E. Cosenza (ed), *Eurocode 8 Perspectives from the Italian Standpoint Workshop*, 291-300, © 2009 Doppiavoce, Napoli, Italy

PERFORMANCE-BASED DESIGN OF EMBEDDED RETAINING WALLS SUBJECTED TO SEISMIC LOADING

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ABSTRACT
In common practice, the seismic design of an embedded retaining wall is carried out using the pseudo-static method. In this approach, constant forces are introduced in a limit equilibrium calculation, and the seismic analysis of a retaining wall is treated similarly to the evaluation of the safety against a collapse mechanism. This paper is aimed to propose a reconsideration of the simple pseudo-static calculation: it shows that the method can be used within the context of the performance-based design to predict the actual seismic performance of the wall, and that concepts employed in the capacity design of structural members can be extended to the design of embedded retaining walls. The paper also points to possible code prescriptions that may provide guidance for the correct application of the pseudo-static method to the design of retaining walls.

KEYWORDS
Seismic design, retaining walls, numerical analysis, nonlinear response, earthquake resistant structures.

1 INTRODUCTION

In the performance-based design of a retaining wall, different degrees of seismic protection can be prescribed, depending on the limit state being analysed. For instance, the current Italian Construction Code (Decreto Ministeriale 14.1.2008) defines four different limit states, each associated to a different seismic action, characterised by a given probability of exceedance in the structure's lifetime.

For a building, the seismic performance is usually expressed by the maximum instantaneous displacement (e.g. the inter-storey drift) that occurs during the earthquake, and by the ductility demand associated to this displacement. On the other hand, the seismic performance of a retaining wall is more commonly expressed in terms of permanent displacements at the end of the earthquake (Richards & Elms 1979, PIANC 2001). A possible reason of this difference is that, unlike buildings, retaining structures undergo permanent displacements in one direction only, and therefore the displacements increase monotononically, attaining a maximum at the end of the seismic event.

In order for the displacements to be irreversible, they must stem from the instantaneous development of a plastic mechanism; therefore these displacements are associated to quasi-rigid body movements. For an embedded retaining wall, if one admits that only the soil strength can be fully mobilised during the seismic event, while the retaining wall and the restraining system remain in an elastic state, then a plastic mechanism can form only if the wall is cantilevered or singly restrained (propped or anchored). The present discussion is largely devoted to the design of these wall categories. It is also assumed that, because of the limited height of these wall types, the dynamic motion of the soil interacting with the excavation is essentially synchronous. Some effects of asynchronicity on the design of the retaining structures for deep excavations are discussed by Callisto & Aversa (2008).

2 EVALUATION OF THE SEISMIC PERFORMANCE

Upon instantaneous attainment of the available strength, cantilever and singly-restrained walls can undergo rigid-body movements in a way which is qualitatively similar to the behaviour of gravity retaining walls. This was shown, for instance, by the experiments of Richards & Elms (1992) and Neelankatan et al. (1992), and by the results of some dynamic tests that were recently carried out using the Cambridge Dynamic Centrifuge (Viggiani & Conti 2008): for accelerations larger than a critical value, a progressive development of wall rotations was observed. Figure 1 is taken from the results of a series of dynamic numerical analyses carried out by Callisto et al. (2008): it shows the instantaneous distribution of the contact stresses σ_h exerted by the soil against a pair of mutually propped retaining walls during the strong motion phases of two different seismic events. During the earthquake, these stresses increase both at the rear and in front of the walls, and permanent displacements occur as a consequence of full mobilization of the soil strength. This phenomenon occurs in an alternate fashion to the two facing walls: in Figure 1 it is happening to the left-hand wall. Figure 2 shows, for the two walls, the progressive accumulation of the computed horizontal displacement of the toe relative to the top.

For a given retaining wall, a critical value a_c for the horizontal acceleration can be evaluated by performing iteratively a limit equilibrium analysis, and finding the pseudo-static acceleration for which soil strength is fully mobilised. Permanent displacements can then be assumed to result from a Newmark-type integration of the relative motion, and are bound to decrease as a_c increases. For a given soil and a given excavation height H, the value of a_c depends essentially on the embedded length d. Therefore, the embedded length (or, equivalently, the total length $L = H + d$) should be chosen on the basis of the maximum displacement allowed for the seismic event (and therefore the limit state) under consideration.

Relationships between the permanent displacements u and the ratio a_c/a_{max} were recently derived by Rampello & Callisto (2008) from a parametric integration of a database of Italian Strong Motion Accelerograms (SISMA, Scasserra et al. 2008). Each recording was scaled to different maximum accelerations a_{max}, with a scale factor not exceeding the value of 2. Figure 3 shows the u-a_c/a_{max} relationships obtained for accelerograms recorded on rock and scaled to $a_{max} = 0.35$ g, that yielded the largest displacements. Also shown in the figure are regression lines through the computed data points, of the form:

$$u = B \exp\left(A \frac{a_c}{a_{max}} \right); \tag{1}$$

specifically, the continuous line, computed with $A = -7.4$ and $B = 1.8$, is close to the upper bound of the results and was used to develop a relationship between the expected displacement (that define the requested seismic performance) and the pseudo-static horizontal acceleration that was then adopted by Italian Construction Code (Decreto Ministeriale 14.1.2008) for the seismic design of embedded retaining walls.

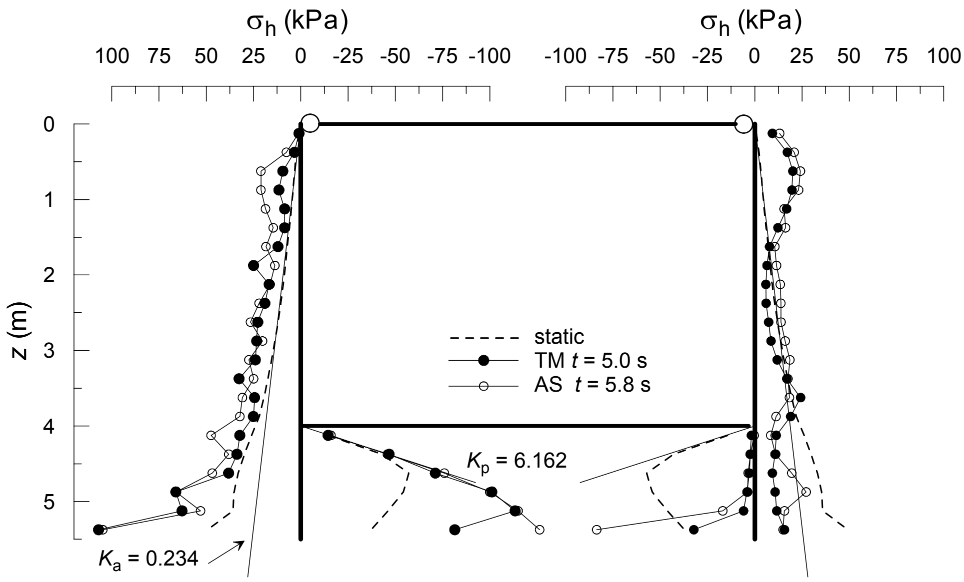

Figure 1. Contact stresses acting against a pair of propped retaining walls (Callisto et a. 2008).

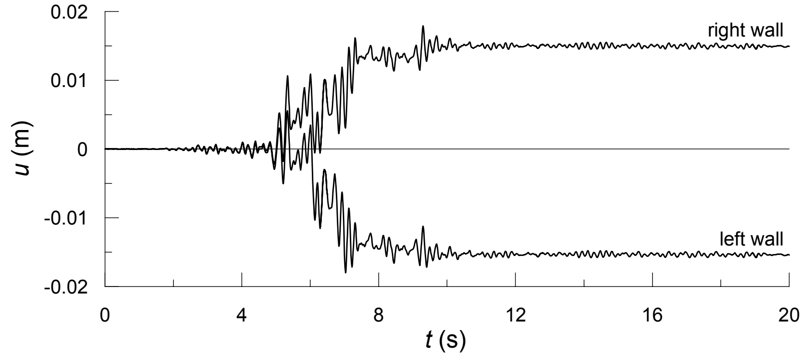

Figure 2. Time histories of the top-toe relative displacements of the retaining walls shown in Figure 1.

Consider the example of Figure 4(a), taken from Soccodato & Callisto (2009), where an excavation with $H = 4$ m in a homogenous coarse-grained soil is retained by a cantilevered wall. The soil has a constant angle of friction $\varphi = 35°$, while the soil-wall angle of friction is $\delta = 20°$. For this retaining wall, the critical acceleration a_c was found using the Blum (1931) limit equilibrium method; the Mononobe-Okabe solution was used for the seismic active pressure, while the coefficient of passive pressure was evaluated with the closed-form solution developed by Lancellotta (2007).

Figure 4(b) shows, for the case at hand, the computed values of the critical acceleration a_c plotted as a function of the total length of the wall. As L varies from 7 to 9 m, a_c increases from 0.15 to 0.4 g. Figure 4(b) also shows the permanent wall displacements u computed using equation (1) and assuming that the maximum horizontal acceleration in the soil interacting with the wall is either $a_{max} = 0.5$ g or $a_{max} = 0.75$ g: as the length of the wall increases, the permanent displacements u decrease rapidly.

Maximum bending moments were computed in the wall using the limit equilibrium method, with a pseudo-static horizontal acceleration equal to a_c: it can be expected that, as the accelerations increase during an earthquake, so do the soil-wall contact stresses, until full mobilisation of soil strength is attained, that is until the critical acceleration a_c is reached. For accelerations larger than a_c, contact stresses cannot vary significantly, since soil strength is

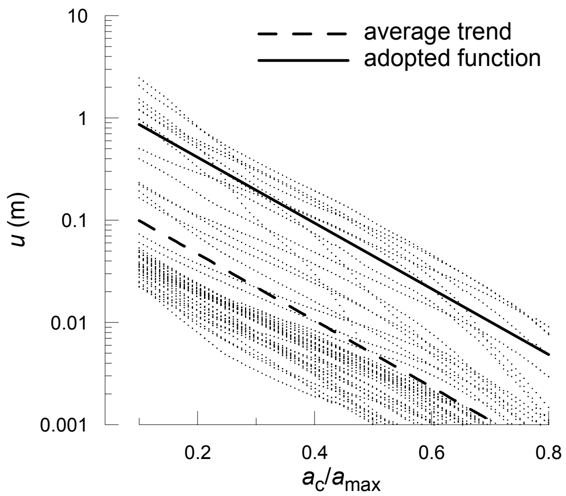

Figure 3. Relationships between permanent displacements and the ratio a_c/a_{max} obtained by Rampello & Callisto (2008) for very stiff soils and a_{max} = 0.35 g.

already fully mobilised, and the acceleration in excess of a_c is spent to produce a movement of the wall under quasi-constant contact stresses. Since the internal forces in the wall are a consequence of the contact stresses, bending moments should increase as the acceleration increases to up the critical value, and then remain constant while relative soil-wall displacements occur.

The maximum bending moments computed using the critical acceleration are indicated as $M(a_c)$ and are plotted in Figure 4(b) as a function of the wall length L. Since a_c increases with L, the bending moments increase as well. This means that if the wall is made longer in order to improve its seismic performance (that is, to undergo smaller displacements) it will have to sustain larger bending moments.

Of course, for a retaining wall with $a_c > a_{max}$ the permanent displacements are negligible and the internal forces in the wall are evaluated with a pseudo-static acceleration equal to a_{max}.

3 DESIGN CRITERIA

In its essential terms, the design of an embedded retaining wall with no more than one restrain could be performed through the following steps:

a. for a given limit state, define the required seismic performance by selecting the maximum permanent displacement u;
b. evaluate the maximum horizontal acceleration a_{max} expected for the limit state under consideration;
c. from a relationship of the type shown in Figure 2, evaluate the critical acceleration a_c needed to meet with the desired seismic performance;
d. search iteratively, through the limit equilibrium method, the wall length L that gives the required critical acceleration; if for any reason (e.g. for hydraulic needs) the chosen wall length is larger than L, the critical acceleration a_c must be recalculated using the actual length;
e. compute the internal forces using the contact stresses evaluated with $a = a_c$;
f. design the wall structure (and the eventual restraining system) on the basis of the internal forces evaluated at the previous step.

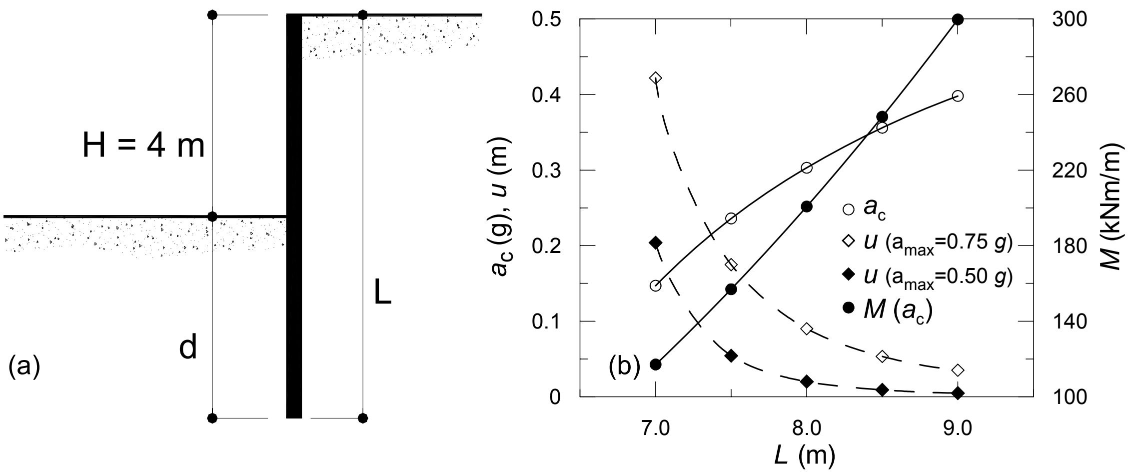

Figure 4. Layout of a cantilevered retaining wall in a coarse-grained soil (a) and plots of the critical acceleration, the permanent displacement and the maximum bending moment as a function of the wall length (b) (Soccodato & Callisto 2009).

Although the above sequence of activities is conceptually clear, some issues are believed to necessitate further investigation.

Firstly, the evaluation of the maximum horizontal acceleration a_{max} at step (b) poses some problems: in principle, a_{max} could be found using the simplified procedures based on amplification coefficients, such as those prescribed by the Eurocode 8 (EN 1998-5) or by the Italian Construction Code (Decreto Ministeriale 14.1.2008). Yet these procedures, being largely based one-dimensional site response analyses, neglect the effect of two-dimensional amplification and may underestimate a_{max}. For instance, a parametric study on cantilevered retaining walls, based on the results of two-dimensional dynamic numerical analyses (Callisto & Soccodato 2009) showed that the maximum acceleration in the soil interacting with the excavation may reach values larger than twice the corresponding maximum acceleration computed in a one-dimensional analysis, and that this effect is not significantly related to the soil or wall stiffness, but rather depends on the two-dimensional nature of wave propagation.

Step (c) implies the availability of relationships similar to the one shown in Figure 2, as specific as possible to the geographic region and to the source mechanisms under consideration.

Step (f) needs further discussion. It may be required that the wall structural strength should be larger than the internal forces evaluated with the pseudo-static method using the critical acceleration a_c. This should be considered equivalent to a common practice used for the capacity design of structural members: energy-dissipating elements of mechanisms are chosen, and other elements are provided with a sufficient reserve strength capacity, to ensure that the chosen energy dissipating mechanisms are maintained at their full strength throughout the deformations that may occur. In the case at hand, a natural choice for the energy dissipating element may be the soil interacting with the retaining wall, also considering that in the initial conditions the strength of significant soil volumes located in the vicinity of the wall is already fully mobilised.

However, a pseudo-static calculation of the bending moments in the walls with $a = a_c$ typically assumes a linear distribution of the contact stresses, while the actual distribution of the contact stresses may deviate from this simple distribution; this is visible in Figure 1, and is further substantiated by Figure 5, that shows the instantaneous distribution of σ_h computed for the cantilevered wall of Figure 4(a) during the development of a plastic mechanism (Soccodato & Callisto 2009), together with the corresponding bending moments and

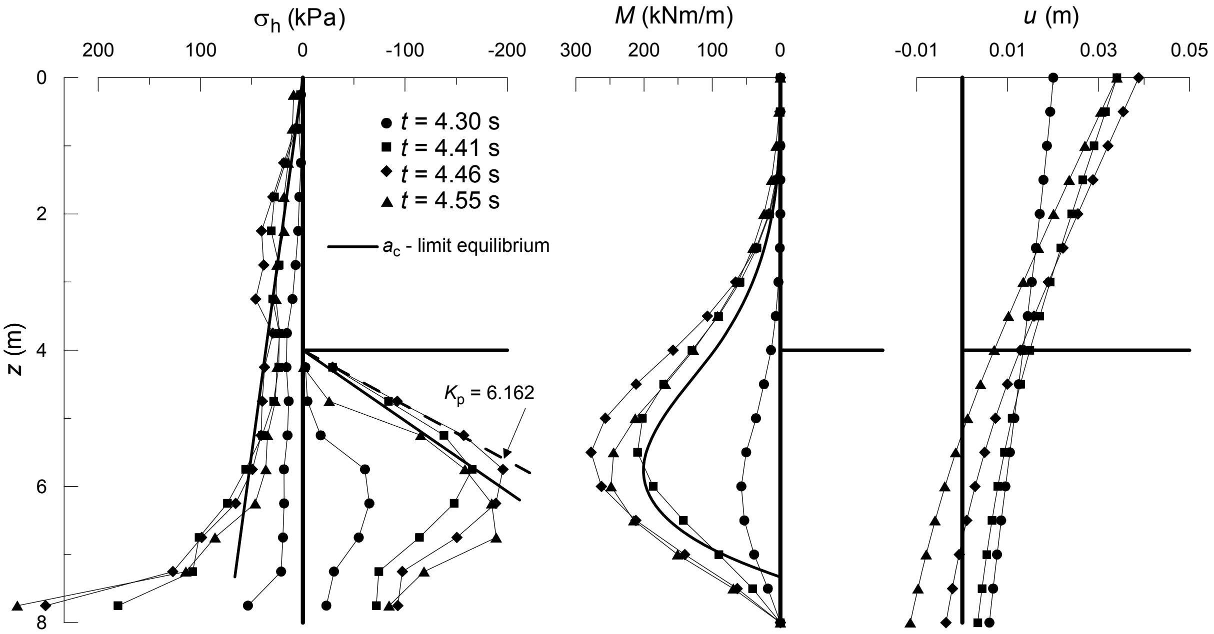

Figure 5. Contact stresses, bending moments and horizontal displacements of a cantilevered embedded retaining wall at several instants during a severe seismic event, compared with results of pseudo-static calculation in critical condition (Soccodato & Callisto 2009).

horizontal displacements. The thick lines in Figure 5 show the limit equilibrium computations with $a = a_c$. It can be seen that the actual instantaneous distribution of the contact stresses can be somewhat different from the simplified distribution, and this makes the maximum bending moments M_{max} larger than $M(a_c)$.

For the same cantilevered retaining wall in a coarse-grained soil, Callisto & Soccodato (2009) showed that the ratio of M_{max} to $M(a_c)$ increases with the stiffness of the wall, but is bounded by a maximum of about 1.6. Therefore, this figure could be adopted as a multiplier of $M(a_c)$ to account for the difference between the simplified and the actual distribution of contact stresses, at least for the particular case examined. In order to protect the retaining wall from bending yielding, a further over-strength factor might be required.

Soccodato & Callisto (2009) explored a different approach, in which no multipliers or over-strength factors were used, and the walls were given a bending strength M_y about equal to $M(a_c)$, allowing both the soil and the retaining walls to undergo plastic yielding during the earthquake. Hence, in this approach yielding of the wall is called to compensate for the inaccuracies of the assumed distribution of the contact stresses. The Authors carried out a series of dynamic numerical analyses, in which two different seismic records were applied to pairs of cantilevered retaining walls with $H = 4$ m and three different embedment depths. The mechanical behaviour of the walls was described with a linearly elastic-perfectly plastic moment-curvature relationship, in which, for the three different walls, the curvature at yielding ψ_y was about constant. The soil behaviour was described by a non-linear hysteretic constitutive model coupled with a Mohr-Coulomb failure criterion (Callisto & Soccodato 2007). Figure 6, taken from Soccodato & Callisto (2009), shows for the two seismic inputs the maximum displacements and bending moments plotted as a function of the overall wall length L. Results are also concisely reported in Table 1.

It can be seen from Figure 6 that the decrease of the displacements with L is qualitatively similar to the results shown in Figure 4(b). The elastic-plastic walls undergo displacements u_y that are larger than those computed for the elastic walls (u_{el}), but the difference becomes very small with increasing L (see Table 1). Bending moments increase with L, as it was expected

Table 1. Main results obtained from the numerical analysis presented by Soccodato & Callisto (2009).

L (m)	u_y/u_{el}	M_{fin}/M_y	ψ_{max}/ψ_y
7	1.59	0.96	43
8	1.39	0.87	20
9	1.13	0.71	2.7

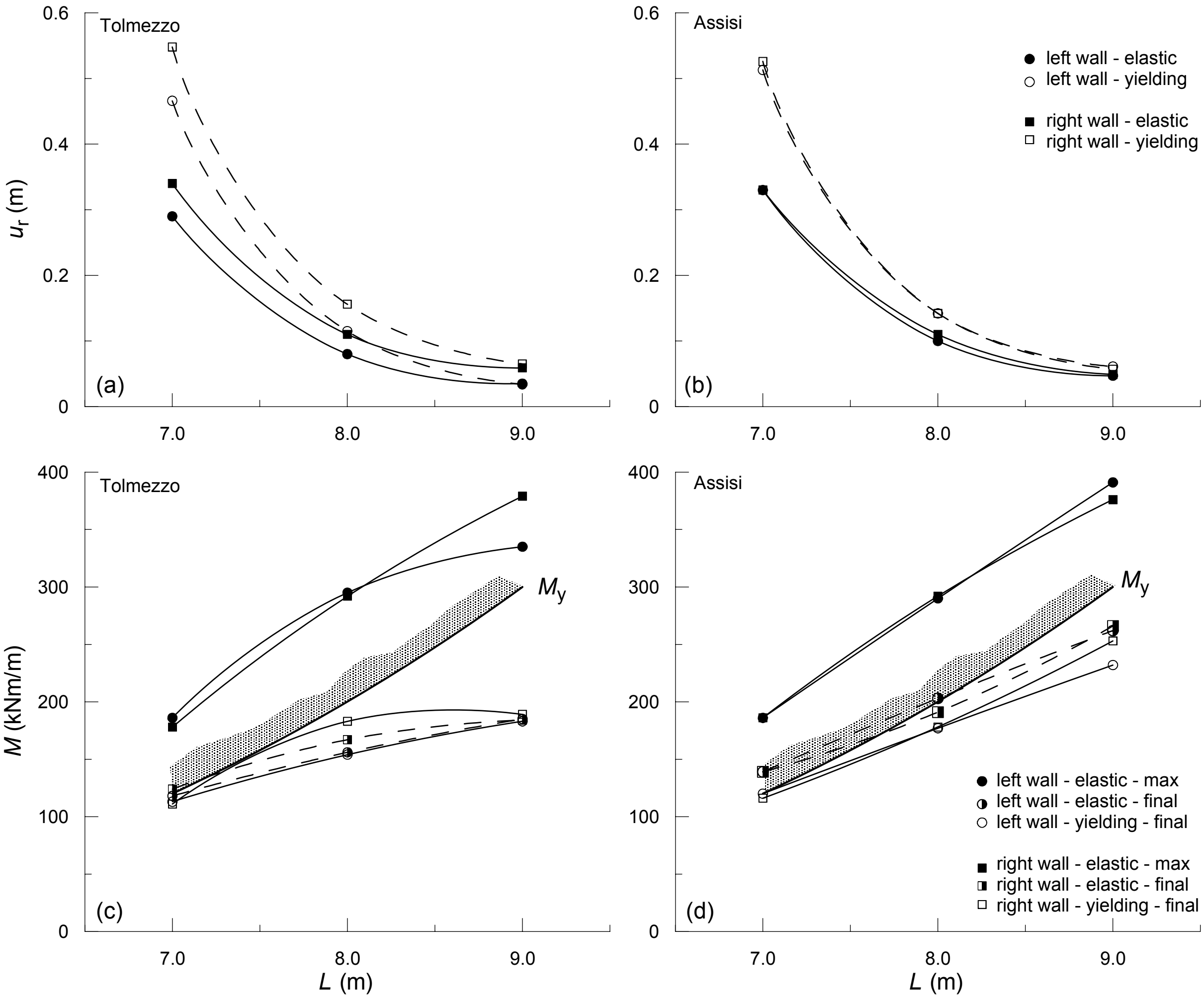

Figure 6. Permanent displacements and maximum bending moments in cantilevered embedded retaining walls of different lengths (Soccodato & Callisto 2009).

(see Figure 4(b)). The final values M_{fin} to range from 70 % (L = 9 m) to more than 95 % (L = 7 m) of M_y.

The mobilised strength in the soil can be quantified by the ratio τ/τ_{lim} of the maximum tangential stress acting at a point to the corresponding available strength. Figure 7 shows, for the cantilevered wall, the contours of the mobilised strength computed by Callisto & Soccodato (2007) before (a) and after (b) a severe earthquake. Before the earthquake, a significant mobilisation of the shear strength is obtained behind the wall, where the soil is in an active limit state, and right below the bottom of the excavation, where the soil is in a passive limit state. At the end of the earthquake the stress state in most of the soil interacting with the walls is quite far from a plastic limit state and the corresponding distribution of the contact stresses produces the large post-seismic bending moments of Figure 6. However, Callisto & Soccodato (2009) showed that a further small excavation causes the soil behind the wall to reach once more a limit active state and therefore produces a decrease of the contact stresses, with an ensuing reduction of the bending moments. Hence, the relatively high internal forces that remain locked into the wall after the earthquake are not deemed capable to endanger the overall safety of the system.

In order to judge the performance of a wall that undergoes plastic yielding during the seismic event, it is of interest to quantify the curvature ductility demand, that is the ductility required for the wall sections to undergo the computed plastic curvatures without a significant strength degradation. Table 1 reports, for the cases considered, the values of the curvature ductility factor, defined as the ratio of the maximum curvature ψ_{max} to the curvature at yield ψ_y. It appears that for the walls with $L = 8$ and 9 m the curvature ductility demand can easily be satisfied by a properly detailed r.c. section.

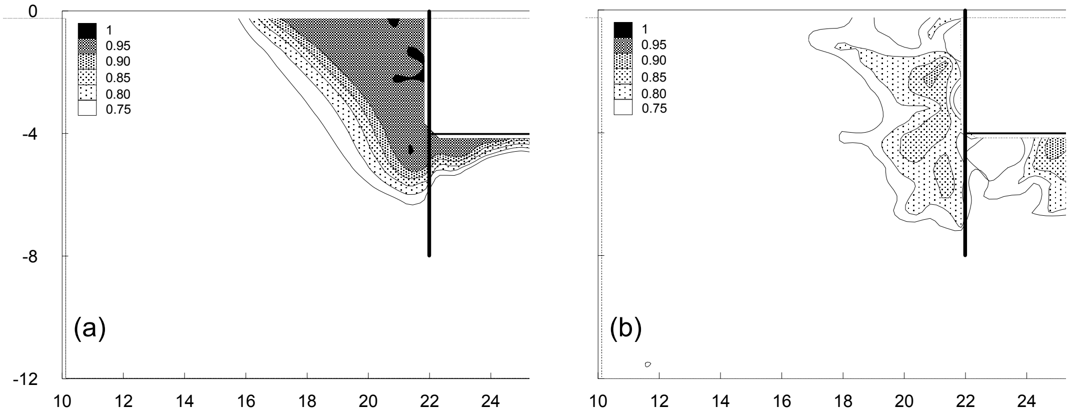

Figure 7. Contour plots of the mobilised strength before (a) and after (b) a seismic event (Callisto & Soccodato 2007).

4 DEVELOPMENT OF TECHNICAL CODES

The Eurocode 8 (EN 1998-5) prescribes that the pseudo-static design of flexible retaining walls be carried out using the maximum expected acceleration a_{max}. From the above discussion, it follows that this prescription is equivalent to the requirement that even under a severe earthquake no permanent displacements should be tolerated.

On the contrary, the Italian Construction Code provides a relationship, obtained directly from Figure 3, between the maximum permanent displacement (that is, the seismic performance) and a factor $\beta = a_h/a_{max} < 1$. The factor β multiplies the expected maximum acceleration a_{max} to yiled the horizontal acceleration a_h that the code requires for use in a pseudo-static calculation. For a cantilevered or singly restrained wall, a_h coincides with the critical acceleration a_c if the wall is designed on the basis of a pseudo-static limit equilibrium calculation performed with unit partial or global safety factors. In this case, if the relationship of Figure 3 holds true, and if the maximum acceleration is evaluated accurately, the wall will not move more than required, and the maximum bending moments will be evaluated as $M(a_c)$, consistently with the approach discussed in the previous section; this value might be amplified by an over-strength factor to protect the structural elements from yielding.

It must be stressed that the seismic internal forces in the retaining wall are proportional to the critical acceleration, and therefore to the soil strength available for the specific plastic mechanism considered. If for any reason this strength is larger than anticipated, the retaining wall will be subjected to larger internal forces. For instance, if the strength parameters of the soil are underestimated, the computation does not err on the safe side: the pseudo-static analysis will lead to an underestimation of the critical acceleration and therefore the maximum bending moments evaluated as $M(a_c)$ will be too low.
Currently, the Italian Construction Code requires that the pseudo-static design of a retaining wall be carried out applying partial coefficients to the strength properties of the soil. This leads to the design of walls with $a_c > a_h$; hence, the embedment depths are larger than those strictly required for the desired performance, and the seismic displacements are likely to be smaller than expected. But these same walls will have to sustain bending moments proportional to a_c, larger than those computed with $a_h < a_c$. A possible, concise way to overcome this difficulty would be to require unit partial coefficients for the seismic pseudo-static calculations. This would be consistent with the real nature of the seismic pseudo-static calculations, than only apparently deal with the safety with respect to a collapse mechanism, but rather serve the purpose to asses the actual seismic performance of a structure.
However, since the length of a retaining wall may be dictated by requirements other than its seismic performance, it may well be that the actual length of a wall is larger than what would be strictly needed for the desired seismic performance: a technical code should therefore explicitly prescribe that the internal forces in the wall be always computed in the hypothesis that the available soil strength is fully mobilised.
The above concepts are yet to be extended to the seismic design of retaining walls with multiple restrains. In these cases, mobilisation of soil strength may not be sufficient for the development of a plastic mechanism, and several different mechanisms may be possible, depending of the choice of the energy-dissipating elements. For this wall types, significant efforts are still needed in order to identify the more convenient plastic mechanisms and to compute the corresponding values for the critical acceleration. In spite of this, it is believed that the concepts exposed herein still hold their validity: relationships like the one shown in Figure 3 may be used to evaluate the seismic displacements, while the maximum internal forces must be calculated considering the full strength of the energy-dissipating elements of the chosen plastic mechanism.

5 ACKNOLEDGEMENTS

The work presented in this paper is part of the ReLUIS research project, funded by the Italian Department of Civil Protection. The contribution of the Italian Ministry of University, through the research project *Effetto delle azioni sismiche sulla stabilità dei pendii e sul comportamento di scavi e gallerie* (PRIN 2007) is also acknowledged.

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