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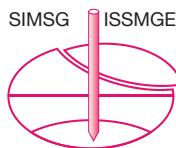
INTERNATIONAL CONFERENCE  
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# Soil-Structure Interaction. Underground Structures and Retaining Walls.



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# PSEUDO-STATIC ANALYSIS OF FLEXIBLE RETAINING STRUCTURES INCLUDING SEISMIC THRUST DEPENDENCY ON WALL DEFORMATION

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**ABSTRACT:** The design of flexible retaining structures is very frequently conducted using simplified soil-structure interaction schemes such as the well known non-linear spring method. With this method, several distinguishing aspects of flexible wall behaviour can be usually captured with reasonable accuracy with regards to construction stages and static conditions. Seismic design, however, still represents a challenging aspect. In this paper some common issues of seismic analysis with the spring method are addressed and a proposal to include seismic effects within a pseudo-static framework is presented. Some examples analyzed with the proposed approach are finally discussed.

## 1. INTRODUCTION

The design of flexible retaining walls is usually based on numerical analyses, aiming at modelling real construction sequence, in particular when several supports are included. One of the most used numerical approaches in this field is the non-linear spring method, a natural extension of the Winkler method traditionally used by practicing engineers in the design of foundation beams and slabs. Even today, in spite of the wide availability of advanced modeling tools, such as 2D and 3D finite element or finite difference codes, the non-linear spring method is very frequently adopted, at least in the early stages of a wall design, because, in authors' opinion, the merits of such simple engineering method still outweigh its intrinsic shortcomings. On the other hand, in seismic design, the non-linear spring approach highlights severe limitations that may, at first glance, discourage its usage. However, once its assumptions and limitations have been clearly addressed, this method may still be helpful in seismic analysis. In this paper, the non-linear spring method as implemented by the authors (Becci & Nova, (1987)) is briefly presented. Then the main assumptions of the pseudo-static methods for seismic design and the so-call performance design approach are reviewed, in the light of its implementation by the non-linear spring approach. A simple modification to the non-linear spring constitutive law is proposed, to automatically include seismic actions on the wall, falling

within selected lower bound thrusts (usually the Mononobe-Okabe (M-O in the following), Mononobe & Matsuo (1929), Okabe (1926)) and an upper bound distribution (to be applied to non yielding walls). Some examples taken from available experimental or numerical benchmarks are reproduced by this approach. The results and limitations of this proposal will be finally discussed.

## 2. THE NON-LINEAR SPRING METHOD

Within a structural or finite element program, the wall is modeled by an array of elastic or inelastic BEAM elements and just lateral behaviour is considered. The interaction with soil is modelled by special non-linear elements at each face, which may be either one-dimensional simple lateral springs applied at each wall node, or more refined interface elements whose stiffness is calibrated by means of continuum finite element models (Pappin et al. (1985)).

In the constitutive model of such elements, lateral stress  $\sigma'_h$  in each spring is initially set equal to the assumed at rest pressure  $K_0 \cdot \sigma'_v$ . Hence the wall is initially subjected to self-balanced distribution of pressures, provided same geostatic conditions are prescribed at each side. Subsequent stages are then studied in which an excavation or the insertion of a prop is modelled, as well as any other modification to previous conditions. Based on lateral wall

deformations  $\Delta$ , soil elements react as shown in Figure 1.

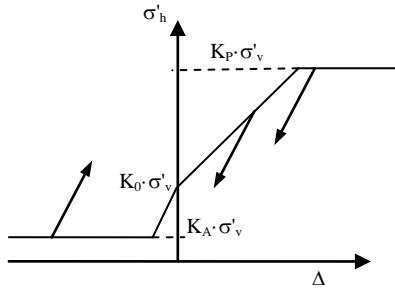


Figure 1. Constitutive law for a non-linear spring (cohesionless soil)

For granular soils, yield limits linearly depend on the effective overburden stress  $\sigma'_v$  via the usual active and passive thrust coefficients  $K_A$  and  $K_P$ , usually determined according to classical Coulomb equations for  $K_A$  and values by Caquot et al. (1973) for  $K_P$ , accounting for wall-soil friction  $\delta$  and dredge line slope.

The spring stiffness depends on the elastic properties of the soil and on the excavation geometry. According to Becci & Nova (1985), at each step, the spring stiffness  $K$  is computed by the following equation:

$$K = \frac{E' \cdot t}{L}, \quad (1)$$

where  $E'$  is the soil Young modulus,  $t$  is the spring spacing and  $L$  is a geometrical parameter accounting for actual wall geometry. This value is different between up- ( $L_M$ ) and down-hill ( $L_V$ ) side:

$$L_M = \frac{2}{3} H' \cdot \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right), \quad (2)$$

$$L_V = \frac{2}{3} (H' - D) \cdot \tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)$$

$D$  is the excavation depth,  $H$  is the overall wall height and  $H' = \min(2D, H)$  (Figure 2).

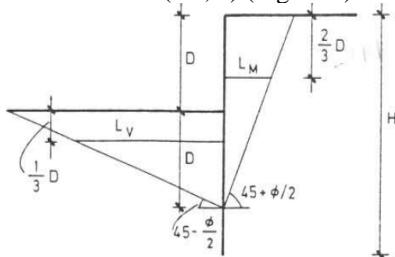


Figure 2. Definition of geometrical scaling factor for non-linear spring stiffness assessment

In essence such geometrical parameters are set equal to the average mobilized soil wedge widths in active conditions (driving side) or passive conditions (resisting side).

The (secant) elastic modulus  $E'$  can be determined by means of usual correlations adopted for other geotechnical problems.

For sandy soils, a reasonable estimate can be obtained by the one of the following equations:

$$E' \approx (2 \div 3) \cdot N_{SPT} \text{ (MPa)}, \quad (3a)$$

or

$$E' \approx (0.8 \div 1) \cdot G_0, \quad (3b)$$

where  $G_0$  is the small strain shear modulus.

In Becci & Nova (1987) a simplified cap logic is also included in order to differentiate loading modulus  $E_{vc}$  from unloading one  $E_{UR}$ . Including such model in a general nonlinear finite element code, several modeling features can be easily considered, such as stage analysis by adding and/or removing parts of the soil as well as ground anchor installation and also the interaction with water. At each analysis stage, balanced conditions are computed by means of Newton-Raphson iterations. The straightforward model definition as well as the simplicity in soil parameters selection makes such kind of procedure a quite attractive option, at least in preliminary design phases, albeit some important design aspects, such as settlements of nearby foundations or global stability checks are not directly available.

### 3. PSEUDO-STATIC SEISMIC ANALYSIS

Pseudo-static seismic analysis is a widely used method used in the design of retaining walls. Stemming from the pioneering proposal by Mononobe & Okabe, several extensions have been published during last decades, in order to encompass most of the aspects that may arise in the practice. In essence a uniform seismic acceleration with horizontal and vertical components is statically applied to the wall and to the soil mass interacting with the wall. The M-O method and its extensions offer a closed form solution of the total soil thrust on the wall, assuming that the soil mass behind yields during seismic event, thus considering a wall subjected to relevant deformations. Following the approximation proposed by Seed & Whitman (1970), the M-O increment to static

active pressures can be applied as a constant pressure distribution given by

$$\Delta p_{E,M-O} \approx 0.375 \cdot \frac{a}{g} \cdot \gamma \cdot H, \quad (4)$$

where  $H$  is the wall height,  $\gamma$  the average backfill unit weight and  $a/g$  the normalized seismic acceleration. Eq. 4 strictly holds for a dry granular fill with a friction angle  $\phi'=35^\circ$  and a soil-wall angle  $\delta=1/2\phi'$ .

Other methods, including the widely used Wood method (Wood, 1973)) provide the seismic increment of the soil thrust on a wall which is essentially rigidly restrained so as to prohibit remarkable wall deformations. Such increment is represented by a constant pressure distribution given by:

$$\Delta p_{E,Wood} = \frac{a}{g} \cdot \gamma \cdot H, \quad (5)$$

Comparing the formulations above, it can be realized that the seismic thrust increment for rigid walls (Eq. 5) may exceed the M-O increment (Eq. 4) by a factor greater than 2.5. Therefore it is important to select the appropriate approach, in the light of wall behaviour under consideration.

Beyond the method adopted in computing seismic pressures, a most important issue arose even since early '70s: i.e. the selection of an appropriate design acceleration  $a_c$  with respect to the prescribed maximum seismic acceleration  $a_{max}$ . Thanks to the fundamental works by Richard & Elms (1979), Whitman & Liao (1985) and later works, the concept of design acceleration  $a_c < a_{max}$  related to selected wall performance (acceptable deformation) rather than to maximum site intensity was clearly established. Since then, most design standards such as Eurocode comply with this approach. Research efforts have been devoted in extending this method that was originally proposed for gravity walls, to other kinds of retaining structures, such as cantilevered or propped bulkheads. Since the performance design method requires the calculation of a desired ultimate capacity, just rigid-plastic methods, such as the Blum method or similar extensions (e.g. Conti & Viggiani, (2013), Callisto (2014)), are essentially applicable. In other words, reduced design accelerations  $a_c$  given by a performance design approach must be applied only if seismic increments are determined by a method like Eq. 4, but not by

Eq. 5 or any other elastic method. In this respect, it is clear that, for many kinds of walls, such as multi-propped bulkheads, the performance (or capacity) seismic analysis with pseudo-static approach requires particular care and further research.

Finally, within the performance design framework:

$$\frac{\Delta p_{E,Wood}}{\Delta p_{E,M-O}} \approx 2.67 \cdot \frac{a_{max}}{a_c}, \quad (6)$$

Therefore active seismic thrusts may drop down to a quite low percentage of elastic thrusts: for example, taking  $a_c/a_{max} \approx 0.70$ , according to Eq. 6, active thrust is just 26% of elastic thrust.

#### 4. PSEUDO-STATIC SEISMIC ANALYSIS BY THE NON-LINEAR SPRING METHOD

##### 4.1. General

Once the construction process has been completed, seismic conditions are usually modelled according to one the two following alternatives:

- A. the active and passive coefficients are modified to account for seismic effects or;
- B. just passive coefficients are modified whereas, on the driving side, an appropriate distribution of external pressures is applied, whose resultant corresponds with the expected seismic thrust increment: for example Eqs. 4 or 5 may be used, depending on the expected wall behaviour.

Approach A is usually not recommended since it is just applicable to very flexible walls, in which active conditions are reached at the end of excavation process and in general may not sufficiently reproduce expected seismic actions. Approach B, on the contrary, allows the designer to keep under his/her control the seismic actions which, however, do not depend on wall deformations.

##### 4.2. The proposed procedure

As an alternative to such conventional approaches, the following procedure is proposed aiming at automatically include intermediate seismic conditions, based on actual

wall behavior. Consider a soil region in the uphill side on the retaining wall. In a very short time lagging in between seismic wave arrival and the actual wall response, incremental wall deformations may be assumed to be negligible; i.e. the wall may be considered as very rigid, for a while. During such very transient conditions, the seismic thrust increment may be estimated by a rigid approach, such as the Wood method (Eq. 5). Due to such incremental pressure spike, previous equilibrium conditions are violated. Therefore additional deflections must develop, in order to reestablish equilibrium conditions, which are pursued by means of the usual iterative process, exactly as in any static stage.

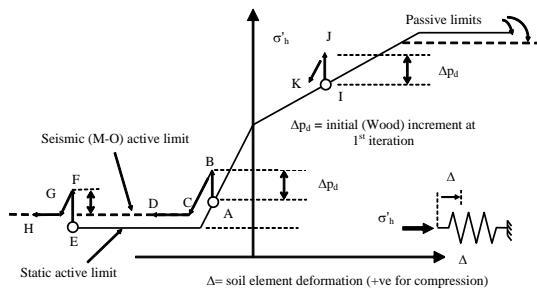
The proposed algorithm requires the following two modifications to the standard calculation scheme:

1. In the first iteration of the seismic stage, in any uphill soil element only, the effective lateral pressure is increased by the rigid (Wood) pressure increment (Eq. 5): note that such stress increment is assigned with no strain increment, thus it can be revised as an inelastic increment.

2. During further iterations, strain increments are allowed and, at the same time, active and passive limits are updated to the seismic values as discussed later.

Through the iterative process, the initial seismic stress increment may diminish or even vanish at all, but the appropriate seismic yield conditions are ensured, at least.

In figure 3 some particular stress paths for uphill soil elements are outlined, during such automatic seismic procedure stage.



A-B-C-D path corresponds with an “elastic” uphill soil region in which the lateral stress was released, in a previous static stage, from at rest conditions, yet not sufficiently to reach active limit state (pt A). Initial rigid seismic stress increment is represented by A-B segment. Further stress evolution toward seismic active conditions is represented by B-C-D path, in which B-C is the “elastic” release part, whereas C-D represents the development of seismic plastic strains.

E-F-G-H path represents an uphill soil region that reached active static conditions (pt. E) prior to earthquake. Initial rigid seismic stress increment (E-F) corresponds with a temporary elastic reloading path. Subsequent elastic strain release is represented by F-G segment, whilst seismic plastic deformation development is associated to segment G-H

I-J-K path: an uphill soil element is represented, which was actually pushed inward rather than being released ( $\Delta > 0$ , point I): initial rigid seismic stress increment is I-J segment, whereas subsequent unloading is J-K.

Active and passive limit seismic conditions should be computed according to general criteria (for example using M-O equations).  $K_p$  should be determined so as to account for wall-soil friction, based on safe formulations (e.g. Lancellotta (2007), Soubra (2000)).

#### 4.3. Discussion

According to this procedure, the computed seismic pressures on the driving side fall between a minimum (active) and a maximum (Wood) distribution. This feature may be considered the most relevant advantage of the procedure because all wall typologies may be analyzed without preliminary (and arbitrary) assumptions on their mechanical behavior. Of course, it also suffers from some limitation, including the following:

- the decrease rate from Wood to active conditions is governed by the same stiffness considered in static calculation;
- the position of the overall active seismic thrust is essentially the same as the static active thrust.

In the light of the performance design method, on authors' opinion, this approach should be used just when the allowed unrecoverable

deformation are very small or even negligible, i.e. when the wall is designed to behave (almost) elastically during seismic conditions.

## 5. EXAMPLES

### 5.1. Proppped bulkhead - centrifuge model

The propped wall PW2 studied by Conti et al (2012) by means of a centrifuge reduced scale model is considered. Prototype dimensions are summarized in Figure 4.

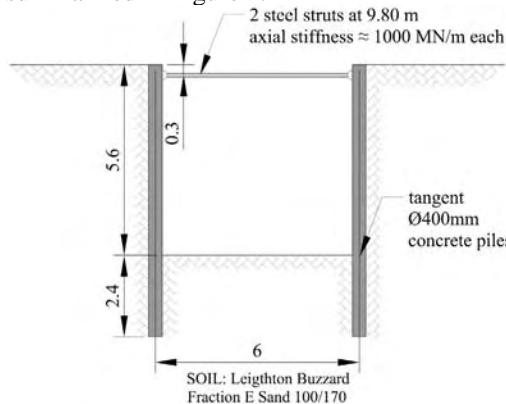


Figure 4. PW2 model (dimensions in [m])

The following soil parameters have been considered:

$$D_r = 42\%$$

$$\gamma_d = 14.37 \text{ kN/m}^3$$

$$E_{vc} (\text{MPa}) = 50 \cdot \left( \frac{\sigma_h}{98 \text{ kPa}} \right)^{0.5}$$

$$\phi = \begin{cases} 38^\circ & \text{uphill side} \\ 35^\circ & \text{downhill side} \end{cases}$$

$$e_0 = 0.84$$

$$\phi_{cv} = 32^\circ$$

$$E_{UR}/E_{vc} = 2$$

$$\delta/\phi = 0.30$$

In order to account for a likely reduced soil density inside excavation (see discussion below), a reduced friction angle ( $35^\circ$ ) and reduced elastic moduli (50% of reference values) are assigned to soil elements at the wall toe.

In the experimental study, the model was excited by a set of five contiguous wave motions, modelling five seismic excitations (EQ1 to EQ5), with increasing intensities from  $0.21 \text{ g}$  up to  $0.41 \text{ g}$ .

Thanks to a set of complex instrumentations, wall displacements, bending moments and strut

forces have been recorded at several model positions.

The following sequence is reproduced:

- stage 0: at rest conditions are established;
- stage 1: a small excavation is performed down to strut position;
- stage 2: the strut is inserted and the excavation is lowered down to final level (results at this stage are compared with static results in Conti et al (2012));
- stage 3: EQ1 seismic conditions are applied.

It should be noted that a reduced value of peak acceleration of the input motion must be usually considered in a pseudo-static analysis, all the more because, in this case, numerical results are compared with the available results at the end of seismic excitation. In this case, a pseudo-static acceleration equal to  $0.15 \text{ g}$  is considered, representing  $\sim 70\%$  of maximum peak EQ1 excitation.

In Figure 5, numerical and experimental results are compared. Bending moments and prop forces are quite well matched by the numerical model, whereas the predicted wall movements are different from the measured ones, in both static and seismic conditions.

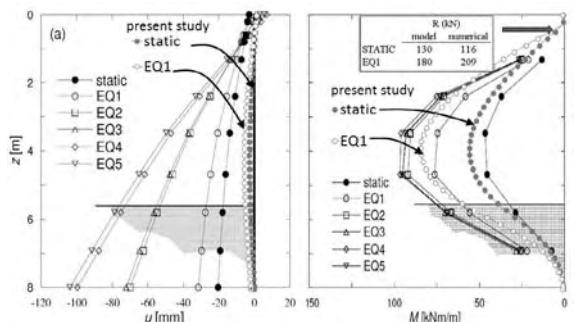


Figure 5. PW2 model results comparison

However, it should be noted that measured deflections in centrifuge tests, in initial static stage, most likely suffer from a insufficient local sand densification, thus displaying some uneven initial tilting not reproduced by our numerical analysis. In this respect, in Conti (2010), such issue is thoroughly addressed.

As for seismic incremental deformations, it should be observed that unrecoverable deformations are accumulated throughout the real seismic event, when applied acceleration exceeds the threshold acceleration  $a_c$  related to wall capacity; of course, such contribution

could not be directly reproduced by a pseudo-static analysis, even if much more advanced numerical tools were used. Some a posteriori correction to pseudo-static deformed shape may be possibly added, depending on maximum wall capacity as compared with maximum seismic acceleration, but such topic is beyond the scope of this paper.

Nonetheless, in spite of its simplicity, the non-linear spring algorithm can predict the most important results for design purposes reasonably well. However it must be noted that since in this case active limit conditions are reached along all the wall, seismic results are poorly affected by the amount of Wood pressure applied at first seismic iteration.

### 5.2. A numerical exercise - seismic analysis of a real wall

A flexible retaining wall benchmark is considered in the following, as a starting point to perform some parametric seismic analyses with the proposed algorithm.

The multi-anchored retaining wall described by Schweiger (2002) is shown, in Figure 6. Such problem, which is selected as a benchmark by some of the most popular numerical modelling tools, such as FLAC (Itasca (2011)) and PLAXIS (Brinkgreve et al. (2010)), is limited to static conditions.

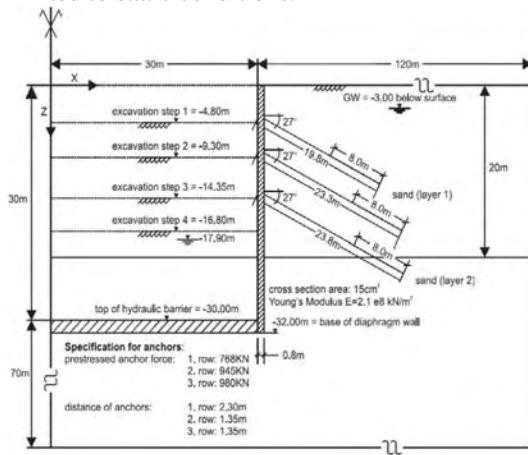


Figure 6. Real wall benchmark (adapted after Schweiger (2002))

A non-linear spring analysis of the constructions stages is performed first, and the obtained results are compared with the reported measures. The following soil properties are assumed (see Schweiger (2002)):

$$E_{vc} (\text{kPa}) \approx 10000 \cdot \sqrt{z[\text{m}]} \quad z \leq 20\text{m}$$

$$E_{vc} (\text{kPa}) \approx 50000 \cdot \sqrt{z[\text{m}]} \quad z > 20\text{m}$$

$$E_{UR}/E_{vc} = 1.5 \quad \phi = 35^\circ \quad \delta = 0.7\phi$$

Dewatering operations are reproduced by assigning a lower permeability coefficient to the 2 m thick hydraulic plug inside excavation (Fig. 6) and computing the pore pressures by means of a simplified mono-dimensional seepage scheme. Ground anchors are modelled as linear springs whose stiffness is given by the axial stiffness (EA) of the steel stem divided by the deformable length, set equal to the free length plus 50% of the grouted length.

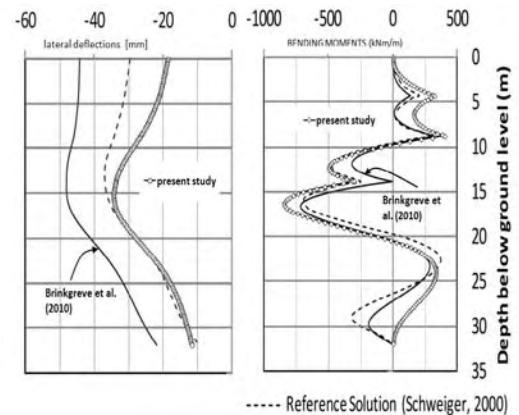


Figure 7. Real wall benchmark (adapted after Brinkgreve et al. (2010))

In figure 7, computed and measured deformed shapes are compared, showing that the numerical model reproduces quite well the observed behaviour, unless in the top part where some reduced anchor stiffness is highlighted by the measures. This effect is likely related to a limited upper anchor length whose foundation is somehow disturbed by wall movements: such aspect cannot be easily reproduced by a non-linear spring method, whereas may be highlighted by a continuum model (Itasca (2011)).

Bending moments are also compared with the values reported in the cited reference. Both absolute values and overall pattern are fairly in agreement, on the safe side. Thus such non-linear spring model can be considered a realistic model of the observed wall behaviour during construction stages, and a valuable starting point for numerical experiments including seismic conditions.

In this respect, seismic experiments outlined in Table 1 are discussed.

Table 1. Numerical experiments on real wall

experiment	$a_c/g$	Initial Wood p.	Lateral restraints
EXP1	0.15	Yes	Soft
EXP2		No	
EXP3		Yes	Rigid
EXP4		No	

Soft lateral restraints correspond with ground anchors only, whereas rigid restraints are represented by very stiff lateral supports applied at elevations shown in Figure 8, along with ground anchor removal. Bending moments for various assumptions are also shown in Figure 8.

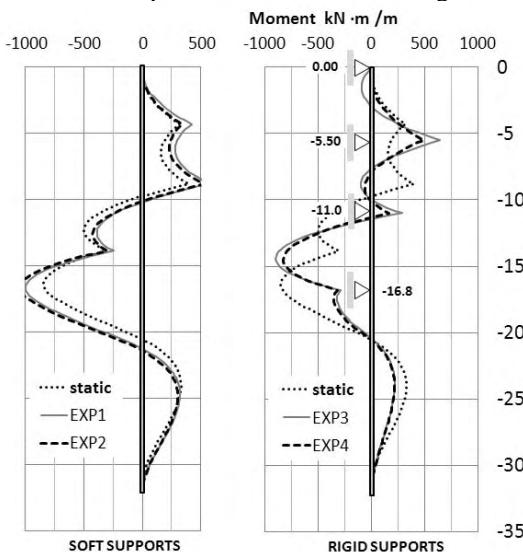


Figure 8. Numerical experiment results

With soft supports, EXP1 and EXP2 results are almost the same since active seismic conditions are easily restored so as to make the initial Wood pressure effect negligible.

With rigid supports, initial Wood pressures (EXP3) significantly increase bending moments and support forces, by ~30% in the upper part and ~9% in the middle part, with respect to an ordinary M-O analysis (EXP4). Support reaction differences are even larger (Table 2).

In practical problems, similar to this example, representing a quite typical configuration for deep excavations in urban areas, the proposed algorithm produces safer predictions with respect to traditional simplified analysis limited to M-O seismic pressures.

Table 2. Rigid support reactions (kN/m)

EL	EXP3	EXP4
0.00	96	6
-5.50	823	520
-11.00	1065	792
-16.80	636	558

On authors' experience on similar examples, this observation can be considered quite general, in agreement with similar findings by other authors (e.g. Psarropoulos et al (2005)).

This statement holds for moderate seismic conditions and for walls designed according to usual criteria, with regards to the most relevant aspects such as the length of ground anchors, which should be normally designed to ensure expected resistance and stiffness in both static and seismic conditions.

## 6. CONCLUSIONS

A simple algorithm to allow pseudo-static seismic analysis of flexible retaining walls with the non-linear spring method is presented.

As most pseudo-static methods, this approach has a reasonable ability to capture internal forces in soil, as well as in wall and props, but fails in computing physical deformations when relevant plastic deformations are progressively accumulated during seismic events: this circumstance is particularly evident in case of cantilevered or singly propped walls and must be clearly taken into account in final design.

However, on authors' best knowledge, this limitation is also present in any kind of pseudo-static analysis, even using more advanced numerical models.

Based on the examples studied with this approach, the following concluding remarks can be proposed:

- Multi-propped walls with deformable intermediate supports such as ground anchors are normally so flexible to reach active limit state at driving side at the end of excavation as well as during seismic events. In this case both the proposed approach and the classical M-O pressure distribution can be used in practical designs.
- For multi-propped walls with rigid supports, active seismic pressures may be inappropriate,

whereas fully elastic solution such as the one by Wood (1973) may be too conservative. The proposed algorithm can provide a more reasonable estimate of seismic conditions falling between lower and upper bound solutions.

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