetic-Reinforced Soil Retaining Walls, Wu (ed.), Balkema, Rotterdam

Murata O., Tateyama, M., Tatsuoka, F. 1994: Shaking table tests on large geosynthetic-reinforced soil retaining wall model, Proceeding of Seiken Symposium No 11, Tokyo, Japan, 6-7 November 1992, “Recent Case History of Permanent Geosynthetic-Reinforced Soil Retaining Walls”, Tatsuoka, F. & Leshchinsky, D. (eds), Balkema, Rotterdam, pp. 259-264

Sugimoto, M., Ogawa, S., Moriyama, M. 1992: Dynamic characteristic of reinforced embankments with steep slope by shaking model tests, Proc. of Seiken Symposium No 11, Tokyo, Japan , 6-7 November 1992, “Recent Case History of Permanent Geosynthetic-Reinforced Soil Retaining Walls”, Tatsuoka, F. & Leshchinsky, D. (eds), Balkema, Rotterdam, pp. 271-275

Michalowski, R. L. 1997: Stability of uniformly reinforced slopes, Journal of Geotechnical Engineering, ASCE, 123 (6), pp. 546-556

Ling, H. I., Leshchinsky, D., Perry, E. B. 1997: Seismic design of geosynthetic-reinforced soil structures, Géotechnique 47, No. 5, pp. 933-952

Ling, H. I., Leshchinsky, D. 1998: Effects of vertical acceleration on seismic design of geosynthetic-reinforced soil structures, Géotechnique 48, No. 3, pp. 347-373

Ausilio E., Conte, E., Dente G. 2000: Seismic Stability Analysis of reinforced slopes, Soil Dynamics and Earthquake Engineering, 19(3), pp. 159-172

Hatami K., Bathurst R. J. 2001: Investigation of Seismic Response of Reinforced Soil Retaining Walls, Proc. of 4th International Conference on Recent Advances in Geotechnical Earth Engineering and Soil Dynamic and Symposium in Honour of Prof. W. D. Liam Finn, San Diego, California, U.S.A., 26-31 March 2001, paper No. 7.18

Watanabe K., Tatteyama M., Kojima K., Koseki J. 2001: Irregular shaking table tests on seismic stability of reinforced-soil retaining walls, Proceeding of the International Symposium on Earth Reinforcement, Fukuoka, Kyushu, Japan, 14-16 November 2001, Landmarks in Earth Reinforcement, Ochiai, H., Otani, J., Yasufuku, N. & Omine, K. (eds), Swets & Zeitilnger, pp. 465-471

Sofronie R. A., Taylor C. A., Iosif, F. 2001: Seismic behaviour of earth reinforcement walls, Proceeding of the International Symposium on Earth Reinforcement, Fukuoka, Kyushu, Japan 14-16 November 2001, Landmarks in Earth Reinforcement, Ochiai, H., Otani, J., Yasufuku, N. & Omine, K. (eds), Swets & Zeitilnger, pp. 465-471

El-Emam. M, M. Bathurst R. J., Hatami K., Mashhour M., 2001: Shaking table and numerical modelling of reinforced soil walls, Proceeding of the International Symposium on Earth Reinforcement, Fukuoka, Kyushu, Japan, 14-16 November 2001.

Steedman R. S., Zeng X. 1990: the influence of the phase on the calculation of pseudo-static earth pressure on a retaining wall, Géotechnique 40, No. 1, pp. 103-112.