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ABSTRACT

Seismic design of retaining walls is traditionally based on the Mononobe-Okabe method of analysis. In recent years a number of theoretic analyses have been presented to predict the seismic behaviour of gravity retaining walls. In this paper some shaking table tests performed a small prototype of gravity wall retaining dry sand are described and the experimental results are presented with the aim to provide, thou qualitatively, an insight into some important aspects of the dynamic behaviour of retaining structures resting on rigid foundation soil. The M- theory do not consider the particular boundary condition that in the practical design of retaining structures are often in use like backf geometries or loading condition. Shaking table studies were carried out in order to study the dynamic behaviour of gravity retaining walls resti on rigid foundation soil. Two different system have been taken into consideration namely, a wall retaining a horizontal backfill on which uniform surcharge was placed and a wall on which the uniform surcharge was placed to a distance «d» to the head of the wall.

INTRODUCTION

Recent earthquakes have shown that retaining walls, though designed according to a seismic criteria, are vulnerable against strong seismic actions.. For example, the Hyogoken-Nanbu Earthquake of January17,1995, caused serious damage to conventional gravity retaining walls for railway embankments. Despite a number of studies have addressed the problem of seismic response and stability of earth retaining walls, there is still a need of understanding of the complex interaction phenomenon occurring between the retained soil and the wall during earthquakes.
In earthquake resistant design of retaining walls, the Mononobe-Okabe theory, is currently used to predict the seismic earth pressure. This theory derives from the method of seismic coefficient and Coulomb's equation for active earth pressure, and its validity is limited to extremely simple cases. In fact the M-O theory do not consider the particular boundary condition that in the practical design of retaining structures are often in use like backfill complex geometries or loading conditions. Shaking table studies were carried out in order to study the dynamic behaviour of gravity retaining walls resting on rigid foundation. Two different systems have been taken into consideration namely, a wall retaining a horizontal backfill on which a uniform surcharge was placed and a wall on which the uniform surcharge was placed at a distance «d» from the top of the wall.

THEORETICAL METHODS OF ANALYSIS

Several methods of analysis have been recently proposed in order to overcome the inadequacies of the M-O theory in evaluating the earth thrust acting on a retaining wall when an infinite uniform distributed surcharge is applied on the surface of a horizontal inclined backfill (Motta, 1994; Caltabiano et al 1999; Caltabian 2000). If the surcharge is placed at a certain distance from the t of the wall, the M-O theory is valid as for as the surcharge completely beyond the intersection between the failure surface a the backfill profile. In other terms, if the surcharge affects only portion of the failure wedge, the M-O solution cannot consistently utilized (fig.1).

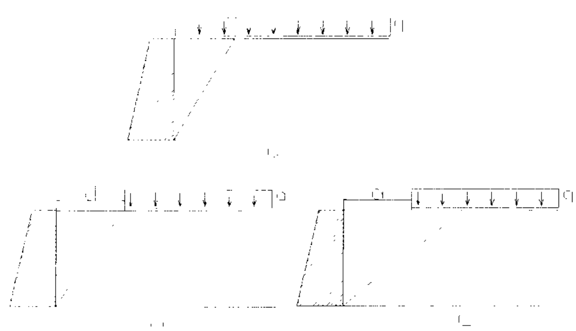

Fig. 1. Wall with surcharge on the backfill: a,b) M-O solution applicable; c) M-O solution not applicable.

In the case of Fig.1a the classical pseudo-static theory gives th following solution for the earth thrust of a homogeneous, dry soi

$$S_a = \frac{1}{2}\gamma H^2 K_{a\gamma} + qHK_{aq}$$

Where the symbols have the usual meaning.

For the case of fig.1c Motta [1994] proposed:

$$S_a = \frac{1}{2}\gamma H^2 K_{a,\gamma q}$$

with:

$$K_{a,\gamma q} = \frac{(1+n_q)\cos^2 i[1 - A\tan(\alpha_c - i)][\cos b - \text{sen}\, b/\tan(\alpha_c - i)]}{\cos\theta[\cos a + \tan(\alpha_c - i)\,\text{sen}\, a]}$$

corresponding to the angle α_c formed by the failure surface with the horizontal given by the expression:

$$\tan(\alpha_c - i) = \frac{\text{sen}\, a\, \text{sen}\, b(\text{sen}^2 a\, \text{sen}^2 b + \text{sen}\, a \cos a\, \text{sen}\, b \cos b + A\cos c \cos a\, \text{sen}\, b)^{0.5}}{A\cos c + \text{sen}\, a \cos b}$$

where the following position have been introduced:

$$a = \varphi' + \delta - i \ ; \ b = \varphi' - i - \theta$$

$$A = \frac{[(1+n_q)\text{sen}\, i \cos i + \lambda n_q]}{(1+n_q)\cos^2 i}; \ \lambda = d/H \ ; \ n_q = 2q/\gamma H$$

Caltabiano et al, (1999) addressed the problem of pseudostatic limit equilibrium of a soil-wall system with a surcharge on a backfill, introducing in the equilibrium equation the shear force mobilized at the base of the wall and due to the soil-wall friction interaction.

This solution is valid for horizontal backfill, but it can be extended to the case of inclined backfill (Caltabiano, 2000). Though the above mentioned solution are quite rigorous, there is however the need of validation against experimental evidence.

EXPERIMENTAL PROCEDURE

The shaking table available at the laboratory of the University of Catania was described by Cascone and Maugeri [1995], Cascone, et al. [2000] and is shown in fig.2.

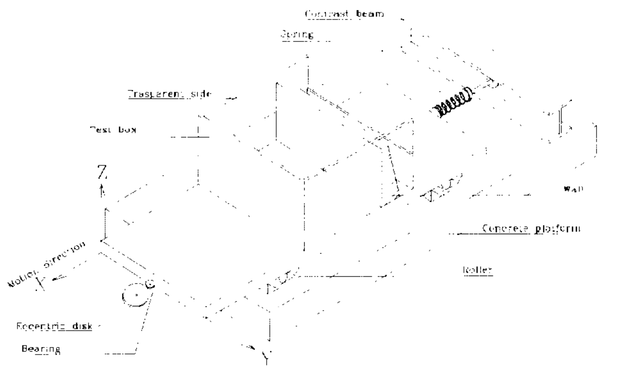

Fig. 2. The shaking table

The table consists of a steel frame and a steel plate bolted on the frame, it is 2 m long, 1m wide and 80 mm 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 1m long, 0.7 m wide and 0.4 m. The motion is provided to the table by a loading unit consisting of an electric three-phase synchronous engine with a steel disk mounted on the engine shaft. The position of the disk is adjustable allowing to produce different eccentricities in the range 1-10 mm.

The motion is transferred from the engine to the table by means of a ball-bearing placed on the edge of the table. The contact between the disk and the bearing is maintained by a spring fixed on a contrast beam and kept compressed throughout the dynamic testing. The sides of the test box are made of transparent glass and allow the observation of the model during the test. The thickness of the glass sides was chosen equals to 10 mm in order to reproduce a plane-strain condition.

The wall used in the tests is a microconcrete gravity retaining wall of height H=cm 30 consisting of a base of 10 cm and 2.5 cm thick at the top. The wall was designed using the Mononobe-Okabe theory to resist a maximum earthquake acceleration a=0.2g and the backfill without surcharge. The wall was designed to undergo, in absence of surcharge, a translational failure mode. This allowed to provide an insight on the rotational effects due to both surcharge and the surcharge inertia force on the soil-wall system. In order to avoid friction between the wall and the glass sides of the box, the wall was made 5mm shorter than the box width and the wall ends were equipped with flexible plastic flags to prevent sand passing through the lateral gaps. The soil used in the test is a silica uniform (D_{60}/D_{10} = 1.60) sand from the southern coast of Catania (Sicily), which has small (D_{50} = 0.3 mm) sub-angular grains, maximum and minimum unit weight γ_{max} = 16.8 KN/m^3 and γ_{min} = 14.5 KN/m^3, respectively, and peak value of the angle of shear strenght φ = 35°, obtained as a result of a certain number of direct shear tests at different values of relative density [Cascone et al.2001]. Backfills were prepared by dry pluviation in the test box from a constant height of 70 cm, in order to obtain a relative density D_R = 85%. The effects of relative density on the angle of shear strenght for this sand, was shown to be negligible (Lo Grasso, 1999) In each test the wall was instrumented with two accelerometers and two LVDT displacement transducers to record both accelerations and displacements at the top and at the base; two accelerometers were placed in the backfill: one at a depth of 27 cm and the other almost at the backfill surface; one additional accelerometer was fixed on the table to record the input motion. A data acquisition system and a software for data processing were employed to record and analyse data obtained during dynamic testing. Fig.3 shows the experimental setup.

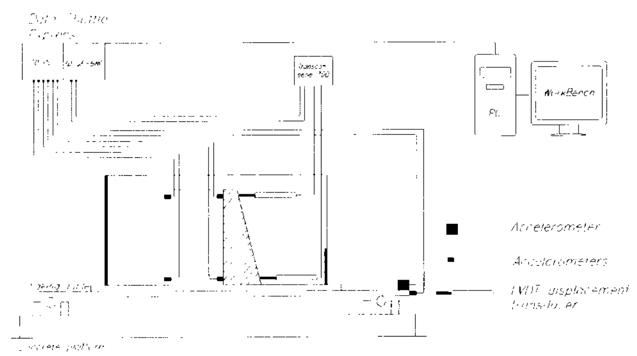

Fig. 3. The experimental setup

In order to detect the formation of the failure surface in the backfill, vertical black sand markers were introduced in the model. To model the surcharge on the backfill, cubic limestone blocks were displayed in rows parallel to the wall: this disposition allowed to avoid the introduction of an undesired shear resistance on top of the backfill as well as to show clearly the development of failure in the soil. The limestone blocks transferred an average pressure of q=0.7 kPa to the backfill. Two different system has been taken into consideration, namely a wall retaining a horizontal backfill with the surcharge placed right behind the wall (d=0, λ=0) and a wall retaining a horizontal backfill with the surcharge placed to a distance d=H/2 (λ=0.5).

TEST RESULTS

The soil-wall systems were subjected to a sinusoidal input acceleration whose amplitude was strongly increased with time. In fact, the table displacement was adjusted at 2 mm and both table frequency and acceleration were varied until a failure surface was clearly distinguished through the glass sides of the test box.

Figure 4 shows a sketch that compare the two tested systems after the development of the failure surface. During testing it was observed that the soil forming the failure wedge moved together with the wall approximately behaving like a rigid block. In fact no permanent deformation occurred until the input acceleration reached a critical amplitude. Although a slight concavity was detected in the failure surfaces, these can be reasonably assimilated to planes originating from the heel of the wall. For the case of the system with λ=0 the failure surface was inclined at an angle α=60° to the horizontal. For the case of the system with λ=0.5 the angle formed by the failure surface with the horizontal was found to be α=52°.

Fig. 4. Sketch of the two tested system.

Photos of the two system subjected to shaking table tests are shown respectively in Figure 5 and Figure 6. In such photos the system are shown before (figs. 5a and 6a) and after (figs. 5b and 6b) the formation of the failure surface. It is apparent the effectiveness of close black sand markers in giving clear evidence of the failure surface. It is worth noting that for the system with the distanced surcharge the failure surface intercepts the surcharge: this situation is not consistent with the hypotheses underlying the M-O theory.

The time-histories of accelerations and displacements of the soil-wall system with the surcharge placed close to the top of the wall are shown in fig. 7a. It can be observed that permanent wall base displacements start to build up after about 13 seconds of shaking, when input peak acceleration is 0.10g. Conversely, top displacements started accumulating from the beginning of the shaking test, showing the tendency of the wall to rotate because of the surcharge. At any increase of the input acceleration amplitude (at 16 and 18 sec) a sudden increase of the permanent wall base displacement corresponded, and finally, at about 22 seconds a slight increase in the acceleration produced an abrupt change in the rate of displacement accumulation. At this stage the input peak

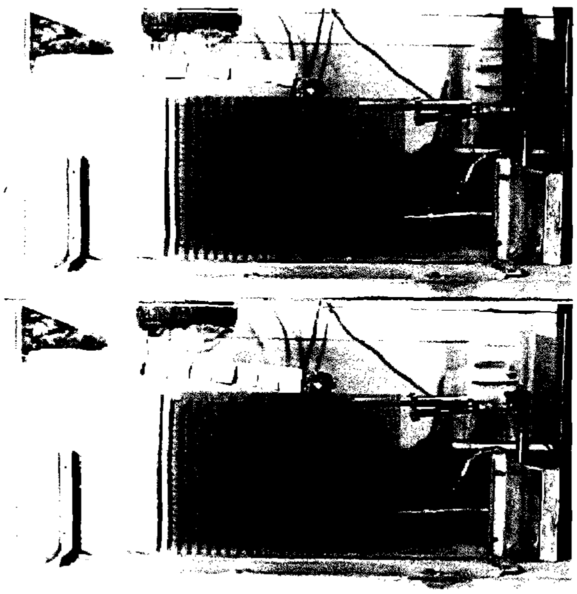

Fig. 5. The system with the surcharge applied close to the top of the wall: a) before and b) after the formation of the failure surface.

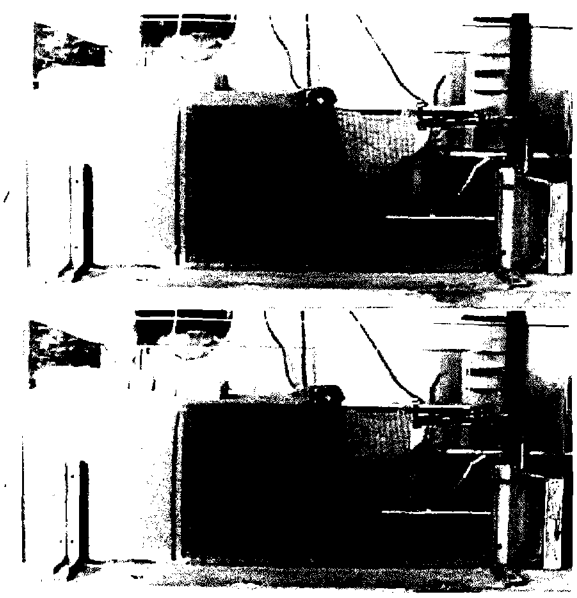

Fig. 6. The system with the surcharge placed at H/2 from the top of the wall: a) before and b) after the formation of the failure surface

acceleration was 0.26÷0.28g and the frequency of shaking was 5.88Hz and the failure surface was observed through the glass sides of the test box and the shaking table was stopped. The final permanent displacements were 7.7mm at the base and 16 mm at

the top of the wall. This result shows the rotational behaviour of the system due to the surcharge inertial force at the top of the wall. In fig 7b it is easy to observe the increase of the value of rotation until failure: the maximum is about 0.057rad, that is 3.27°. The rotational behaviour of the system is also emphasized by the inclination of the black sand markers introduced in the model.
In fig.7c the table acceleration (circle) and the wall base acceleration (cross) and displacement time-histories are plotted for the interval 22÷23,5sec. It is apparent that permanent displacements build up in the outward direction when the table is

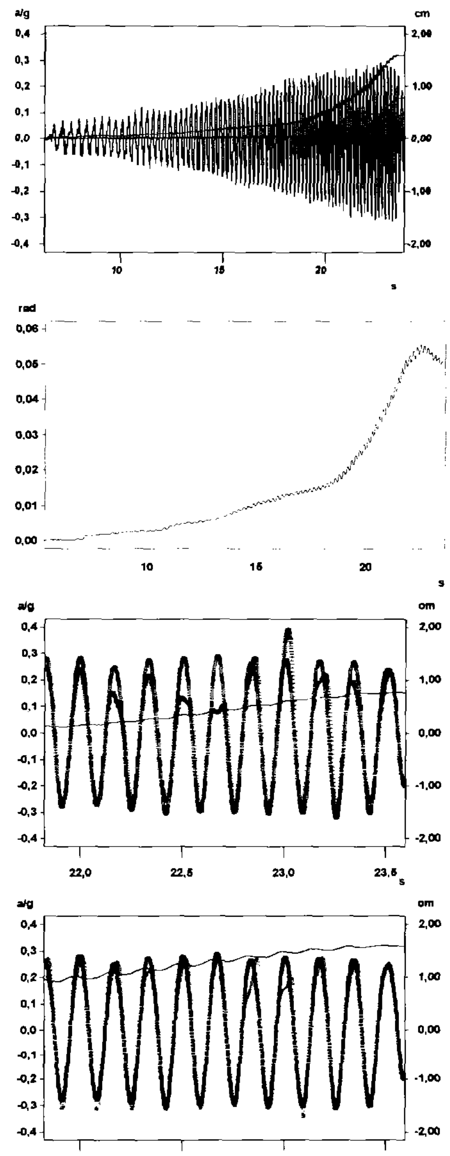

Fig. 7. a) input acceleration, wall top and base displacements; b) system rotation; c) input and wall base acceleration compared with wall base displacements; d) input and wall top acceleration compared with wall top displacements.

moving backward. After 22 seconds the wall base shows a reduction of the acceleration of about 30% of its maximum value; in this phase the peak shape is irregular and shows a sort of doubling. In fig.7d the table acceleration and the wall top both acceleration (triangle) and displacement time-histories are plotted for the same short interval. It is possible to observe that top displacements have large oscillations and increase when the wall and the table acceleration are negative, that is, directed backward. The analysis of the accelerometric data shows a similar behaviour to the case of base displacements. These results compare well with those obtained by Richards and Elms [1990] and by Richards et al.[1996] who demonstrated that prior to a threshold acceleration the wall acceleration is similar to the acceleration in the backfill. However, for input accelerations beyond such threshold, a cutoff acceleration for the wall is clearly evident, indicating that a relative acceleration has developed in the system.
The time-histories of accelerations and displacements of the soil-wall system with the surcharge placed at H/2 from the top of the wall are shown in Fig.8. In this case the accumulation of permanent displacement was more gradual: base displacements started to build up to about 12sec. When peak acceleration was approximately 0.13g. After 26 seconds the base displacements was still negligible, whilst the top displacement was 4mm. Again a tendency of the wall to rotate, due to the surcharge, was observed from the beginning of the test. At a frequency of about 5.9 Hz and table maximum acceleration around 0.3÷0.35g, the permanent displacement reached 11.4mm at the base and of 16.3 mm at the top of the wall, allowing to detect the failure surface. The final rotation after failure resulted about 0.028rad, that is 1.61° (fig.8b). Top (dashed line)and base displacements in the interval 33.5÷35 seconds are plotted in figure 8c together with input and top wall (cross)acceleration time histories.

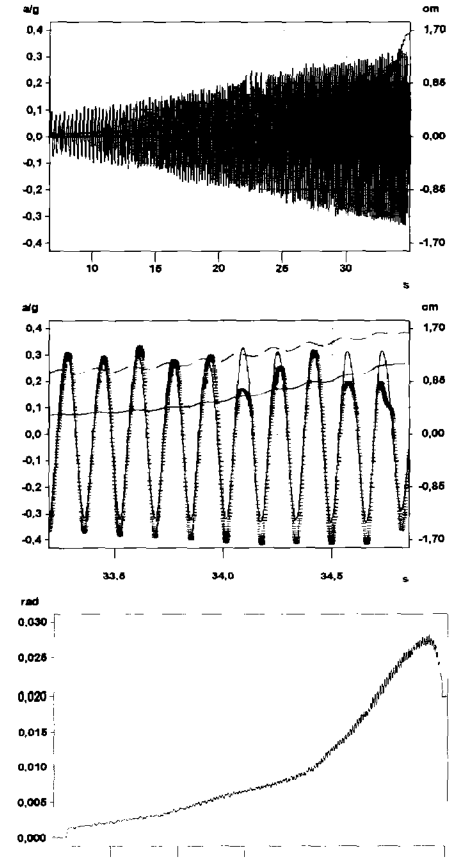

Fig. 8. a) input acceleration, wall top and base displacements; b) system rotation; c) input and wall top acceleration compared with wall base and top displacements.

The experimental results are consistent with those obtained in the test with λ=0 in which, however, the presence of the surcharge on the backfill influenced more markedly the rotational response of the soil-wall system. From the photographic detail of figure 9, the particular shape of the failure surface, presenting a small concavity, and the intersection between the failure surface and the third row of the surcharge blocks can be clearly observed.

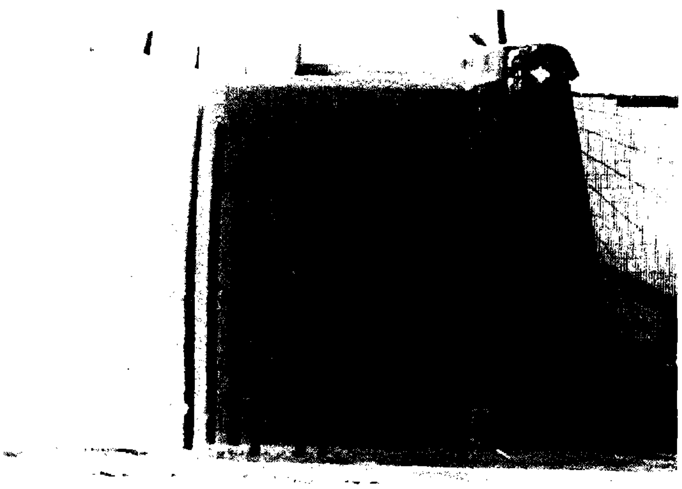

Fig.9. A detail of the failure.

Comparing the results of the two tests it is possible to conclude that both the considered systems, exhibit in a first stage, at low input acceleration amplitude, an elastic behaviour; in a second stage after the critical acceleration (i.e. the minimum acceleration at which initial sliding displacements occur) has been reached the system undergoes permanent displacements whose accumulation rate decreases with time if the input acceleration is kept constant. At any increase of the input acceleration larger displacements build up. The failure surface becomes visible only when the wall has moved outward of a few millimeters, though the critical equilibrium conditions have already been reached. This delay might be due to the slight friction effect between the sand and the glass sides of the test box. These results compare well with those obtained by Cascone et al.[1995] and Oldecop et al. [1996].
Critical accelerations and failure surface angles obtained through shaking table tests have been compared with those computed using Motta [1994] and Caltabiano et al [1999] methods of analysis. The results of such comparison are shown in Table 1 and schematically in Fig. 10.

Table 1. Comparison between experimental and theoretical models.

System	Caltabiano et al		Motta	Experimental	
	α_{cr}	a_{cr}	α_{cr}	α_{cr}	a_{cr}
Wall without surcharge	51.7°	0.103g	/	54°	0.1g
Wall with λ=0	58°	0.093g	64°	60°	0.1g
Wall with λ=0.5	53.5°	0.129g	60°	53°	0.13g

It may be noted that for the system with λ=0 experimental and theoretical angles are in reasonable agreement; for the system with λ=0.5 the method of Caltabiano et al.[1999] Approximates with better accuracy the experimental measure of the angle formed by the failure surface with the horizontal. The method of Caltabiano et al.[1999] provides also a very good evaluation of the critical acceleration. In Table I the experimental values of α_{cr} and a_{cr} are given for the case of wall without surcharge. Again a good agreement is found between experimental and theoretical results.

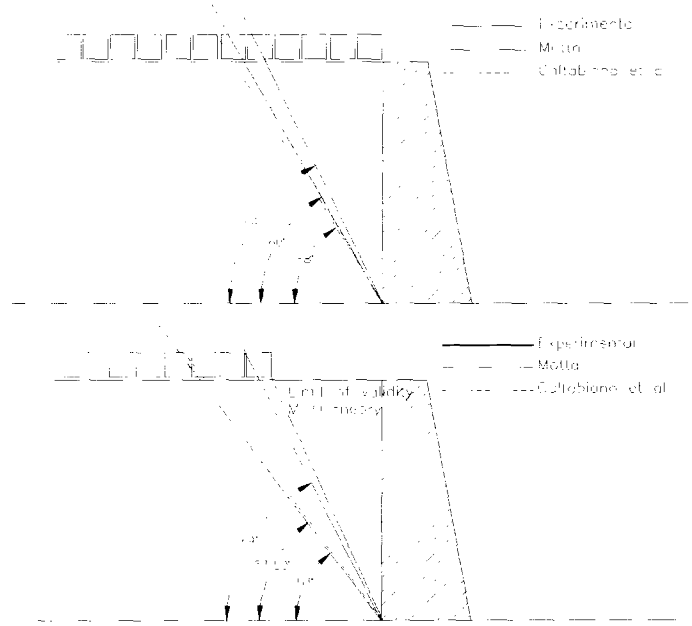

Fig.10. comparison between experimental and theoretical results. a) system with λ=0; b)system with λ=0.5.

CONCLUSIONS

Shaking table test have been carried out on rigid retaining walls presenting a uniformly distributed surcharge on the backfill. On the basis of the experimental results the following conclusion can be drawn:

- The soil-wall system in both cases of λ=0 and λ=0.5 behave similarly: they exhibit an initial elastic response until the input acceleration reaches the system critical acceleration and permanent displacements start to build up;
- The surcharge affects the response forcing the system to experience some rotation and loweing the system critical acceleration;
- For the system with λ=0.5 it has been shown that the failure surface intersects the surcharge and therefore this case cannot be consistently studied applying the M-O theory;
- The system with distanced surcharge (λ=0.5) has a larger resistance than the system with close surcharge but haviong a small angle α_{cr} involves as well larger soil masses; in other terms a distanced surcharge may attract the failure surface producing damages to larger distances from the wall;
- The experimental results obtained in the tests are in good agreement with the theoretical results obtainable by means of the equations proposed by Caltabiano et al.[1999].

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