SHAKING TABLE TEST ON SMALL PROTOTYPE OF SOIL RETAINING WALL
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FINAL REPORT
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ABSTRACT
A shaking table test was carried out on a 65cm high gravity retaining wall, arranged in a 3m long tank with glass sides. The wall was put on a 10cm thick layer of San Juan River clean sand and backfilled with the same material. The whole was mounted on the IDIA shaking table. Accelerations of shaking table and wall's toe, horizontal and vertical relative displacement between tank and wall's toe and head and earth pressure magnitude and distribution were measured during the test.

The observed wall dynamic response was composed by vertical and horizontal displacements and rocking. The rocking response played an important role in dissipating the earthquake energy input and preventing from large residual displacement accumulation.

The registered wall behaviour of the wall treated as a small prototype is compared with predicted behaviour by some usual analytical approaches: Mononobe-Okabe expression for earth pressure, Richard-Elms method for residual displacement computing and Zarrabi improved model.
SHAKING TABLE TEST ON SMALL PROTOTYPE OF SOIL RETAINING WALL

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1 - INTRODUCTION

The stability analysis of earth retaining walls under earthquake loading requires the determination of earth pressures. The amount of earth pressure depends on the interaction between wall, backfill and foundation.

The traditional way for computing the earth thrust under static loads is the sliding wedge method proposed by Coulomb for frictional materials. Okabe (1924) and Mononobe and Matsuo (1929) extended the Coulomb’s method to take account of the earthquake effect by means of introducing equivalent static forces in the sliding wedge equilibrium analysis (figure 1). The Mononobe-Okabe analytical expressions allow to determine the active and passive thrust as a function of geometry, strength parameters and horizontal and vertical seismic coefficients. Theoretical investigations (Wood, 1975) showed that the M-O expressions give a good estimation of the backfill forces acting on the wall, provided that the structure (or its foundation) is able to deform enough to reach the active condition.

Under static loads, the backfill pressure resultant may be assumed to act at a third of the height of the wall "H" from its bottom. There is no agreement on which is the height "h" of the earthquake pressure resultant. Theoretical results obtained by Wood (1975) showed that the dynamic pressure resultant acts approximately at the half of the wall height. Seed and Whitman suggested that the static component acts at H/3 from the bottom of the wall and that the additional dynamic effect should be taken as acting at 0.6H. The Argentine Code NAA-80 requests to take the static component acting at H/3 and the dynamic component at 2/3 H. Richard and Elms (1979) proposed -for practical purposes- to assume a uniformly distributed pressure (h = H/2).

The use of the M-O expression involves the selection of the seismic coefficient that
will represent the earthquake effect on the structure. Once the total backfill thrust is computed, the stability of the wall has to be evaluated against sliding and overturning. The wall inertia forces are usually computed with the same seismic coefficient. The use of a seismic coefficient value equal to the maximum peak acceleration of the expected earthquake ensures that the wall will not slip at all during the movement. However, for most cases, this leads to an excessively conservative and therefore uneconomical design. Most retaining structures allow the occurrence of significant outward displacements which do not mean a failure. Admissible wall displacements and tilting depend on the function of the structure and its importance.

An alternative design criteria is to adopt a reduced seismic coefficient to consider the wall displacement capability in order to get a more economical design. Empirical formulas have been proposed to relate the design seismic coefficient with the maximum peak acceleration (Noda, 1976).

Considering the above mentioned shortcomings of the traditional approach, Richard and Elms (1979) proposed a more rational design criteria consisting in the limitation of the permanent outward displacement of the wall caused by the earthquake to an admissible value. Richard and Elms developed an extension of the Newmark sliding block method to compute the wall displacements. Original Newmark's method was used to compute permanent displacements of slopes (figure 2). The R-E development consisted in applying this method to retaining walls assuming that the earth thrust at failure can be computed with the M-O formula.

The basic assumptions in this approach are: 1) materials behave rigid-plastic; 2) only the horizontal forces are taken into account in the equilibrium equation and only the horizontal translation movements of the wall are possible; 3) the total earth thrust can be computed with the M-O expression.

A shaking table test on a 25 cm high retaining wall performed by Sim and Berrill (1979) showed that the R-E method always overestimate the permanent displacement. The reason for this is that this approach neglects the vertical inertia forces acting on the soil.
wedge due to its down-slip motion following the wall motion. Zarrabi (1979) improved the R-E model in its cinematic aspect, taking into account the wall and the backfill displacement compatibility and the vertical inertia forces acting on the soil slip-wedge.

The limitations of the described design methods are: 1) the dynamic amplification effects are neglected since the constitutive laws used are rigid-plastic. This means that the excitation frequency has no effect on the structure response. 2) The only wall movement considered is horizontal translation, neglecting the rocking response. 3) The possibility of a saturated backfill and liquefaction development is not considered. Other researchers have developed partially improved models in one or more of these aspects. Prakash (1981) proposed a model for computing displacements in horizontal translation with elastic-plastic constitutive laws for backfill material and wall-soil interfaces. This allows the consideration of the dynamic amplification effects due to the backfill and foundation deformability. He also proposed another similar model for computing wall displacements in rocking. Nadim and Whitman (1984) developed a Newmark-like model including coupled horizontal displacement and rocking wall response. However, these methods are not yet applicable to practical cases since they need the determination of one or more model parameters and there is no analytical or experimental specified way to obtain these parameters.

This paper presents a shaking table test carried on a 65 cm high gravity retaining wall. The main research program aims were: 1) To observe and register the structure behaviour during the motion in order to get a better understanding of the failure mechanism and the factors that influence the structure response. 2) To check the validity of the assumptions of the most commonly used stability analysis methods and mathematical models and the degree of accuracy of their results.

2 - TEST SETUP

Test was conducted on a 65cm high and 82cm wide gravity wall (figure 4), arranged in a 3 meters long glass-sided test tank (figure 3). The wall and the tank were built with steel sections and sheet. The wall mass is variable by adding weights of 30 Kg. each (figure 5).
The friction on the wall base-foundation and the wall vertical face-backfill interfaces was increased by bonding clean sand with epoxy resin. The friction angle on the wall base was measured by jacking the wall resting on the foundation material and reading the force needed to cause the slip along the base (figure 6). Measured values ranged between 28° and 33°.

The bending moment caused by the earth thrust acting on the vertical wall face was measured at three different heights with strain gage bridges (figure 8). The record of these bending moments, allow the calculation of the soil pressures acting on the wall at any time during the test.

The wall alignment in relation to the test tank was provided by four ball bearings fixed at the wall base corners and resting against the tank glass sides (figure 7).

3 - TEST PREPARATION

The test tank was mounted on the IDIA shaking table. This is a servo-hydraulic actuated device, which can accelerate a 10-ton weight to 0.5g in a frequency range of 10 to 30 Hz. Motion is controlled by computer (figure 11).

San Juan clean river sand was used as foundation and backfill material (Unified Classification: SP). It was placed in 5 cm thick hand compacted layers (figure 9) at 1.81 ton/m³ dry density which corresponds to 60% ASTM Relative Density. The internal friction angle of this material range between 34° and 40°.

The wall was founded on a 10cm thick sand layer, the 8 weights necessary to reach the design mass were fixed and then the backfill was executed. The total weight of the wall was 289.6 Kg. Seal between the wall and the tank glass sides was provided by a 5mm thick rubber band impregnated with oil in order to keep friction to a minimum.

The test instrumentation comprises:
- Three variable inductance position transducers arranged so as to get a complete register
of the wall movements during the test. (figure 10).
- Two strain gage acceleration transducers measuring the tank (ground) and the wall base acceleration.
- Three full strain gage bridges for earth thrust indirect measurement (figure 8).

The transducer analogue signals were amplified, converted to digital information and recorded in PC. Recording sample rate was set at 200 data/second. The test was also filmed with a video camera in order to obtain a register of the whole behaviour of the structure. Figure 12 shows the test set-up ready to start the test.

4 - TEST EXECUTION

The analysis of the tested wall treated as a small prototype, was carried out by means of the Coulomb method in order to evaluate the pre-test stability condition. The safety factor against sliding under static loads is 2.60 and against tilting is 4.15. The structure yield acceleration -the ground acceleration which initiates the outward slip of the wall- was computed by trials with the M-O expression, with the aim of predetermine the level of acceleration to be used in the test motion. Due to the uncertainties in the soil and wall-soil interfaces friction properties, the yield acceleration possible values ranged from 0.15 g to 0.34 g.

The frequency of excitation motion was fixed at 10 Hertz and its duration in 16 seconds. The test started with 0.25 g of acceleration amplitude, having no visible effect on the wall or the backfill. In successive motions, the amplitude was increased in 0.1g steps. Cracks appeared on the backfill top surface (figure 13) as the peak acceleration was set at 0.5g but no permanent displacement of the wall was observed until the last motion which caused the failure took place. This happened with a 1g amplitude motion.

The final position of the wall after failure is showed on figure 15. The wall's head displaced 10cm outwards from its initial position and the toe 6.8cm. The permanent outward tilt angle was 2.8°. Figure 14 shows three displacement records and two acceleration records.
5 - OVERALL BEHAVIOR

Figure 15 shows photographs of both sides of the test set-up after failure, with the thirteen fill layers numbered from bottom to top. Failure surfaces were identified and marked on the tank plate-glass sides and. A vertical line indicates the initial position of the wall’s vertical face. The observation of the relative position of the fill layers, cut by failure surfaces allowed to determine the failure mechanism as shown in figure 16.

Processing the recorded test data yielded the following results:
- Wall base horizontal displacement history (figure 17a).
- Total earth thrust history (figure 17b).
- The complete wall motion representation in the PC monitor (figure 18).

These results and the videotape showed that the structure behaviour has two well-defined stages with the following features:

a) First stage:
   - Duration: 10.5 seconds.
   - Low rate of permanent displacement accumulation (1.7 mm/sec.)
   - Large rocking response.
   - No evident failure surface.
   - Mean "static" thrust increase.

b) Second stage:
   - Duration: 5.5 seconds.
   - High rate of permanent displacement accumulation (10 mm/sec).
   - Large translation response.
   - Well defined failure surfaces.
   - Mean "static " thrust remains constant or decreases.
   - Registered wall base peak positive (backfill wards) acceleration is somewhat smaller in coincidence with the maximum permanent accumulation rate.

The videotape also showed that the two identified failure surfaces (figure 16) appeared in the second stage. The first failure plane forms a 63° angle with the horizontal
plane. A second failure plane somewhat less steep than the first (52°) appeared in the latest part of the motion. Both failure planes have an unexpectedly steep angle, in relation to the predictions of the Coulomb stability analysis.

A second Coulomb's stability analysis was carried out trying to get a better understanding of the observed failure mechanism. Since the failure plane inclination depends mainly on the friction properties of the vertical wall face, trial stability analyses were carried out varying the wall face friction angle $\delta$. Data and results presented in table 1 make the static failure plane to fit the 63° plane and the dynamic failure plane with the 52° plane. As shown in table 1 the $\delta$ value consistent with the observed failure plane inclination is markedly small.

The wall motion representation in a PC monitor allowed observing the response differences between the two stages. During the first stage the response is mainly in rocking and the slow outward displacement accumulation is caused by small slips which occurred as the wall started rocking at each motion cycle (figure 19). The second stage response is mainly in translation and large horizontal permanent displacements accumulate at each motion cycle while rocking motion amplitude keeps small.

6 - MATHEMATICAL MODELLING

Mathematical modelling of the tested structure treated as a small prototype was carried out in order to compare the predicted with the observed behaviour.

Table 1 shows M-O stability analysis data and its results. Figure 17 compares the computed M-O dynamic active thrust value -the earth thrust, which acts during the wall slip with the measured thrust history. Marked minimum peaks are the dynamic active thrust values measured during each wall slip. At the beginning of the test, measured values are smaller than the computed ones. Dynamic active thrust peaks become closer to the predicted values as the mean static thrust -this is the earth thrust which would remain if the test would be stopped at any time- reaches the wall base shear strength.
To predict permanent displacements the R-E and Zarrabi models were applied, assuming that the failure will occur in agreement with the M-O analysis results, this is a yield acceleration of 0.25 g and a 52° inclined failure plane. Results and measured horizontal displacement of the wall are plotted together in figure 20. Both models overestimate the final permanent displacement.

The rate of permanent displacement accumulation is similar in the Zarrabi model prediction and in the second stage of the recorded wall behaviour.

**7 - CONCLUSIONS**

1) Rocking plays an important role in the in a first stage of the wall dynamic response. The observed effect of rocking is the reduction of the permanent displacement accumulation rate by reducing the inertia forces.

2) This is, for the tested structure, the main reason for the overestimation of the mathematical models predicted displacements.

3) The R-E model seems to always overestimate the displacements since it neglects vertical slip wedge inertia forces. Zarrabi model overcomes this problem and gives a good approach of the permanent displacement when rocking response is not possible (during the second stage of the test).

4) The mean static earth thrust increases during the motion, so that the final static thrust is greater than the initial value. Whitman (1993) reported a similar behaviour from finite element analysis and centrifuge tests results.

5) The structure changes from first-stage behaviour to second-stage behaviour as the mean static thrust approaches the wall base shear strength. A possible explanation for this is that the larger backfill pressures hinder rocking response.
REFERENCES:
7 - Sim, L.; Berrill, J.B.; 1979 "Shaking Table Test on a Model Retaining Wall". Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2.
\[ E_{an} = \frac{\gamma H^2 (1-kv) \cos^2(\theta-\beta)}{2 \cos \theta \cos^2 \beta \cos(\delta+\theta)} \left[ 1 + \sqrt{\frac{\sin(\epsilon+\delta)}{\cos(\delta+\theta) \cos(\beta-\theta)}} \right]^2 \]

\[ E_{pe} = \frac{\gamma H^2 (1-kv) \cos^2(\theta-\beta)}{2 \cos \theta \cos^2 \beta \cos(\delta+\theta)} \left[ 1 - \sqrt{\frac{\sin(\epsilon+\delta)}{\cos(\delta+\theta) \cos(\beta+\theta)}} \right]^2 \]

\( \gamma = \) Soil unit weight.
\( H = \) Wall height.
\( \epsilon = \) Backfill friction angle.
\( \delta = \) Horizontal seismic coefficient.
\( kv = \) Vertical seismic coefficient.
\( W = \) Sliding wedge weight.
\( \theta = \tan^{-1} \left( \frac{kh}{1-kv} \right) \)

FIGURE 1. Momonobe-Okabe active and passive thrust formulas
FIGURE 3. Test tank mounted on the IDIA shaking table.

FIGURE 4. Gravity retaining wall small prototype.
1, 2, 3: Bending moment measuring sections.

FIGURE 5. Retaining wall structure and dimensions.
FIGURE 6. Wall base - foundation friction angle measurement.

FIGURE 7. Alignment ball bearings.
FIGURE 7. Tank filling with San Juan River sand.
FIGURE 10. Relative displacement transducers arrangement.

FIGURE 11. Control and data acquisition system.
FIGURE 12. Test setup ready to start the test.
FIGURE 13. Cracks on backfill top surface appeared as the acceleration amplitude was set at 0.5g.
FIGURE 15. Failure surfaces identification.
FIGURE 16. Failure mechanism.
FIGURE 17. a) Measured wall base horizontal displacement history. 
b) Measured horizontal backfill thrust history.

\[ E_{as} = M-0 \text{ active thrust under static loads.} \]
\[ E_{ae} = M-0 \text{ active thrust under static plus earthquake loads.} \]
\[ S_b = \text{Wall base - foundation frictional shear strength.} \]
\[ g_{mx} = \text{Ig acceleration x wall mass.} \]

Black line: Measured backfill thrust history.
White line: Mean "static" backfill thrust.
\( \triangledown \): Measured active thrust peak.
FIGURE 18. Successive wall positions during the test plotted every 0.8 seconds.
Figure 19. Observed wall response types.
FIGURE 20. Comparison between wall gravity center recorded displacement and wall horizontal displacement predicted by mathematical models.
TABLE 1

STABILITY ANALYSIS WITH THE COULOMB METHOD

DATA

\[ H = \text{wall height} = 65 \text{ cm.} \]
\[ \phi = \text{backfill material internal friction angle} = 38^\circ \]
\[ \delta = \text{wall vertical face - backfill friction angle} = 3.8^\circ \]
\[ \beta = \text{wall base - foundation friction angle} = 33^\circ \]

Backfill material dry density = 0.0018 Kg/cm³
Total wall weight = 289.6 Kg

STABILITY ANALYSIS UNDER GRAVITY LOADS

Safety Factor = 2.80
Active thrust = 72.52 Kg
\[ \alpha = \text{failure plane angle} = 63.08^\circ \]

STABILITY ANALYSIS UNDER EARTHQUAKE LOADS

Trial analysis were carried out in order to compute the yield acceleration.
Safety Factor = 1.00
Active Thrust = 118.07 Kg
Yield acceleration = 0.24 g
\[ \rho = \text{failure plane angle} = 52.37^\circ \]