Dynamic behaviour of reinforced earth walls
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Abstract

This work analyses the behaviour of some typologies of reinforced earth retaining structures during a seismic event comparing calculation results with those obtained for a traditional concrete wall.

Four different typologies of walls are considered:
- concrete cantilever wall;
- steel strips reinforced earth wall with concrete face panel;
- geosynthetic sheets reinforced earth wall with concrete face panel;
- metallic grids earth reinforced wall with metallic gabbions filled with stones as face panel.

Analytical results are relevant to some typical ground stratifications and to different values of the wall height for a II category seismic Italian region. It’s necessary to specify that for the seismic design of reinforced earth structures the usual procedures founded on pseudostatic methods can result very conservative and so it is preferable to use alternative solutions.

In general, it is known that soil dynamic analysis is more significative as it is founded on the total induced seismic displacement but other methods, like numerical methods may be important for the reinforced earth walls to calculate strains and stresses in the structural components. In this work, pseudostatic method and finite elements method are considered in order to compare their hypotheses and applicability ranges.

1 Introduction

It is known that the engineering structures which best resist to the seismic actions are those that deform while dissipating energy.

These structures must be constructed of materials resisting to shear and tension: in addition they must be flexible, simple and regular in shape with
individual parts which must be jointed to form continuous systems in order to redistribute the earthquake induced stresses. Reinforced earth walls possess all these properties, thus explaining the high degree of development of reinforced earth technology in those regions which are exposed to earthquakes [1]. It must be said that there are only a few studies on the seismic performance of retaining structures and the codes themselves provide at best only relatively simple guidelines for design. On the contrary, because of the interest raised by engineers and owners, very soon after his introduction, the behaviour of reinforced earth in seismic areas has been the subject of numerous experimental studies by means of scale models and full scale structures.

An extensive review with a critical synthesis of all the previous studies was carried out by H. B. Seed (1970) at the request of the Reinforced Earth Group. He drew simple design conclusions, still valid, about the excess dynamic tensile load in the reinforcements and about the acceleration threshold under which no residual deformation occur. These and other concepts will be recalled in the foregoing.

Dynamic calculations by finite element analysis were moreover conducted under the request of the R.E.G., the obtained results closely agreed to the ones calculated by Seed, thus confirming his working hypotheses. It must be outlined that these numerical results are relevant to linear non extendible reinforcements and should not be generalised for other types of reinforcements like grids and non metallic extensible reinforcements, which are, on the contrary, the most technologically advanced.

In the present work, pseudostatic method results and finite elements results are considered in order to compare their hypotheses and applicability ranges for a traditional concrete cantilever wall and for three typologies of advanced reinforced walls. Earth pressure acting on the facing panels are calculated using firstly the traditional pseudostatic method and secondly the finite elements method which provides also shear stresses and bending moments on the facing panels and tensile stresses into the reinforcements. Elastic deformations of the facing panel are then calculated by means of the equation of the elastic line, interpolating the output shear values relevant to each element of the wall mesh.

The applied numerical code is the FLUSH [2] program whose characteristics, relevant to an equivalent linear elastic soil and to a bidimensional deforming model, are well known. As concerning the general behaviour of a reinforced earth mass, reference may be made to the work of H. Vidal [3], while as concerning the seismic behaviour of reinforced earth walls, it may be useful to recall some particular concepts.

Under static conditions the external stability, that is to say the safety to sliding, rocking, collapsing may be calculated in the same way as for a gravity wall; for the internal stability it's necessary to verify, furtherly, that the wall, under the actions of its own weight and of the backfill pressure does not collapse because the reinforcement traction limit value is reached or because the reinforcements themselves are pulled out of the ground, as their anchorage is insufficient. Under dynamic conditions external stability and internal stability must be still guaranteed: it seems of course that the great mass and the great deformability of the reinforced structures are favourable with respect to the
seismic actions [4], on the other side the inertial force, $F_1$, depending on the wall mass and the dynamic active pressure behind the wall, $F_{AE}$, are not so easily evaluable, as it happens for a traditional wall, because the “backfill” is at the same moment inside the wall and behind the wall.

From this point of view, as H. B. Seed himself suggested, a finite element analysis should be recommended especially for particular reinforcements when numerical calculations or experimental results are not so extensive to support the current design assumptions.

2 The traditional approach to the dynamic behaviour of reinforced structures

2.1 Pseudostatic method

The method proposed by Seed [5] for the Terre Armée Internationale S.A. Group substantially moves from Mononobe-Okabe limit analysis method. The forces which must be considered in the external stability are: the dynamic additional force that must be summed to the static one $(a/g \cdot W_s)$ and the inertial force $F_1$ depending on the wall mass. As the two dynamic forces don’t reach their maximum values at the same time [6], it seems reasonable to reduce them by an opportune coefficient. The external forces have been represented in Fig. 1:

\[
0.2(1+5/3a/g)\gamma \frac{Ha}{g} \quad 0.6\gamma \frac{Ha}{g}
\]

\[k_a \gamma H \quad H/3 \quad F_1 \quad H/2 \quad F_{AE} \quad 0.6H \]

Figure 1: Representation of the external static and dynamic forces

In this figure the backfill pressure on the wall is equal to $P = 1/2 \gamma H^2 k_a$, where $\gamma$ is the unit weight of the fill, $K_a = \tan^2 (45^\circ - \varphi/2)$ is the coefficient of active earth pressure and $H$ is the wall height; the inertial force is equal to $F_1 = 0.15 \gamma H^2 \frac{a}{g} (1+5/3 \frac{a}{g})$, in which $a/g$ is the ratio between the maximum value of the acceleration to the ground during the seismic action and the gravity acceleration, finally, the dynamic force is assumed to be $F_{AE} = 0.375 \gamma H^2 \frac{a}{g}$.

The forces which must be considered in the internal stability analysis are evaluated by the experimental studies that Richardson [7] conducted on the vibrating table and by the semiempirical results obtained by Seed: the traction values in the reinforcement levels are then established thus allowing the pull out safety verification.
2.2 Dynamic displacements analysis

Reference may be made to the well known method of Newmark [8] which is based on the evaluation of the total wall displacement accumulated during the earthquake shock. In particular it is necessary to define an acceleration threshold under which no permanent deformation exists. Returning back to the distributions of forces $P$, $F_{AE}$ and $F_1$ represented in Figure 1 and isolating an element whose thickness is $\Delta h$ and whose width is $0.7 \, H$ as the most applied design criterions suggest, it is obtained that the wall mass: $0.7 \, \gamma \, H \, \Delta h$ is subjected to:

- a static force: $1/2 \, 0.30 \, \gamma \, \Delta h^2$,
- a dynamic force: $0.6 \, \gamma \, H \, \Delta h \, a/g - 1/2 \, 0.45 \, \gamma \, \Delta h^2 \, a/g$,
- an inertial force: $0.2 \, (1 + 5/3 \, a/g) \, a/g \, \gamma \, H \, \Delta h$.

The most unfavourable condition leads to the following horizontal equilibrium expression:

$$0.15 \, \gamma \, \Delta h^2 + \gamma \, H \, \Delta h \, a/g (0.6 \, H - 0.225 \, \Delta h) + (0.2 + 1/3 \, a/g) \, a/g \, \gamma \, H \, \Delta h = 0.7 \, \gamma \, H \, \Delta h \, \tan \varphi$$

assuming $0.7 \, \tan \varphi \equiv 0.5$, that is to say $\varphi = 35^\circ$ it is obtained:

$$(a/g)^2 + a/g (2.4 - 0.675 \, \frac{\Delta h}{H}) + (0.45 \, \frac{\Delta h}{H} - 1.5) = 0$$

The limit value of $a/g$ ranges from 0.515 for $h = 0$ to 0.50 for $h = H/2$.

It may be concluded that no permanent deformations will occur into the reinforced earth wall of width $\geq 0.7 \, H$ until the ratio $a/g$ does not exceed 0.5.

3 Finite elements analysis for traditional walls and reinforced earth walls

From a geotechnical point of view a smarting finite elements analysis should be able to describe the fill dishomogeneity and the non linear soil characteristics that is to say a tangent modulus $G(\gamma)$ and a damping modulus $D(\gamma)$ depending on the achieved current value of the deformation $\gamma$. It must be outlined that the analysed system constituted by the wall and the surrounding soil is a finite portion of the whole semispace, so necessarily it must be assigned a fictitious contour. This contour, in both cases of propagation waves along an inclined direction and in the case of propagation along a vertical direction, reflects the seismic waves reaching it, so proceeding with the analysis in the time domain the wave content of the modelled zone changes with respect to the real one because of the continual edges reflections. The well known “transmitting boundaries” absorb completely the energy of the incidental waves in order to simulate the finite portion of the semispace as an infinite space.
The transmitting boundaries have been introduced for the first time by Lysmer and Waas [9] to study the propagation of the shearing waves in an horizontally stratified deposit which rests on a rigid base. Analysing the wave content of the fill deposit and the wave induced forces, it is possible to express the forces $Q$ that the fill applies to the edges in the form $Q = K_b u$, where $u$ are the displacements along the edge and $K_b$ is the matrix of rigidity at the edges. In this way, the problem of the simulation of an infinite medium may be solved with a simple correction, by means of the $K_b$ term, in the rigidity matrix of the system [10].

The above mentioned requisites, implemented in the FLUSH program used in the presented analysis, give a still valid instrument for the solution of interaction problems among which those related to the reinforced walls surrounding soil system.

### 4 Stresses and displacements resulting from the finite elements analysis

Four different typologies of walls are analysed:

- **Concrete cantilever wall.**

- **Steel strips reinforced earth wall with concrete face panel.**
  In this typology of wall the reinforcing strips are almost exclusively made by metal, usually galvanised steel, 45 mm wide and 5 mm thick placed horizontally at a constant vertical spacing $h = 0.75$ m, connected with the face panel by means of four steel lugs cast in situ during manufacture. The concrete face panel is constituted of facing units which are cruciform shaped in front elevation, standard units weight approximately 1 tonne, and are 1.5 m by 1.5 m wide with a total thickness of 140 mm.

- **Geosynthetic sheets reinforced earth wall with concrete face panel.**
  Non metallic reinforcements are almost exclusively made of one, or more combined together polymers. One considerable advantage of polymers is that they do not suffer for corrosion but they are less strong and more extensible than their metallic counterparts. The geosynthetic sheets used in the present work are fabrics, in particular polyester sheets with polyethylene sheath, their thickness is 2.4 mm and their nominated breaking load is 400 kN/m. These sheets are connected with the face panel by means of appropriate lugs. In this case the facing units are rectangular shaped in front elevation, with approximately the same dimensions of the cruciform panels.

- **Metallic grids earth reinforced wall with stones filled into metallic gabbions as face panel.**
  In this last solution the reinforcements are metallic grids which mesh is hexagonal shaped and constituted of a galvanised steel wire. The typical mesh used is 8 mm wide and 10 mm long, the composition of these meshes constitutes a grid that is the reinforcement of the wall. The advantage of the metallic grids is that they guarantee a continuous reinforce on the horizontal plan in which they are placed. The face panel is constituted of 1m by 1 m metallic gabbions filled with stones; finally, it is necessary to underline that the grids are extended to the face panel.
In particular, the two first typologies of walls above listed are examined by pseudostatic methods and afterward by a finite elements simulation; the third and the fourth typologies of walls are examined only by means of a finite elements grid. The investigated wall heights are 6 m and 10.5 m, the height 6 m has been analysed because it is valid for all types of wall while the height 10.5 m is valid for the reinforced earth structures only. With respect to the foundation soil two general situations are considered: the first consists of an altered clayey layer above a bank of an overconsolidated clay, the second consists of a bank of a medium dense sand. In both cases the backfill properties are dictated by the Terre Armée Internationale Research Group. The physical mechanical characteristics of the considered soils are summarised in the following Table I:

<table>
<thead>
<tr>
<th>Backfill Type</th>
<th>( \gamma ) (kN/m(^3))</th>
<th>( \Phi ) ((^\circ))</th>
<th>( c' ) (N/cm(^2))</th>
<th>( c_u ) (N/cm(^2))</th>
<th>( N_{SPT} )</th>
<th>( G_0 ) (N/cm(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>altered clay</td>
<td>20.70</td>
<td>32</td>
<td>3</td>
<td>20</td>
<td>-</td>
<td>60000</td>
</tr>
<tr>
<td>o.c. clay</td>
<td>20.60</td>
<td>35</td>
<td>4</td>
<td>40</td>
<td>-</td>
<td>120000</td>
</tr>
<tr>
<td>sand</td>
<td>20.00</td>
<td>35</td>
<td>0</td>
<td>-</td>
<td>47</td>
<td>30000</td>
</tr>
<tr>
<td>backfill</td>
<td>18.00</td>
<td>36</td>
<td>0</td>
<td>-</td>
<td>50</td>
<td>25700</td>
</tr>
</tbody>
</table>

Table I: Physical and mechanical characteristics of the soils

It’s interesting to note that different values of the layers thickness don’t lead to significative different results. The seismic input to the bedrock considered in the program consists of a “time-history” of horizontal accelerations given in \( N \) points at constant intervals of time \( \Delta t \); in the examined cases we have considered the eight artificial accelerograms obtained from the response spectrum of the in force Italian Code (D. M. 24/01/1986). The considered eight accelerograms have a duration of 20.00 seconds with a time step of 0.01 seconds and each accelerogram has a different frequency content.

The finite elements analysis outputs are: the shearing stresses and the bending moments on the face panel of the wall and the traction values in the reinforcements; the displacements are calculated by means of the equation of the elastic line starting from the function interpolating the output values of the shearing stresses. The envelope of the shearing stresses relevant to the eight accelerograms considered in the analysis is represented in Fig. 2; the envelope of the elastic displacements during the earthquake are represented in Fig. 3.

It’s interesting to observe that for a traditional wall, the value of the dynamic force obtained by the finite elements analysis (\( F_{AE} = 200 \text{ kN/m} \)) results greater than that calculated by the equivalent pseudostatic method for the assumed value \( a/g = 0.07 \) (\( F_{AE} = 123.6 \text{ kN/m} \)) because in this last method a condition of incipient rupture with a large strain regime is assumed. Moreover, the negative values which result in the shearing actions may be regarded as decreasing values of the reinforced backfill pressures on the panel due to the changing seismic input and thus resulting in a more realistic evaluation of the safety wall conditions. For the same reason, we must note that the arm of the
dynamic force as it may be calculated from Fig 2 is equal to 1/3 of the height of the wall thus inferior to the value of 2/3 of the height of the wall suggested from the Italian Code, and still inferior to the value of 1/2 of the height of the wall proposed by the Eurocode 8.

For a steel strips reinforced wall the values of the shearing stresses and then of the forces on the face panel result smaller than those obtained for the concrete cantilever wall, as expected, but in this case finite elements analysis gives a value of the dynamic force equal to 23.2 kN/m while the pseudostatic method gives a value equal to 18.9 kN/m, then showing a better accord with pseudostatic results which may be due to the semiempirical corrections applied by Seed.

For the geosynthetic sheets reinforced wall the values of the shearing stresses and of the dynamic force ($F_{AI} = 33.8$ kN/m) result still smaller than those obtained for the concrete cantilever wall but greater than those obtained for a steel strips reinforced wall because of the best deformability of the geosynthetic.

Finally, for the metallic grids reinforced wall the values of the shearing stresses and the value of the dynamic force are smallest in absolute. In fact, as the gabbions facing is more heavy than the correspondent concrete facing panel (a gabbion has a thickness of 1 m which must be compared with the thickness of 0.14 m of the concrete facing panel). The more consistent mass produces a better interaction with the surrounding soil when the seismic action arrives. On the other hand, the stress in the earth and its deformation are absorbed by the reinforcements.

The maximum displacement resulting from the analysis is obtained for a concrete cantilever wall of 6 m height that rests on sand and it is equal to 2.8 cm. In the same case we obtain, for example, for a wall reinforced with metallic strips a maximum displacement of 0.09 cm. It is worth while noting that the displacements relative to the highest reinforced structures are smaller than those relative to the lowest (Fig. 3 ). The expected increase of deformation is balanced by the greatest increase of the reinforcement lengths and by a greatest mass of the reinforce structures according to the current design criterions of this kind of walls.

5 Conclusions

It may be concluded, on the basis of the above presented results, that all kinds of reinforced structures should exhibit a very good behaviour during a seismic event. Previous works on this subject specified without quantifying that reinforced wall displacements and stresses are significantly less than those relevant to a traditional wall. It's worth noting that the knowledge of shear stresses, bending moments, elastic deformations on the facing panels, may be very useful from a structural point of view. Further developments of the present work must consist of a parametric study where the height of the reinforced wall, the type of foundation soil, the strate thicknesses, the seismic motions, the water level position, the bedrock depth, the reinforcement typology and its phisical mechanical properties should be the selected design parameters.
Figure 2: An example of the shearing stresses envelope on the facing panels of the different wall typologies for overconsolidated clay and for the eight considered accelerograms.
Figure 3: Displacements envelope of the facing panels for the different wall typologies
References


