# Numerical analysis of a full scale earth reinforced wall: static and seismic behavior

Piera Paola Capilleri,[1] Maria Rossella Massimino,[2] Ernesto Motta,[3] Erminia Raciti,[4] Maria Todaro,[5] Francesco Ferraiolo,[6] Anna Bortolussi,[7] Jaime da Silva Duran[8]

## *Summary*

Nowadays several reinforced earth typologies are available. One of the most interesting is the vertical earth walls reinforced with polymeric geostrips with concrete facing panels. Geostrips tensile stiffness is much larger than that of typical geosynthetic sheets. Uncertainties may arise about the real geostrips behaviour: if they should be considered as inextensible or extensible. The aim of this work is giving a contribution to better understand the real behaviour of such polymeric geostrips. An experimental full-scale model of reinforced soil wall was built applying the quoted technology. The experimental model was fully instrumented to measure both stress and deformations on strips and to investigate about earth pressure. The Authors carried out a two-dimensional (2D) FEM analysis to simulate the wall performance. Experimental, numerical and analytical results have been compared in static condition, in order to detect the main aspects of the behaviour of soil walls reinforced with polymeric geostrips and to validate numerical analysis. Then, a prediction of the behaviour of the wall under seismic loading has been assessed.

Keywords: FEM analysis, reinforced earth, design methods, static analysis, seismic analysis.

## 1. Introduction

The modern soil reinforcement concept for retaining wall construction was introduced by Henri Vidal in the early '60s. His researches led to the development of "reinforced earth", a system in which steel bars reinforcements were used [FHWA-NHI, 2009].

In recent years reinforced earth technologies have been greatly developed. In terms of stress/strain behaviour, reinforcing elements may be considered as inextensible or extensible. Metallic reinforcements are usually considered as inextensible, while polymeric reinforcements are usually considered as extensible.

Vertical walls with concrete panels facing and steel bars reinforcement are commonly used technologies worldwide. These structures are usually designed with the *Coherent Gravity Method* [Anderson *et al.,* 2010] based on the assumption that reinforcements are inextensible. This method was first developed in France. Schlosser [1978] and Schlosser and Segrestin [1979] report valuable references about the early development of this design method. Among the earliest papers describing reinforced earth walls design methodology Minami and Adachi [1981] and Cazzuffi [1983] should be remembered.

On the other hand, mechanically stabilized earth (MSE) walls with extensible reinforcement are usually designed with the *Tieback Wedge Method* [Anderson *et al.,* 2010]. Nowadays, vertical walls with concrete facing panels are sometimes built using polymeric geostrip reinforcements in place of steel bars. Since tensile stiffness of geostrips is much larger than that typical of geosynthetic sheets, probably geostrips could be assumed to behave as inextensible reinforcement as well. Further researches, however, are required to ensure that such an assumption is correct: this work aim is to give a contribution about this aspect.

To better understand reinforced earth system behaviour, some experiments on full-scale reinforced structures have been carried out, but most of them were conducted on MSE walls reinforced with metallic strips, because they could be easily instrumented. On the contrary, only very few full-scale model tests have been conducted on MSE walls with geosynthetic reinforcements so that some aspects on these structures behaviour, including the slip surface location and the tensile force acting on reinforcements, remain unclear.

In this paper, a particular type of reinforced earth wall will be analysed, both in a full-scale model and in a FEM analysis, for better understanding its behavior, both in static and in seismic conditions.

[1] Ph.D. Geotechnical Engineering, Department of Civil Engineering and Architecture, University of Catania
[2] Associate Professor of Geotechnical Engineering, Department of Civil Engineering and Architecture, University of Catania
[3] Associate Professor of Geotechnical Engineering, Department of Civil Engineering and Architecture, University of Catania
[4] Ph.D. Geotechnical Engineering, Department of Civil Engineering and Architecture, University of Catania
[5] Ph.D. Student in Geotechnical Engineering, Department of Civil Engineering and Architecture, University of Catania
[6] Corporate Technical Director, Officine Maccaferri S.P.A., Zola Predosa, Italy
[7] Vertical Walls Corporate Technical Coordinator, Officine Maccaferri S.P.A., Zola Predosa,Italy
[8] Research & Development Department Manager, Maccaferri, Latin America

The chosen full-scale model is a reinforced soil wall that was built and fully instrumented at the Maccaferri establishment of Jundiai, Brazil. The applied technology is "MacRES", developed by Maccaferri. It is a Vertical Wall with Concrete Facing Panels and ParaWeb™ strips as reinforcements. In particular, ParaWeb™ strips are planar structures consisting of a core of high tenacity polyester yarn tendons encased in a polyethylene sheath.

Half of the experimental wall retained a sandy backfill, while the other half retained a silty backfill. The aim of the experimental full-scale model was to measure stresses and deformations on soil, strips and concrete facing.

During a first phase, laboratory tests for static and dynamic soil characterization have been performed. In a second phase a two-dimensional FEM analysis has been brought about to evaluate the wall behaviour in terms of stress and strain of soil, reinforcing elements and concrete panels, in the framework of the new performance-design approach.

In this paper, the results of a FEM numerical analysis are reported and discussed and a comparison between numerical and experimental results is shown. Finally, a FEM model prediction of the wall seismic behaviour, is presented.

## 2. MSE wall design methods

The most used methods for reinforced earth walls, reported in design codes and guidelines, are the *Coherent Gravity Method* [AASHTO, 1996] and the *Tieback Wedge Method* [AASHTO, 1996]. Both the methods use the limit equilibrium concept to develop the design model. Small-scale gravity and centrifuge models have been used to evaluate the design models performance at true limit equilibrium conditions (JURAN and SCHLOSSER, 1978; ADIB, 1988; CHRISTOPHER,1993).

The *Coherent Gravity Method* was initially developed by JURAN and SCHLOSSER [1978]. It assumes a bilinear envelope of maximum axial forces and a minimal elongation due to inextensible reinforcements. The horizontal pressure is evaluated multiplying the vertical pressure by a lateral coefficient that is the earth pressure coefficient at rest $k_0$ at the top of the wall and the active earth pressure coefficient $k_a$ at a depth of 6 metres below the top. A linear variation between $k_0$ and $k_a$ along the depth is assumed. The vertical earth pressure in turn is increased with respect to the typical value because of the overturning moment produced by lateral load at the back of the reinforced soil. So vertical stress is greater that $\gamma z$, being $\gamma$ the unit soil weight and $z$ the depth below the top of the fill. In a simplified formulation vertical stress could be assumed as $\sigma'_v = 1.20\,\gamma\, z$. Design characteristics of the *Coherent Gravity Method*, are shown in figure 1. Applied surcharges on the reinforced soil mass obviously increase vertical and horizontal stresses within the structure. The location of maximum axial forces is bilinear as show in figure 1. For inextensible reinforcements, the displacement at the leading end is nearly the same as the displacement at the free end, thus the reinforcement strain is negligible. The friction developed between the reinforcement and the soil is determined for a leading edge displacement of 20 mm, and the transfer of load to the soil via friction is uniformly distributed over the full length of the reinforcement.

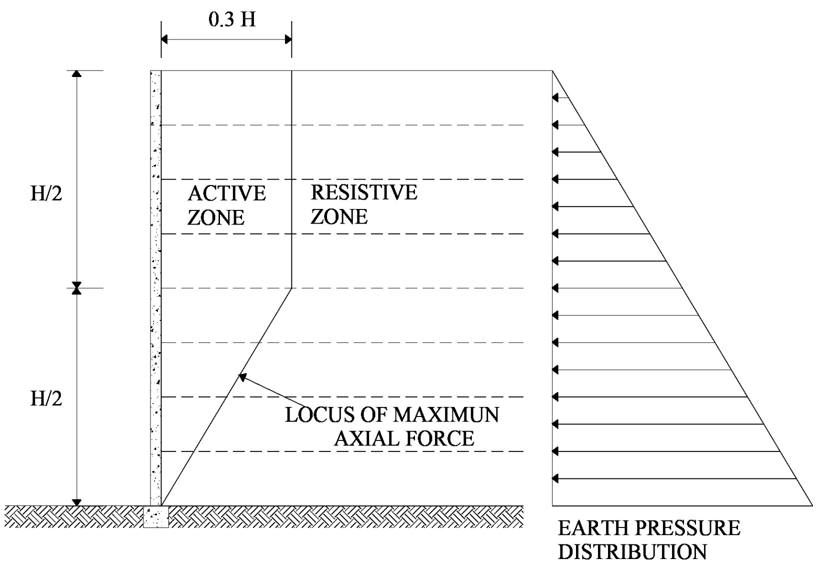



Fig. 1 – Scheme of *Coherent Gravity Method* for MSE walls design.
*Fig. 1 – Schema del* Coherent Gravity Method *per il progetto di muri in terra rinforzata.*

In the *Tieback Wedge Method* the wall is assumed to be flexible because geosynthetic reinforcements are considered as extensible. Due to the flexibility of the wall, the lateral pressure at the back of the reinforced zone does not influence the vertical pressure, which is simply equal to $\gamma z$. The method assumes that enough deformation occurs to allow an active state of stress to develop. Hence, horizontal stress can be determined multiplying the vertical stress by the active earth pressure coefficient $k_a$. Differently from *Coherent Gravity Method*, in an MSE wall with geosynthetic reinforcements designed by *Tieback Wedge Method*, the failure plane is assumed to develop along the Rankine rupture surface defined by a straight line oriented at an angle of $45°+\phi/2$ from the horizontal. The Rankine failure plane is not modified by inclusion of the extensible geosynthetic reinforcements. Therefore, reinforcement strain actually allows the failure plane to develop. As in the *Coherent Gravity Method*, in *Tieback Wedge Method* reinforcements are designed to resist the lateral pressure within their tributary area, treating each reinforcement layer as a tieback.

Both the *Coherent Gravity Method* and the *Tieback Wedge Method* are described in AASHTO LRFD Bridge Design Specifications [AASHTO, 2010]. The proper design method should be selected according to reinforcement properties and behaviour.

## 3. The full scale model test

A full-scale model test was carried out at the Maccaferri establishment of Jundiai, Brazil, by the technology of the Vertical Walls with Concrete Facing Panels (MacRES) developed by Maccaferri. Experimental data and measures of the instrumented wall are reported in table I. Reinforcements anchored to vertical facing concrete panels were utilized and embedded within the backfill during construction.

The wall retained a reinforced compacted soil layer about 6.0 m high. Soil reinforcement was realized using polymeric strips, characterized by high strength and soil-ground friction, located in eight layers throughout the height of the retaining walls at a distance of about 0.75 m from each other and were connected with the panels passing through special lugs. In the ground layer, each reinforcement tape was fixed using a row of bolts.

A surcharge of 20 kPa was afterwards applied on the backfill starting from the concrete panels.

Tab. I – Strain gauge readings on ParaWeb (after JAYAKRISHNAN, 2013).
*Tab. I – Letture di deformazione sui ParaWeb [JAYAKRISHNAN, 2013].*

| Paraweb Level | Distance from Face of Wall (m) | Strain Gauge No. | Sand before surcharge | | | | | | | Sand after surcharge | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Load (kN) | | | | | | | Load (kN) | | | | | | |
| | | | Nov (15/11) | Dec (15/12) | Jan (15/1) | Feb (15/2) | March (15/3) | April (2/10) | $Load_{max}$ | May (23/5) | June (2/6) | July (2/7) | Aug (2/8) | Sept (2/9) | Oct (2/10) | $Load_{max}$ |
| Level 1 | 0.17 | CS01 | 0 | 0.9 | 0.91 | 1.03 | 0.99 | 0.95 | 1.03 | 1.19 | 1.2 | 1.71 | 1.85 | 1.89 | 1.81 | 1.89 |
| | 0.43 | CS02 | 0 | 1.64 | 1.68 | 1.73 | 1.24 | 1.03 | 1.73 | 1.64 | 1.79 | 2.17 | 1.06 | 1.13 | 1.17 | 2.17 |
| | 0.72 | CS03 | 0 | 0.96 | 1.26 | 2.55 | 0.88 | 1.24 | 2.55 | 0.9 | 0.93 | 0.44 | 1.31 | 1.51 | 1.48 | 1.51 |
| Level 3 | 0.38 | CS04 | 0 | 0.28 | 0.28 | 0.28 | 0.28 | 0.28 | 0.28 | 0.27 | 0.27 | 0.26 | 0.26 | 0.27 | 0.26 | 0.27 |
| | 0.76 | CS05 | 0 | 1.3 | 1.16 | 1.76 | 2.06 | 1.71 | 2.06 | 2.19 | 2.33 | 2.7 | 1.89 | 2.23 | 2.11 | 2.7 |
| | 1.14 | CS06 | 0 | 0.93 | 1.25 | 1.51 | 1.64 | 1.52 | 1.64 | 2.06 | 2.22 | 2.33 | 1.95 | 1.53 | 1.66 | 2.33 |
| | 1.52 | CS07 | 0 | 1.45 | 1.53 | 1.56 | 1.63 | 1.57 | 1.63 | 1.92 | 2.07 | 2.23 | 2.3 | 2.31 | 2.23 | 2.31 |
| | 1.9 | CS08 | 0 | 0.29 | 0.3 | 0.29 | 0.29 | 0.2 | 0.3 | 0.45 | 0.56 | 0.96 | 1.08 | 1.08 | 1.07 | 1.08 |
| | 2.28 | CS09 | 0 | 0.16 | 0.72 | 0.74 | 0.75 | 0.7 | 0.75 | 0.9 | 0.97 | 1.13 | 1.14 | 1.18 | 1.14 | 1.18 |
| | 2.66 | CS10 | 0 | 0.49 | 0.44 | 0.55 | 1.02 | 0.84 | 1.02 | 0.65 | 0.73 | 0.73 | 0.77 | 0.79 | 0.72 | 0.79 |
| Level 5 | 0.68 | CS11 | 0 | 1.84 | 1.85 | 1.92 | 1.95 | 1.91 | 1.95 | 2.52 | 2.67 | 2.75 | 2.68 | 2.66 | 2.66 | 2.75 |
| | 1.1 | CS12 | 0 | 0.46 | 0.92 | 1.23 | 1.39 | 1.42 | 1.42 | 2.19 | 2.4 | 2.68 | 2.73 | 2.66 | 2.59 | 2.73 |
| | 1.44 | CS13 | 0 | 1.36 | 1.41 | 1.59 | 1.44 | 1.49 | 1.59 | 2.09 | 2.45 | 2.65 | 2.66 | 2.57 | 2.57 | 2.66 |
| | 1.86 | CS14 | 0 | 0.73 | 0.83 | 0.91 | 0.63 | 0.75 | 0.91 | 1.29 | 1.38 | 1.54 | 0.97 | 1.37 | 1.49 | 1.54 |
| | 2.2 | CS15 | 0 | 1.24 | 1.4 | 1.67 | 1.25 | 1.14 | 1.67 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | 2.62 | CS16 | 0 | 0.87 | 1.59 | 2.86 | 1.94 | 3.01 | 3.01 | 0.77 | 12.45 | 4.92 | 1.75 | 1.78 | 4.36 | 12.45 |
| | 2.96 | CS17 | 0 | 0.56 | 0.49 | 0.62 | 0.61 | 0.61 | 0.62 | 0.91 | 1.02 | 1.17 | 1.19 | 1.19 | 1.19 | 1.19 |
| Level 7 | 1.5 | CS18 | 0 | 0 | 0 | 0 | 0.00 | 0.92 | 0.92 | 2.23 | 2.35 | 2.61 | 2.65 | 2.61 | 2.62 | 2.65 |
| | 1.88 | CS19 | 0 | 0 | 0 | 0 | 0.00 | 0.98 | 0.98 | 1.85 | 2.02 | 2.22 | 2.22 | 2.19 | 2.17 | 2.22 |
| | 2.26 | CS20 | 0 | 0 | 0 | 0 | 0.00 | 0.78 | 0.78 | 1.49 | 1.56 | 1.67 | 1.69 | 1.68 | 1.66 | 1.69 |
| | 2.64 | CS21 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0.19 | 0 | 0 | 0.19 |
| | 3.02 | CS22 | 0 | 0 | 0 | 0 | 0.00 | 0 | 0 | 0.15 | 0.17 | 0.18 | 0.19 | 0.23 | 0.21 | 0.23 |
| | 3.4 | CS23 | 0 | 0 | 0 | 0 | 0.00 | 0.35 | 0.35 | 1.1 | 1.21 | 1.36 | 1.36 | 1.29 | 1.26 | 1.36 |
| Level 8 | 1.5 | CS24 | 0 | 0.76 | 0.94 | 0.92 | 0.99 | 0.89 | 0.99 | 2.27 | 2.34 | 2.46 | 2.45 | 2.41 | 2.35 | 2.46 |
| | 1.88 | CS25 | 0 | 0.69 | 0.65 | 0.63 | 0.65 | 0.62 | 0.69 | 2.11 | 2.22 | 2.35 | 2.37 | 2.37 | 2.33 | 2.37 |
| | 2.5 | CS26 | 0 | 0.72 | 0.77 | 0.87 | 0.87 | 0.81 | 0.87 | 2.17 | 2.32 | 2.4 | 2.39 | 2.35 | 2.34 | 2.4 |
| | 2.88 | CS27 | 0 | 0.68 | 0 | 0.66 | 0 | 0 | 0.68 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

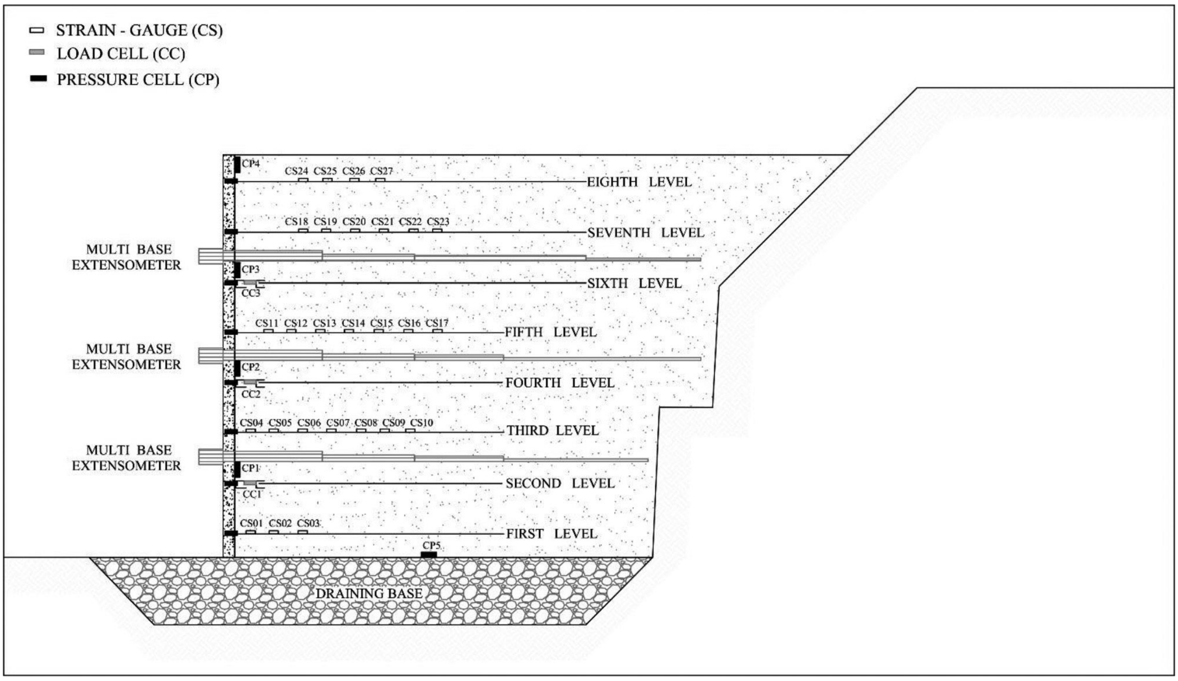



Fig. 2 – Instrumentation scheme of the tested MSE walls (after JAYAKIRISHAMN, 2013).
*Fig. 2 – Schema della strumentazione utilizzata nel muro sperimentale in vera grandezza [JAYAKIRISHAMN, 2013].*

The wall was divided into two parts. These allowed us to use sandy and silty soil separately as backfill, in order to understand the actual difference in behaviour in using these types of fills. Both backfill were compacted in order to reach a relative density of about 95%. The numerical analyses discussed in the present paper deals with the part of the wall retaining the sand backfill.

The instrumentation scheme of the experimental wall is as follows:

1. strain gauges on geostrips for reading actual force on soil reinforcement;
2. load cells on connections (MacLoop) between strips and concrete panels for reading actual force transferred to connections;
3. pressure cells below reinforced soil block for determining actual bearing pressure;
4. pressure cells on rear side of panels for determining actual pressure acting on the panel;
5. multi-base extensometers at different levels to detect soil deformations within reinforced and retained fills.

Figure 2 shows details of the instrumented section. More precisely, along geostrips 54 strain gauges were installed: 27 of them were installed in the sandy backfill and the other 27 in the silty backfill. The strain gauges were fixed on the strips by means of a special mechanical device, to crimp the plastic strip and to reduce external friction that would have influenced the readings.

Further load cells and pressure cells were used to measure forces at connections. Three load cells were installed in the sandy backfill wall and the other three in the silty one. From the readings of forces on geostrips, the locus of maximum tension was detected.

In order to know actual forces acting on panel connections and on rear side of panels, five pressure cells were used in the sandy backfill and five in the silty one.

Finally multi-base extensometers were used to evaluate axial deformation on strips.

A large number of static and dynamic laboratory tests were performed at the Department of Civil Engineering and Architecture at the University of Catania, on reconstituted specimens, to define the mechanical behaviour of Brazil soil employed as backfill in MSE walls. Static tests include direct shear tests on specimens reconstituted by the pluvial deposition method with different relative densities. Resonant column tests were also performed for the dynamic characterization of the soils, in order to detect shear modulus and damping ratio and their variations with shear strain. Results of static and dynamic laboratory soil tests are extensively reported in CAPILLERI *et al.* [2013].

A summary of main laboratory tests results, utilized for the static analysis in the MSE wall with sandy backfill is shown in table II. In table II the relative density has been achieved combining pluvial deposition and tapping methods. The grading curve of the sand backfill is reported in figure 3.

Reinforcements in MacRES walls tested in Brazil are geostrips disposed with a spacing of 0.75 m. The

Tab. II – Friction angle vs relative density.
*Tab. II – Angolo di resistenza al taglio in funzione della densità relativa.*

| $D_r$[%] | $\gamma$ kN/m$^3$ | e | Box 6x6 | | Box 10x10 | |
|---|---|---|---|---|---|---|
| | | | $\phi'_{average}$ | $f_{cv\_average}$ | $\phi'_{average}$ | $f_{cv\_average}$ |
| 25 | 1.55 | 0.74 | 29.87 | 29.29 | 32.45 | 31.78 |
| 50 | 1.62 | 0.67 | 34.44 | 32.67 | 33.46 | 32.33 |
| 95 | 1.77 | 0.53 | 45.04 | 44.12 | 37.52 | 38.47 |

length of the geostrips changes with the depth being 4.08 m at the lower levels and 5.08 m at the top.

According to technical specifications of ParaWeb-MD_45 the value of the ultimate tensile force is equal to 46.30 ±1 kN, while the strain at failure is equal to 12±1% (Tab. III).

## 4. Numerical analysis under static loading and comparison with experimental results

A two-dimensional numerical analysis of the half part of the wall retaining the sand backfill has been carried out with the finite element code Plaxis.

The FEM model has been meshed using the 15-node triangular element that, even if leads to computational time, allows an accurate calculation of stresses and failure loads [BRINKGREVE *et al.*, 2002]. This kind of element generates an irregular mesh that has usually a better numerical performance than the regular mesh [AMOROSI *et al.* 2007; 2008].

Tab. III – Mechanical properties of the utilised ParaWeb.
*Tab. III – Proprietà meccaniche dei ParaWeb utilizzati.*

| Reinforcement type | | MD 45 |
|---|---|---|
| Ultimate Tensile Strength (EN ISO 10319) | kN | 46.3-1.1 |
| Elongation at UTS (EN ISO 10319) | % | 12±1 |
| Strip width | mm | 48 |
| Strip thickness | mm | 3.3 |
| Strip weight | Kg/100m | 13.2 |
| Young Modulus E | MPa | 2400 |

Reinforcements are simulated using geostrip elements that are slender structures with an axial stiffness but with no bending stiffness. For the geostrip elements the Authors choose a linear elastic constitutive model. The equivalent unit axial stiffness of geostrip is evaluated in 1200 kN/m. The equivalent unit axial stiffness was evaluated multiplying the geostrip Young modulus for its transversal area for 1 m, then dividing the result for the geostrip length. Interface elements, between soil and geostrip elements are also included in the mesh. The friction along the soil-geostrip contact was taken into account through a reduction factor *R*, reducing the interface strength (Tab. IV).

The height of the mesh is 30 m on the left side and 46 m on the opposite side. The total width of the mesh is 200 m. At the lateral boundaries, horizontal displacements are not allowed; at the horizontal boundary, located at the bottom of the mesh, vertical and horizontal displacements are not al-

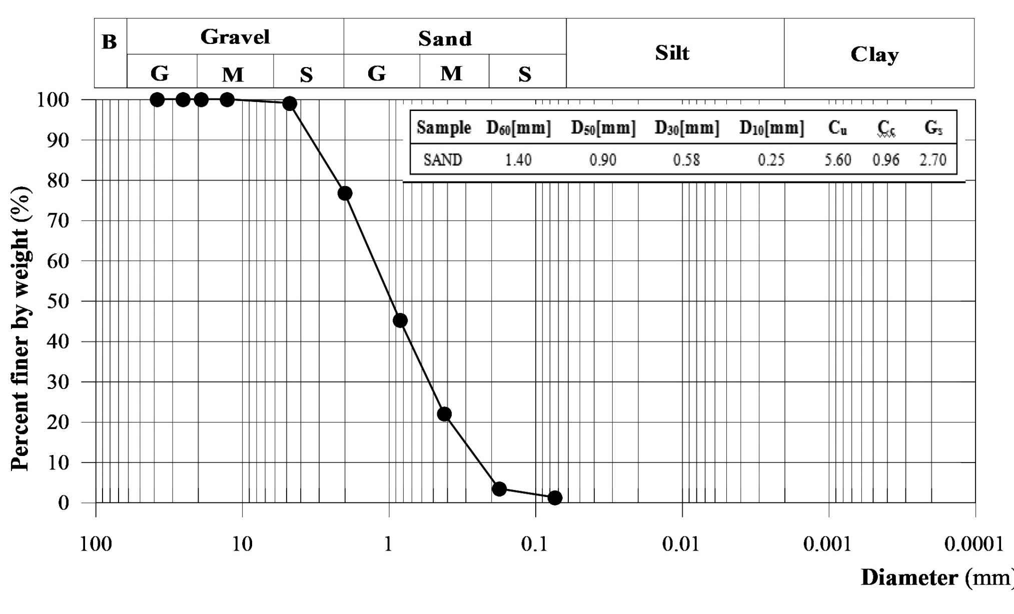



Fig. 3 – Grading curve for the sandy sample.
*Fig. 3 – Curva granulometrica per il campione sabbioso.*

Tab. IV – Soil parameters for FEM analysis.
*Tab. IV – Parametri del terreno utilizzati nell'analisi FEM.*

| | | Sandy backfill | Soil under and behind the wall | Interface geogrid |
|---|---|---|---|---|
| Material model | - | Mohr-Coulomb | Mohr-Coulomb | Mohr-Coulomb |
| Material type | - | dry | dry | - |
| Unit weight g | $kN/m^3$ | 17 | 18.5 | 17 |
| Young Modulus E | MPa | 50 | 60 | 50 |
| Poisson ratio n | - | 0.3 | 0.3 | 0.3 |
| Cohesion c′ | kPa | 0 | 20 | 0 |
| Effective friction angle j′ | (°) | 42 | 38 | 42 |
| Dilation angle ψ | (°) | 12 | 8 | 0 |
| Reduction factor | - | - | - | 0.70 |

lowed. The size of the mesh is sufficiently large so that the wall behavior is not influenced by boundaries.

Figure 4 shows respectively the whole mesh utilized in the analysis and a detail of the mesh in the zone where the MSE wall is placed.

The Mohr-Coulomb elasto-perfectly plastic constitutive model with non associate flow-rule is considered for both the soil characterizing the backfill and the soil underneath and behind the investigated wall. Soil parameters utilized in the numerical analysis are listed in table IV.

Young modulus of 50 MPa was assumed for the retained soil. For the underneath and behind soil a Young modulus of 60 MPa was utilized. Both modula where derived from shear modulus measured by resonant column test in the range of actual deformations.

The FEM analysis has been performed in the following steps:
– excavation;
– wall construction;
– surcharge application;
– seismic loading.

The horizontal displacement contour deduced from numerical analysis of MSE wall is shown in figure 5. FEM results indicate that, except for the lower part of the wall, which is restrained at the base, the horizontal displacements of the facing panels are almost the same. This indicates a translational behav-

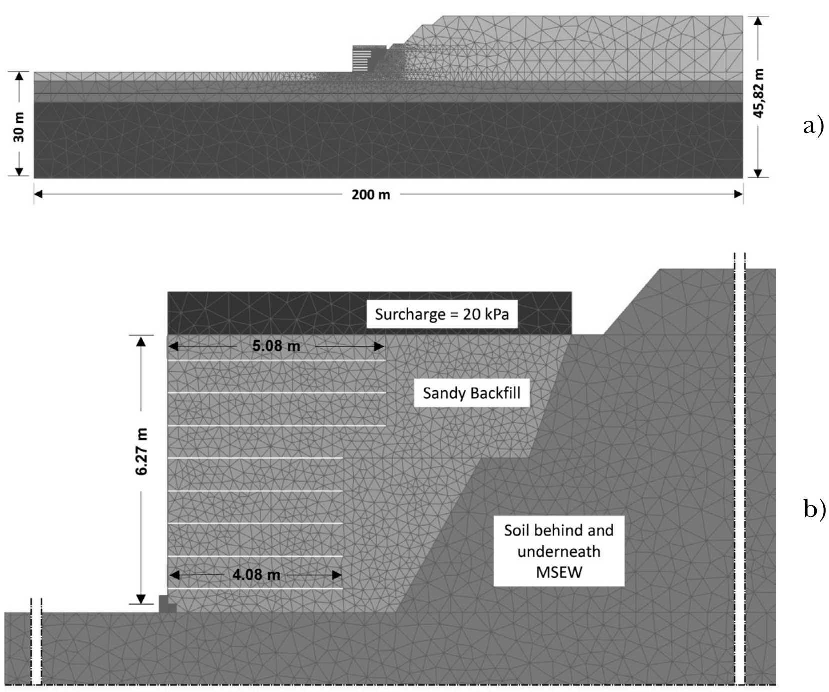



Fig. 4 – a) General view of the utilized mesh in the FEM analysis; b) A detail of the mesh.
*Fig. 4 – a) Vista generale della mesh utilizzata nell'analisi FEM; b) Dettaglio della mesh.*

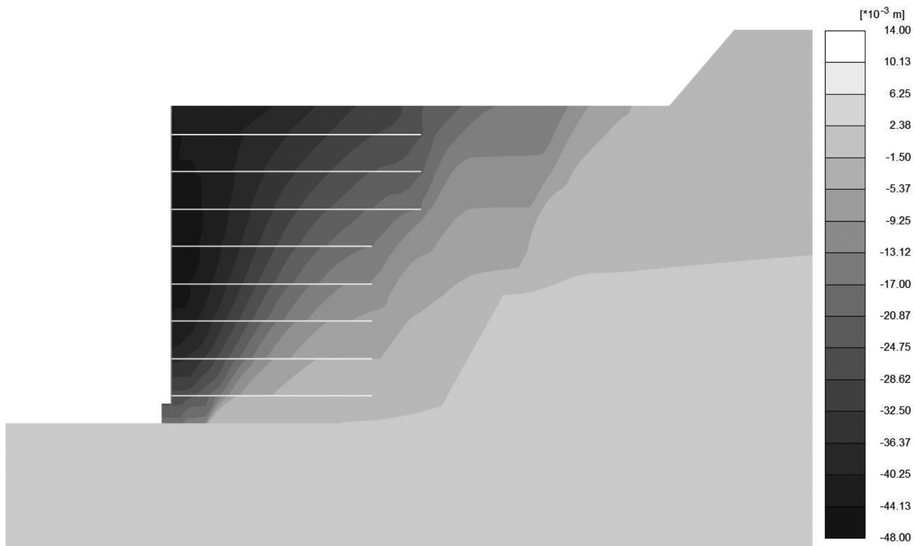



Fig. 5 – Horizontal displacement contour in FEM analysis.
*Fig. 5 – Spostamenti orizzontali determinati dall'analisi FEM.*

ior of the deformed wall. The maximum horizontal displacement deduced by FEM analysis at the elevation 1.5 m is about 47 mm.

Figure 6 shows the facing displacements before and after the application of the surcharge. It is possible to observe that the effect of surcharge is negligible with respect to that induced by self weight. Figure 7 shows the maximum axial forces $T_{MAX}$ in reinforcements for the analysis carried out without and with the surcharge. In both cases, the maximum axial force is found on the lowest reinforcement, being 19.35 kN/m for the analysis without surcharge, while it grows up to 23.63 kN/m when the surcharge is considered in the analysis.

In Figure 8 experimental and numerical results in terms of the locus of maximum axial forces are reported for without (Fig. 8a) and with surcharge (Fig. 8b). Figure 8 also reports the location of maximum axial forces according to *Tieback Wedge Method* (TWM) and the *Coherent Gravity Method* (CGM). Experimental and numerical curves are very close to the non-extensible reinforcement one calculated with the *CGM*. The locus of maximum axial forces rise up almost vertically and at a distance of about 0.3 *H* from the facing panels according to indications of the *Coherent Gravity Method,* being *H* the height of the wall. Moreover, experimental and numerical results are in good agreement for both the analyses carried out with and without surcharge.

Table V also shows the comparison between experimental results, FEM results and design values obtained with the *CGM,* in terms of axial forces in the geostrip reinforcements ($T_{MAX}$). Experimental axial forces measured in the full-scale model are at levels 1 and 3 lower than those evaluated by FEM analysis. This probably is a consequence of the lateral resistance developed at both the edges of the wall. Edge effects, due to a limited length of the wall, cannot be taken into account in a two-dimensional model so the actual earth pressure coefficient is somewhat less than that deducible by a plane analysis (MOTTA, 2012).

Maximum experimental axial strain, recorded at elevation 3, is equal to 0.65%, that is in a good agreement with the maximum axial strain deduced by FEM analysis ($\varepsilon$ = 0.78%). Thus, experimental results as well as FEM results, show that ParaWeb reinforcement, also being polymeric strip, can be used in the design as an inextensible reinforcement. So the *Coherent Gravity Method* can be used for this kind

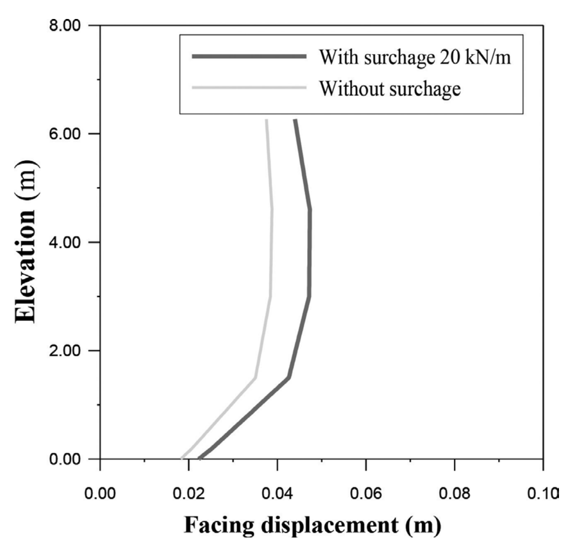



Fig. 6 – Facing panels displacements.
*Fig. 6 – Spostamenti dei pannelli di facciata.*

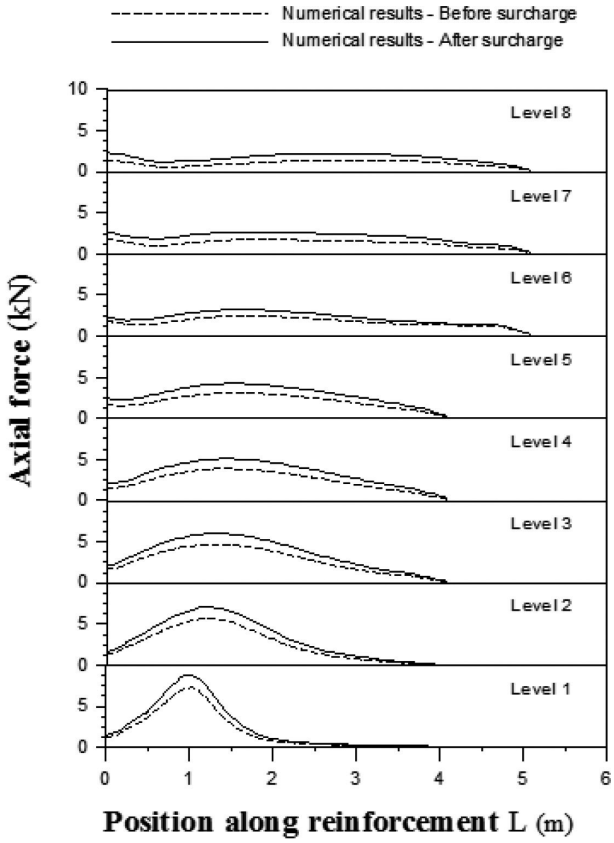



Fig. 7 – Axial forces on reinforcements by FEM analysis.
*Fig. 7 – Forze assiali sui rinforzi ricavate dall'analisi FEM.*

of walls design. This means also that, being the total elongation of geostrips lower than 1% in the limited length where the maximum strength is developed, the total elongation of the strip is negligible and the wall is perfectly vertical without any movement during and after the construction. $\varepsilon_{max}$ = 1% is the limit (maximum value) specified by BS 8006-1 for inextensible reinforcements.

Finally, table V reports a comparison between the experimental and the numerical tensile force on Macloop connection ($T_{MAX,MACLOOP}$). Once more, numerical results are in a very good agreement with the experimental results. In the instrumented wall, the maximum value of the connection strength was less than 5kN, as registered by the dedicated instruments. If compared with the values got in the design, the average reasonable over-design percentage may be from 50 to 60%.

Based on MOTTA [2012] solution, figure 9 shows the value of the 3D active earth pressure coefficient versus *H/D* ratio, being *H* and *D* the height and the length of the wall respectively. Curves of the 3D earth pressure coefficient in figure 9 are given for two different *ks* values, where *ks* is a lateral earth pressure coefficient relating horizontal stress acting at the lateral boundaries and vertical normal effective stress. *ks* usually ranges between 0.5 and 1. For a value *H/D*=0.4 (like in the full scale model), the three-dimensional active earth pressure coefficient $k_{aD}$ is lower than that

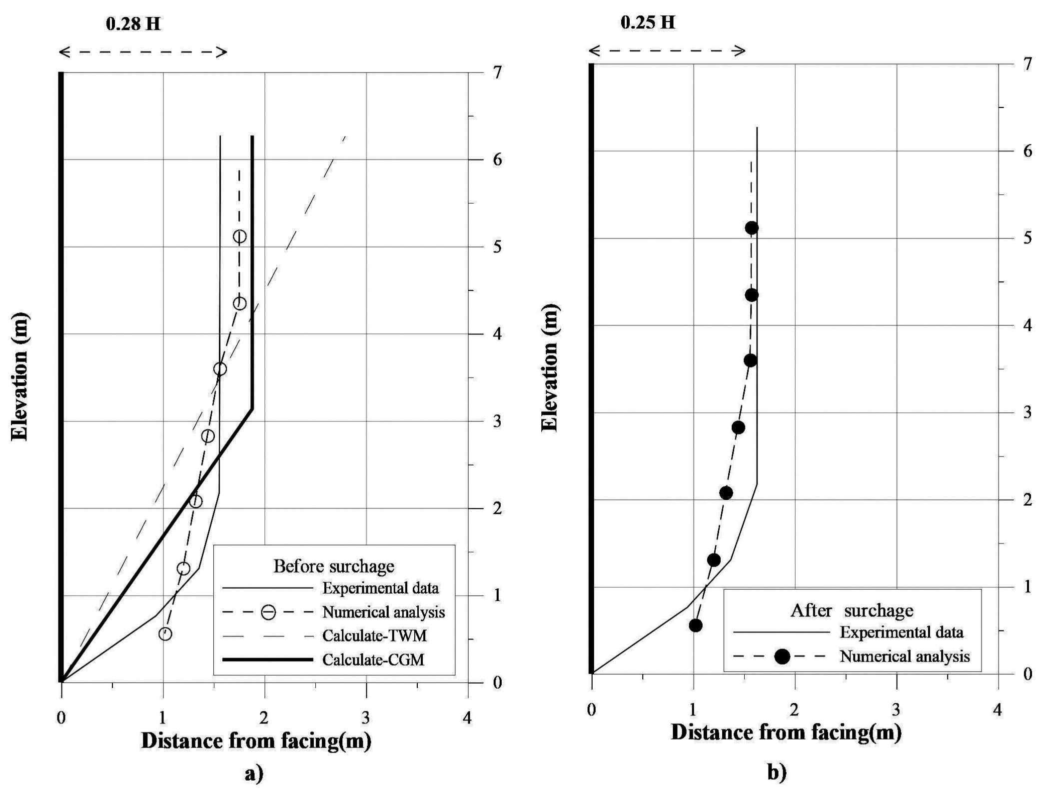



Fig. 8 – Locus of the maximum axial forces $T_{max}$ in reinforcements by FEM analysis: a) before surcharge b) after the surcharge.
*Fig. 8 – Luogo dei punti della massima forza assiale $T_{max}$ nei rinforzi ricavati dall'analisi FEM: a) Prima dell'applicazione del sovraccarico b) dopo l'applicazione del sovraccarico.*

Tab. V – Experimental, analytical and FEM-computed results for the MacRES wall System with an applied surcharge of 20 kN/m$^2$.

*Tab. V – Risultati sperimentali, analitici e numerici per il sistema MacRES con sovraccarico applicato di 20 kN/m$^2$.*

| Paraweb level | Paraweb Elevation (m) | Paraweb Length (m) | $T_{max}$ (Test) (kN) | $T_{max}$ (CGM Design) (kN) | $T_{max}$ (FEM) (kN) | Axial strain (Test) ε % | Axial strain (FEM) ε % | d (m) | d/h | $t_{max,macloop}$ (Test) (kN) | $t_{max,macloop}$ (FEM) (kN) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.37 | 4.08 | 2.17 | 11.24 | 8.86 | - | 0.54 | 1.02 | 0.16 | | 2.94 |
| 2 | 1.12 | 4.08 | NO LOAD CELLE | 10.47 | 7.07 | - | 0.7 | 1.20 | 0.19 | | 3.43 |
| 3 | 1.87 | 4.08 | 2.70 | 10.13 | 5.95 | 0.65 | 0.78 | 1.32 | 0.21 | 5.19 | 4.41 |
| 4 | 2.62 | 4.08 | NO LOAD CELLE | 9.70 | 5.08 | - | 0.74 | 1.44 | 0.23 | | 4.11 |
| 5 | 3.37 | 4.08 | 4.92 | 9.06 | 4.25 | 0.30 | 0.66 | 1.56 | 0.25 | 5.54 | 4.92 |
| 6 | 4.12 | 5.08 | NO LOAD CELLE | 5.94 | 2.65 | - | 0.47 | 1.57 | 0.25 | | 4.56 |
| 7 | 4.87 | 5.08 | 2.65 | 4.73 | 2.61 | - | 0.47 | 1.57 | 0.25 | NO READING | 5.18 |
| 8 | 5.62 | 5.08 | 2.46 | 3.49 | 2.22 | 0.09 | 0.37 | 2.72 | 0.43 | | 4.37 |

(Test) = after Jayakrishnan 2013 ;
D = distance from facing
H = wall height

evaluated in a two-dimensional analysis (plane case; $H/D = 0$). For example, for $ks = 0.5$ and $H/D = 0.4$, $k_{aD} = 0.130$ while for $ks = 1$ and $H/D = 0.4$, $k_{aD} = 0.089$. In the first case the reduction of the earth pressure coefficient with respect the plane case ($k_a$=0.182) is about 29%. In the second case the reduction is about 51%.

## 5. Prediction of the macres system wall seismic behavior

The seismic behavior of the MSE Wall has been also investigated using the Loma Prieta Earthquake recorded accelerogram, scaled at 0.25g as seismic loading. The recorded accelerogram of Loma Prieta Earthquake is shown in figure 10.

During seismic loading, axial forces on reinforcements, axial strains and horizontal displacements increase. In figure 11 results of the dynamic analysis are summarized.

The maximum axial force on reinforcements under seismic loading is shown in figure 11a. It is also compared with that obtained under static condition. The seismic analysis indicates that the axial forces increase during seismic loading mainly in the lower layers. A maximum increase of about 68% has been deduced at the lowest reinforcement.

The comparison of maximum axial force loci, under static and seismic loading, is reported in figure 11b. The locus of maximum axial force under dynamic condition is slightly shifted backwards with respect to the locus detected under static loading.

## 6. Conclusions

A FEM analysis has been carried out to study the behaviour of a full-scale MSE wall tested in Brazil. The comparison between experimental measures and numerical results appeared very precious

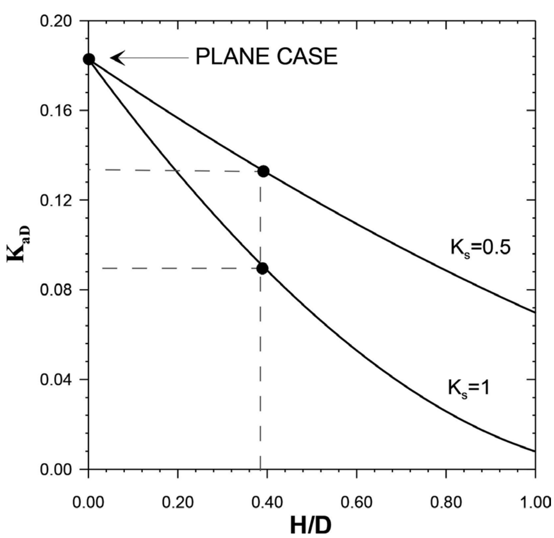



Fig. 9 – Three-dimensional active earth pressure coefficient versus H/D ratio.

*Fig. 9 – Coefficiente di spinta attiva tridimensionale in funzione del rapporto H/D.*

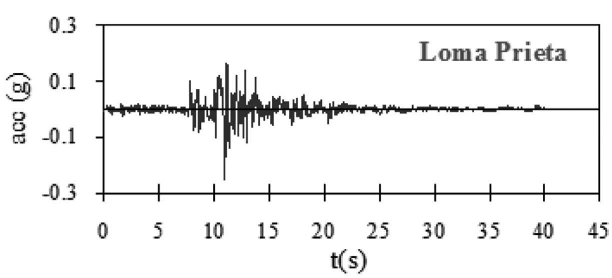
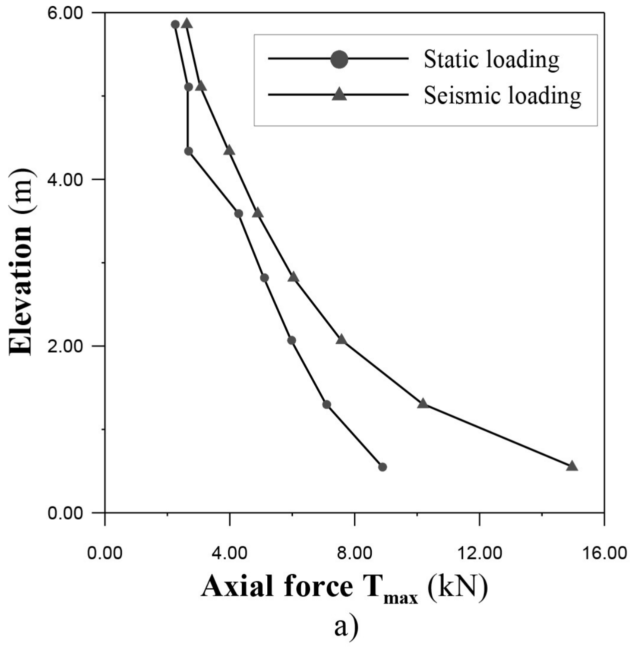



Fig. 10 – Input accelerogram recorded at Loma Prieta.

*Fig. 10 – Accelerogramma registrato a Loma Prieta.*



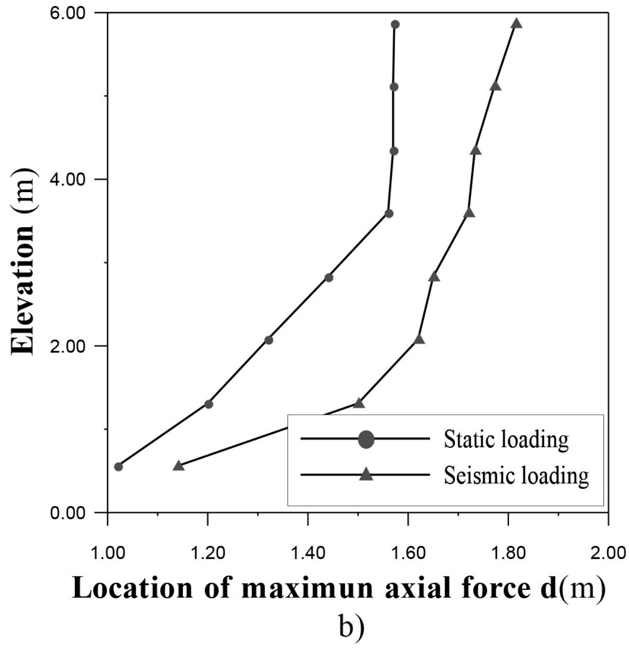




Fig. 11 – Seismic analysis results: a) axial force on reinforcements; b) locus of the maximum axial forces $T_{max}$ in reinforcements.

*Fig. 11 – Risultati dell'analisi sismica: a) forze assiali sui rinforzi; b) luogo dei punti della massima forza assiale $T_{max}$ nei rinforzi.*

due to the poorness of full-scale tests on MSE walls and in order to validate a FEM methodology to be easily applied to different initial and boundary conditions during the design routine. Experimental and numerical results has been also compared to results of the most used methods reported in design codes and guidelines. Thanks to these comparisons, a series of significant considerations have been made.

As observed in experimental results, the analysis confirms that the locus of maximum tension line starts from the bottom inside of the wall and ends, at the top, within a zone approximately 0.30 *H* far from the facing panels. According to SCHLOSSER [1978] the MSE wall behavior is similar to that of walls with inextensible reinforcements. This suggests the choice of the *Coherent Gravity Method* for design. More precisely the locus of maximum axial forces is found to rise up vertically at a distance of 0.28 *H* from the facing panels, if no surcharge is located at the top of the backfill; while applying a surcharge of 20 kN/m$^2$ the distance slightly decreases to about 0.25 *H*. Both experimental and numerical results show that geostrip reinforcement axial strains are usually less than the 1%, so the wall can be considered perfectly vertical without any movement during and after the construction. Thus, experimental results, as well as FEM results show that such geostrip can be considered as inextensible reinforcements and the *Coherent Gravity Method* can be applied for the design.

The seismic analysis, carried out using Loma Prieta earthquake scaled up to 0.25g, shows that during seismic loading axial forces on reinforcements can be significantly greater than those in the static case and the locus of maximum axial forces is slightly shifted backwards with respect to the static case.

The above-mentioned considerations are related to the used soil type, wall height, intensity of the surcharge, etc. New FEM analyses are in progress considering other soil backfill, as well as other recorded accelerograms. Preliminary results of these in progress FEM analyses confirm the considerations discussed in the present paper.

## Acknowledgments

This study was supported by cooperation agreement between Department of civil engineering and architecture (DICAR) and OFFICINE MACCAFERRI S.p.A. Research Project: *Reinforced earth retaining walls with rigid facing panels: experimental tests on physical models, theoretical analyses and design codes references.*

This paper is dedicated to the memory of Prof. Michele Maugeri.

## References

AASHTO (1996) – S*tandard specifications for highway bridges, with 1999 interims,american association of state highway and transportation officials, sixteenthedition.* Washington, D.C., USA, 686 p.

AASHTO (2010) – *LRFD bridge design specifications, 5th* *edition, American Association of State Highway and Transportation Officials.* Washington, DC.

ADIB M. E. (1988) – *Internal Lateral Earth Pressure in Earth Walls. Doctoral thesis submitted to the University of California.* Berkeley, California, 376 pp.

AMOROSI A., ELIA G., BOLDINI D., SASSO M., LOLLINO P. (2007). *Sull'analisi della risposta sismica locale mediante codici di calcolo numerici.* Proc. of IARG 2007 Salerno, IT.

AMOROSI A., BOLDINI D., SASSO M. (2008). *Modellazione numerica del comportamento dinamico di gallerie superficiali in terreni argillosi.* Rapporto di ricerca Politecnico di Bari.

ANDERSON P.L., GLADSTONE R.A., WITHIAM J.L. (2010) – *Coherent gravity: The correct design method for steel-Reinforced MSE Walls.* Proceedings of ER2010 Earth Retention Conference 3, ASCE, Bellevue,Washington.

BRINKGREVE R.B.J., AL-KHOURY R., BAKKER K.J., BONNIER P.G., BRAND P.J.W., BROERE W. (2002) – *Plaxis 2D- version 8 general information.* Delft University of Technology & plaxis *b.v.*, The Netherlands.

BS: 8006-1 (2010) – *Code of practice for strengthened / reinforced soils and other fills.*

CAPILLERI P., CAVALLARO A., MARE V., MAUGERI M. (2013) – S*tatic and dynamic characterization of soil in a reinforced earth wall.* XV Convegno Nazionale Anidis, L'ingegneria Sismica In Italia. Genova, 30 giugno - 4 luglio 2013.

CAZZUFFI D. (1983) – *The use of geosynthetics in reinforced earth walls: design criteria and experimental test* (in Italian). Proc. XV Italian Geotechnical Conference, vol. III, Spoleto, May, pp. 311-320.

CHRISTOPHER B. R (1993) – D*eformation response and wall stiffness in relation to reinforced soil wall design.* PH.D. Dissertation, Purdue University, 352 pp.

COLIN J.F.P. (1985) – *Earth reinforcement and soil structures.* Butterworth advanced series in geotechnical engeenering.

FHWA NHI-10-024 (2009) – *Design and construction of mechanically stabilized earth walls and reinforced soil slopes* – volume I

JAYAKRISHNAN P.V. (2013) – *Report on full scale instrumented macres wall at jundiai-Brazil* (Part-1).

JAYAKRISHNAN P.V. (2013) – *Report on full scale instrumented macres wall at jundiai-Brazil* (Part-2).

JURAN I., SCHLOSSER F. (1978) – *Theoretical analysis of failure in reinforced earthstructures.* Proceedings, ASCE Symposium on Earth Reinforcement, Pittsburgh, pp. 528-555.78.

MINAMI K., ADACHI K. (1981) – *Behaviour of reinforced soil wall and design.* Civil Engineering Journal, Public Works Research Center, 23-7, pp.15-20.

MOTTA E. (2012) – *Sul coefficiente di spinta attiva in terrapieni di lunghezza finita.* Italian Geotechnical Journal, n. 4, ottobre-dicembre 2012, pp. 57-66.

PASLEY C.W. (1822) – *Experiments on Revetments.* vol. II, Murray, London.

SCHLOSSER F. (1978) – *History, current development, and future developments of reinforced earth.* Symposium on soil reinforcing and stabilizing techniques, Sponsored by New South Wales Institute Of Technology And The University Of Sidney, Austrailia 79.

SCHLOSSER F., SEGRESTIN P. (1979) – *Dimensionnement des ouvrages en terre armee par la méthode de l'équilibre local.* International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques, Paris, vol. I, pp. 157-162.

## Analisi numerica di un muro in terra rinforzata in vera grandezza: comportamento statico e dinamico

### *Sommario*

*Attualmente sono disponibili molteplici tipologie di muri in terra rinforzata. Una delle tipologie più interessanti è rappresentata dai muri a parete verticale con rinforzi polimerici nastriformi e con pannelli di facciata in calcestruzzo. La rigidezza a trazione di questi rinforzi è molto più grande di quella dei rinforzi geosintetici usuali e una delle incertezze che possono insorgere è se tali rinforzi possono essere considerati estensibili o inestensibili. Lo scopo del presente lavoro è dare un contributo per una migliore comprensione del reale comportamento di tali rinforzi polimerici. Allo scopo è stato realizzato un muro sperimentale in vera grandezza in cui sono stati utilizzati rinforzi nastriformi denominati "ParaWeb". Il muro è stato strumentato al fine di misurare deformazioni e tensioni sui rinforzi come anche le pressioni prodotte dal terreno. Come confronto con i risultati sperimentali è stata condotta un'analisi bidimensionale agli elementi finiti per simulare il comportamento del muro. I risultati sperimentali e analitici sono stati messi a confronto fra loro in condizioni di carico statico al fine di individuare gli aspetti peculiari del comportamento di tali muri rinforzati con "ParaWeb" e per validare le analisi numeriche. Infine è stata fatta una previsione del comportamento di tali muri sotto carico sismico.*

*Parole chiavi: analisi FEM; terra rinforzata; metodi di progettazione; analisi statica; analisi sismica.*