Structural response of buildings on mountain slopes subjected to earth pressure under seismic conditions

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SUMMARY
In recent years, it has become a common practice in growing cities located in mountainous regions to undertake the construction of buildings of considerable height and large base areas on slope terrains. These conditions lead to a situation where the lower floors end with only the side facing the top of the hill located beneath the ground surface and, thus, with a slope leaning on that side, which a priori seems inconvenient because of the asymmetric lateral earth pressure involved and the extension of the contact zone between two bodies of such different stiffness and responses as are a building and a slope. Thus, the analysis of the effects that earth pressure can produce under seismic conditions on the behaviour of such structures located on slopes is herein addressed. Finite element models considering various support conditions, solution schemes, structure heights, soil types and ways to account for soil pressure, are implemented. Results indicate that deformations as well as internal forces do increase for elements located above the crown of the slope both because of its presence and in contrast with those estimated by current analysis practices.

Keywords: buildings on slope, soil – structure interaction (SSI), finite elements method (FEM), seismic analysis.

1. INTRODUCTION

Soil-structure interaction (SSI) is an area of increasing popularity that has been applied for several years to cases in which the consideration of contact between a structure and the soil in which it lies could result in important changes of the system's behaviour. Growing literature on the matter describes numerous and different situations, including cofferdam construction and dike projects (Van den Berg & Visschedijk 1991), seismic analysis of dams considering foundation flexibility and nonlinearity (Burman et al. 2008, Huang & Zerva 2008), analysis of soil – pile interaction (Ahmadi 2008, Brandenberg 2004, Chiou & Yang 2008), soil – structure interaction between bridge abutments and surrounding soil (Koskinen 2005), dynamic behavior of earth – retaining walls and sheet – pile walls (Tsompanakis 2008, USACE 1994, Pathmanathan 2006) and design of immersed tunnels (Lyngs 2008), just to mention a few.

However, as is natural, there are still many cases that have not yet been analyzed from an SSI point of view. One of them arises from the fact that in recent years, the growth and expansion of several cities located in mountainous regions around the world have been continuously pushing the urban limits from the original plain terrains towards the hillsides that surround these populations (Fig. 1a). As a consequence, it has become a common practice to undertake in such slopes the construction of, not just 1-or-2-floor houses, but of buildings of considerable height and large base areas that make it necessary to use excavations which, coupled with the slope of the ground, lead to the situation shown in Fig. 1b, where the lower floors are partly located beneath the ground surface and partly above it, consequently subjected to lateral earth pressure on just one side.

While this is not an isolated case rather than a recurring practice around the globe, and considering that experience in recent earthquakes shows that structures on sloping ground and retaining sloping backfill are at significant risk of failure due to a variety of factors that deserve careful scrutiny and experimental work (Atik & Sitar 2007), little study has been carried out on the specific effects that this asymmetric earth pressure would cause on the response of such structures under seismic conditions. In his article on dynamic earth pressure of soils against basement walls, Navarro (1981) compared different calculation methodologies and found that there was no exact solution of the problem, but that it was possible to accept the Mononobe – Okabe method (Mononobe and Matsuo 1929) as the lower limit and Wood`s elastic analysis (Wood 1973) as the upper one. He also concluded that finite element modelling of both the ground and the wall was the most appropriate calculation tool for the correct interpretation of the problem. Rodriguez (2009) examined a particular case of a building with a slope leaning on one of its sides, considering a fixed based versus a flexible foundation modelled with springs and including or excluding a static distributed load to account for the earth pressure. Results indicated that lateral soil pressures could generate significant increases in axial loads, moments and deformations of structural members located up to two floors above the height of the backfill.

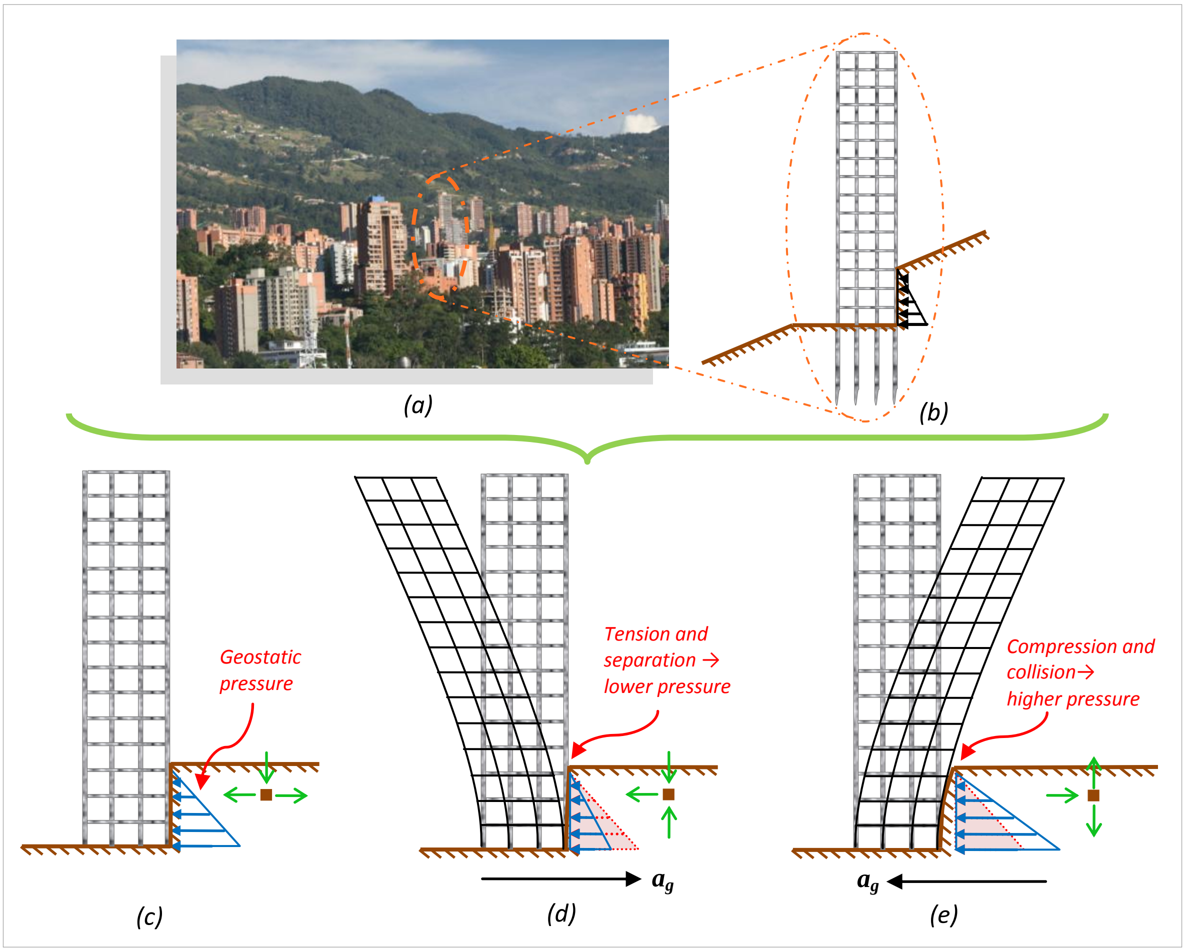

Figure 1. Outline of the situation under study: (a) Buildings on sloping ground in Medellín, Colombia; (b) Vertical plane showing the asymmetric lateral earth pressure against the building; (c) static conditions; (d) movement of the structure in the direction of soil pressure and (e) against the direction of soil pressure.

Facing this lack of understanding and research about the case described, this study was developed in order to begin elucidating possible ways in which the dynamic response of structures built on sloping ground with soil pressing against one of their sides could result altered in terms of deformations and internal forces, focusing attention towards three presumably adverse conditions: (i) the interdependent behavior and coupling between the soil and the structure, both of them with so different mechanical characteristics; (ii) the asymmetry in loads and geometry due to the combination of the slope of the ground surface with the use of excavations (Fig. 1c); and (iii) the sequential development of two modes of vibration of the structure, one when it moves away from the backfill, freely drifting with respect to its base and subjected to a reduced earth pressure (Fig. 1d), and the other when it deflects towards the backfill, in a restrained manner due to the presence of the slope and under an increased soil pressure (Fig. 1e), the former being interrupted by an abrupt collision between the building and soil when inertia pushes the structure against the backfill.

2. STRUCTURE CHARACTERIZATION

As a first step towards the numerical analyses planned, the type and dimensions of the structural system to evaluate were to be defined. Hence, two reinforced concrete moment frame residential buildings were proposed, each of them with a different height, one 14-story building (M) and one 20-story structure (A). Moment frame system was selected due to the fact that it is still the most commonly used and because its greater flexibility compared with shear walls systems would make it the most likely affected by the asymmetrical slope's pressure.

Parameters chosen were based on typical values taken from various real designs of structures built on sloping ground and according to design provisions specified in the current Colombian earthquake resistant building code NSR-10. Fig. 2 presents the general structural floor plan of the example buildings, which consists of a single span between two lines of columns arranged perpendicular to the backfill leaning against the structure, with a symmetrical distribution with respect to both orthogonal axes, free of any irregularities either in a plain view or in elevation and with corresponding dimensions listed in Table 1. Numerical analysis were limited to two-dimensional models of a frame axis aligned parallel to the direction of the slope's earth pressure, as indicated in Fig. 2.

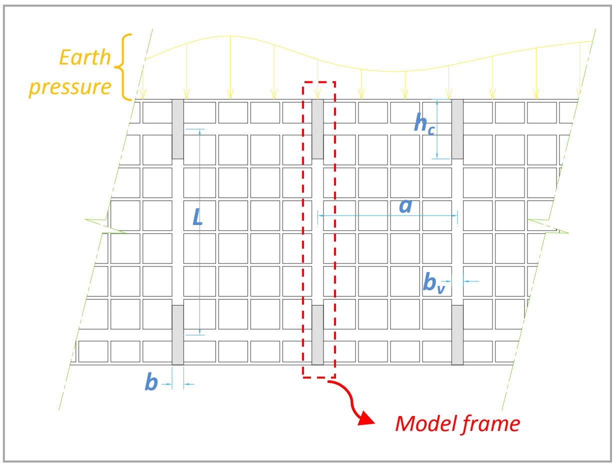

Figure 2. Typical floor plan of the analyzed structures.

Table 1. Dimensions of the example structural systems.

Parameter	Unit	Structure M	Structure A
No. floors	#	14	20
No. floors under soil pressure	#	3	3
Story height (H)	m	2.60	2.60
Columns cross section:			
Width (bc)	m	0.25	0.25
Length (hc)	m	1.50	2.00
Beams span (L)	m	8.00	8.00
Beams cross section:			
Width (bv)	m	0.25	0.25
Length (hv)	m	0.40	0.40
Tributary width (a)	m	5.00	5.00

Provided that the results obtained for building A and building M should be susceptible of comparison and contrast regardless of their differences in heights, their behavior under lateral loads should be similar. As observed form Table 1, all the design parameters (dimensions and materials) were set to be equal, except for the height (number of floors) and the length of the columns' cross section (h_c). These two were varied so as to achieve the greatest possible similarity in the responses of both buildings, as depicted in plots of (*Horizontal displacement / Total height) vs (Height/Total height)* and (*Story drift) vs. (Height/Total height)* (Fig. 3). These responses were calculated through modal spectrum analysis using perfect base fixing, rigid slabs, live loads of $LL = 2.0\ kN/m^2$, dead loads of $DL = 5.5\ kN/m^2$ and the acceleration spectrum specified by the NSR-10 code, with parameters $Aa = 0.15$, $Av = 0.20$, $Fa = 1.60$, $Fv = 2.00$ and $I = 1.00$. Final models of the structure built in SAP2000 are shown in Fig. 3.

3. NUMERICAL ANALYSES

Several numerical analyses were carried out in order to compare results obtained from a response spectrum analysis as is currently done by practicing engineers in routine project designs, in contrast with those derived from a more complex and realistic nonlinear, time-history, soil-structure model. This approach was guided by the ultimate goal of: i) determining if there were any significant changes in the structural dynamic response of a building subjected to a slope lateral pressure during a seismic event when obtained from a more refined calculation and ii) consequently establishing if current analysis practices applied in everyday projects are deemed adequate for estimating displacements and internal forces under the specific conditions of this study with safe design purposes.

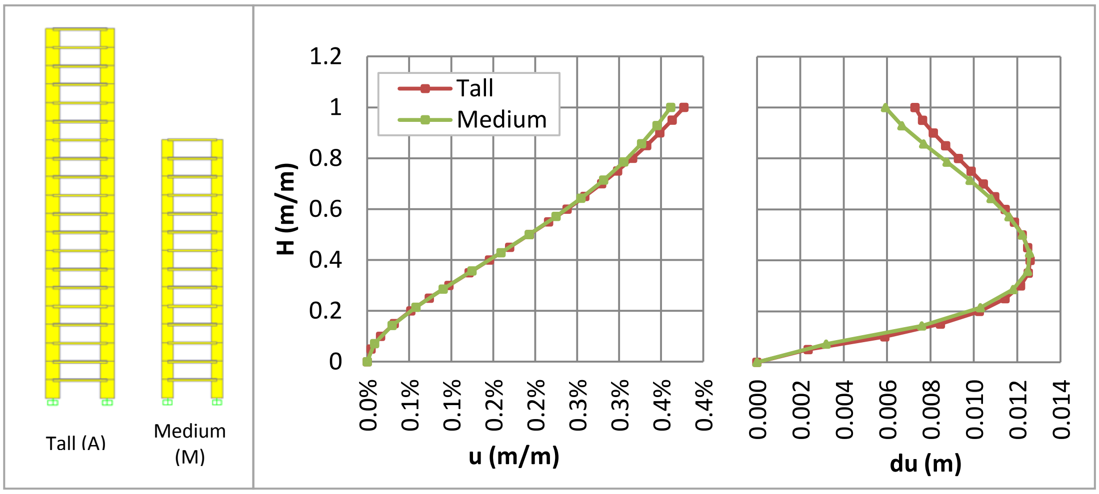

Figure 3. (Left) Two-dimensional frames modeled for buildings *A* and *M*. (Right) Displacement, u, and story drift, du, for both frames resulting from a modal response spectrum analysis.

With the goal of achieving this objective, a sequence of increasingly complex study cases was developed, starting with the model customarily adopted by structural engineering practitioners, which consists in adding the effects of a response spectrum analysis, with perfect fixings assigned to column bases, plus those derived from a static analysis of the structure subjected to a lateral line load applied along the columns of the lower floors, with a rectangular uniform distribution of magnitude:

$$E = 0.65\gamma H K_a \quad where \quad \begin{aligned} \gamma \ &= Soil\ unit\ weight\ [kN/m^3] \\ H \ &= Backfill\ height\ [m] \\ K_a &= Active\ earth\ pressure\ coefficient \end{aligned}$$

The sequence of cases, aimed at confirming the correctness of the final refined model, consisted of a series of four steps as illustrated in Fig. 4, each of them including one out of three modifications of variables required: i) changing modal spectrum response for time history analysis, ii) modifying support conditions from a fixed base to a simplified SSI model accounting for the flexibility of the foundation and iii) replacing linear static earth pressure distribution with a dynamic variable force derived from soil - structure interaction between the building and the backfill.

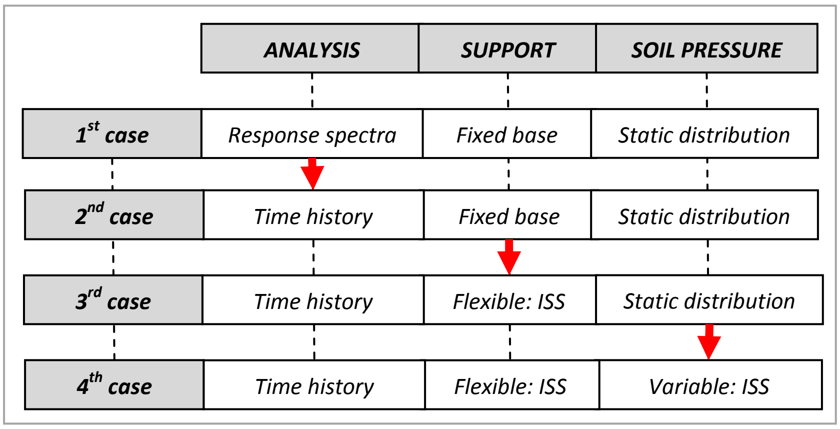

Figure 4. Sequence of study cases considered.

4. FINITE ELEMENTS MODELS

Numerical analyses previously described were implemented using the commercial finite element package Abaqus 6.10. Two-dimensional FEM models corresponding to each case of study and their schematic sequence are shown in Fig. 5.

Structural concrete members, i.e. beams, columns, foundation beams (0.4 × 0.7 m cross-section) and piles (1.2 m in diameter and 10 m in length), were modeled using 2-node linear beam finite elements (B21 type) and a linear elastic constitutive model, with Poisson`s modulus of $\upsilon = 0.2$ and an elasticity modulus of $E = 21.5$ GPa for beams, foundation beams and piles and $E = 24.9$ GPa for columns.

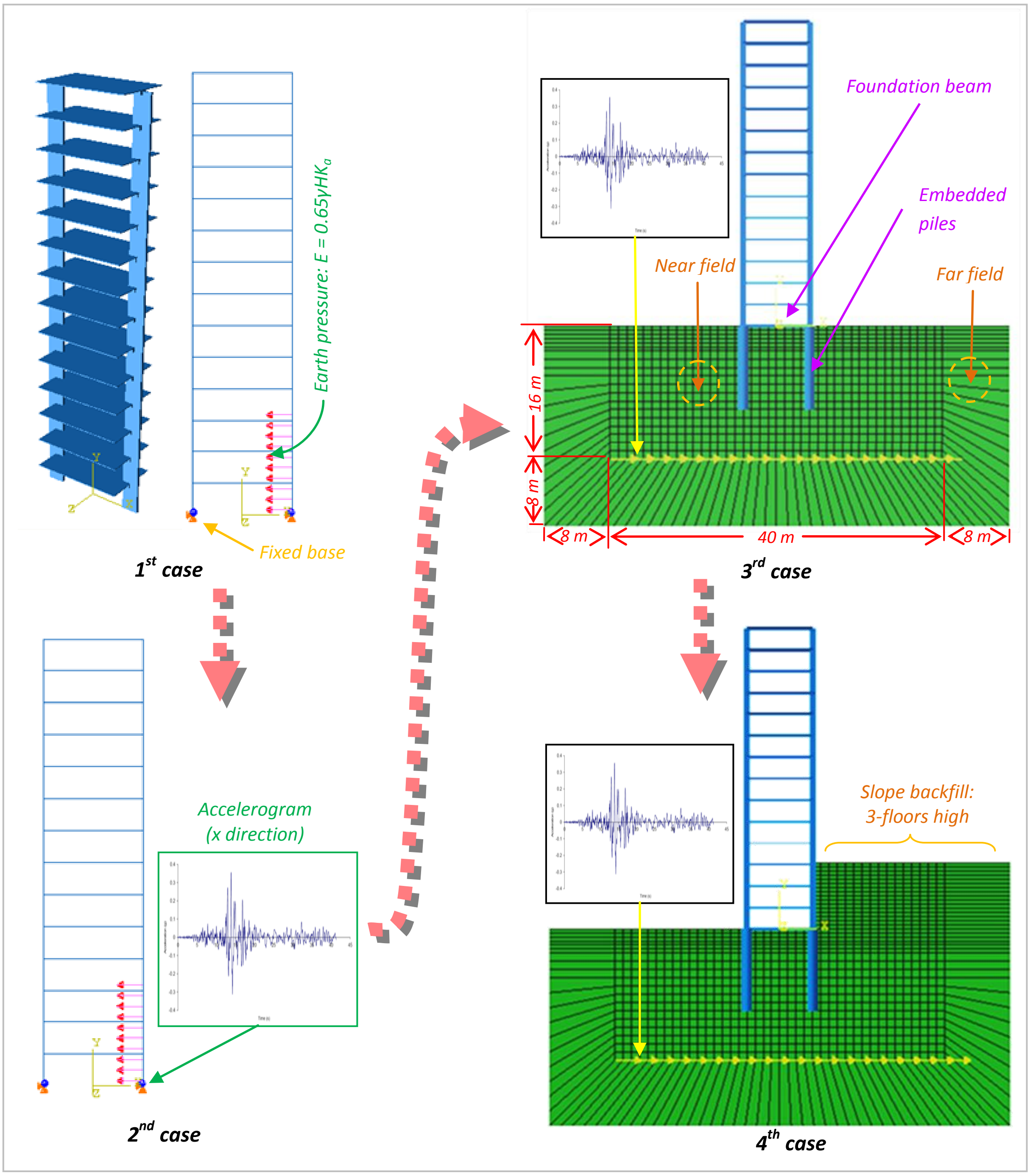

Figure 5. Sequence of finite elements models implemented.

For all of the cases, two different foundation soils were included en each analysis in order to evaluate the effects of soil properties. Mechanical properties of both are listed in Table 2. In cases 3rd and 4th, to adequately represent its unlimited nature, soil was divided into two subdomains: the near-field region and free far-field region (see Fig. 5), approximating near field region with 4-noded two-dimensional plane-strain elements (CPE4) and far-field with 4-noded two-dimensional infinite elements (CINPE4), so as to provide for absorbing boundaries with which to simulate radiation and avoid "box effect".

Table 2. Soil properties.

Parameter	Unit	Soil type	
		Good (B)	Poor (M)
Unit weight (γ)	kN/m^3	20.00	18.00
Cohesion (c)	kPa	45.00	20.00
Friction angle (ϕ)	°	35.00	25.00
Elasticity modulus (E)	MPa	22.00	6.90
Poisson`s modulus (ν)	-	0.30	0.45

In addition, near-field region was assigned an elasto-plastic Mohr-Coulomb law, assuming a non-associated flow criterion with $\psi = \phi/2$, while far-field conditions were represented by a linear elastic law with the same elastic parameters as those of the near-field. It shall be observed that the backfill was modeled with a horizontal surface for simplification purposes.

As for seismic excitations, two types of sources were used (Fig. 6): for spectral analyses, seismic stimulus consisted of the acceleration spectrum proposed by the NSR-10 code, with parameters previously defined in Section 2; and for transient analysis, input corresponded to the first 20 seconds of the acceleration record at El Centro station for the EW component of the Imperial Valley 1940 earthquake, scaled by a factor of 1.15 in acceleration and by a factor of 1.25 in time so that its acceleration spectrum was similar to the one defined according to the NSR-10 code. As shown in Fig. 5, acceleration time history used in the 2nd case was applied to the base of the columns, while in the 3rd and 4th cases it was applied directly to the base of the near field, i.e., in the bottom border that separates the near-field from the far-field regions.

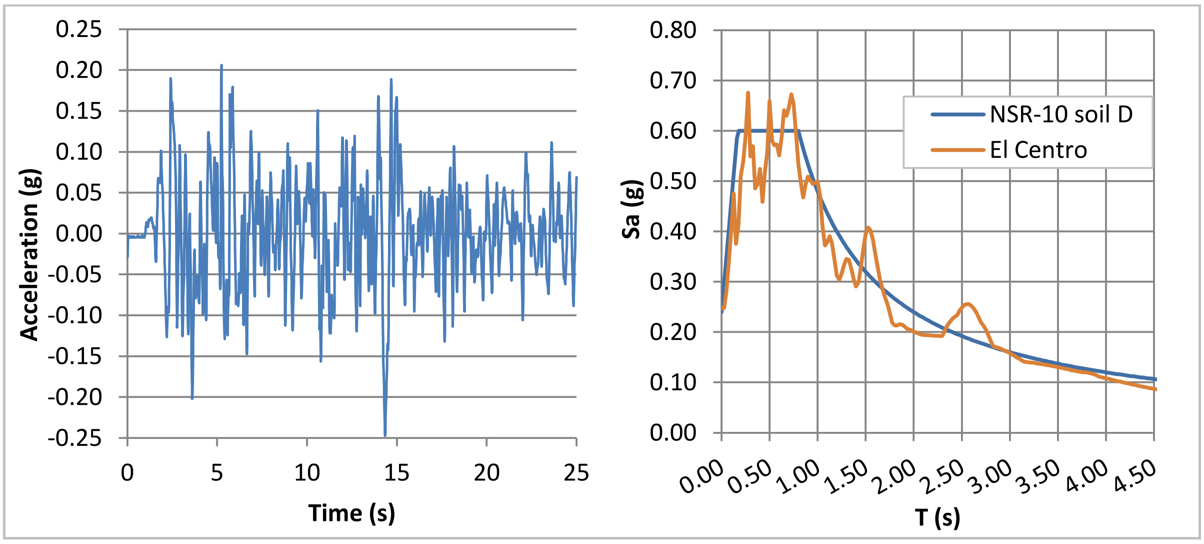

Figure 6. Seismic inputs used: (Left) El Centro EW accelerogram. (Right) Acceleration spectrum according to NSR-10 and compared to that of El Centro EW.

Finally, in cases 3rd and 4th, soil medium side boundaries were constrained against horizontal direction and bottom boundary was constrained against both horizontal and vertical directions. Piles were completely embedded in the soil. The interface between the structure and the slope, as well as between the foundation beam and the ground, was simulated by means of a frictionless, "hard" contact, surface-to-surface contact method, to allow for separation in tension and ensured compatibility in compression. And prior to transient analysis, a static analysis was carried out to estimate geostatic stress field, supposing that retention of the backfill by means of the structural system was such that there were no initial lateral deformations.

4. RESULTS AND DISCUSSIONS

Once the complete series of numerical analyses were finished, results for the full sequence were examined so as to confirm that values obtained for the different variables characterizing the buildings behavior (displacements and internal forces) were within normal structural engineering ranges of magnitude and that no unusual effects had arisen, thus making it possible to accept the general correctness and validity of the numerical models implemented and the verisimilitude of the results, as well as their adequacy for comparison purposes. In order to facilitate the presentation of results, analyses were identified with a three-characters code, where the first one represents the case study ordinal (1st, 2nd, 3rd or 4th), the second stands for the building type, medium (M) or tall (A), and the third one specifies the soil type, good (B) or poor (M).

As stated at the beginning, one of the presumably adverse conditions of the problem under study was that because of the presence of the slope, the structure would tend to respond, somehow simultaneously, in two different ways depending on the direction of displacement, the first consisting of a free drift with respect to its base when moving away from the backfill, and the second corresponding to a deflection restrained at the lower floors when moving against the backfill. Such hypothesis is readily examined and verified by plotting maximum rotations along the columns for both directions of movement. As seen in Fig. 7, for the lower 40% of the building height, rotations of the columns when the structure moves against the slope are clearly inferior than those resulting for movement in the opposite direction, indicating a restriction due to the backfill pressure. However, because inertia contributes to generate almost the same total roof displacement (less than 10% difference), as shown in Fig. 7, rotations towards the backfill increase and intensify for the upper 60% height so as to achieve such similar displacement in a reduced length.

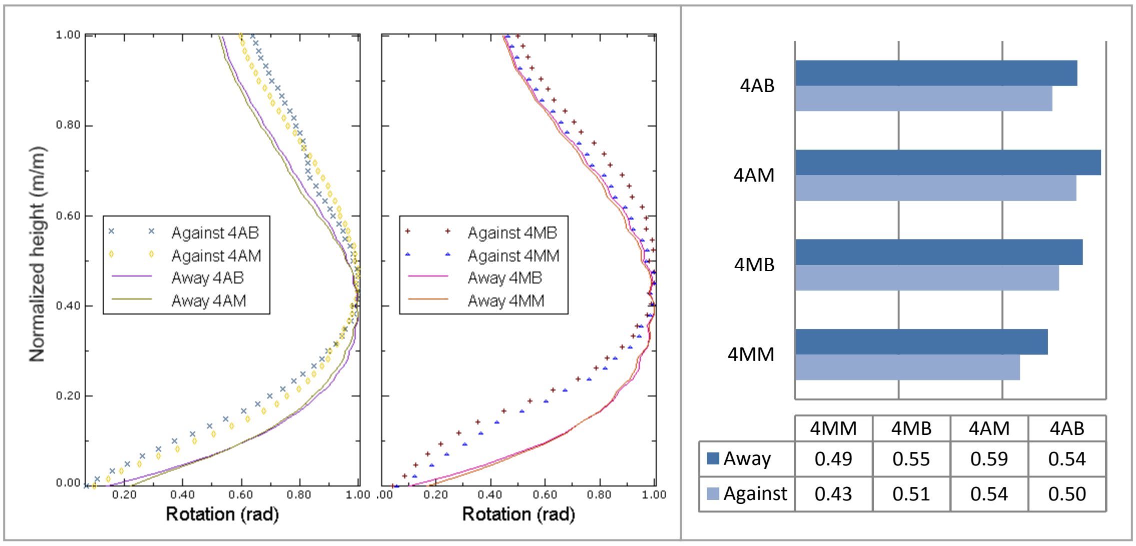

Figure 7. (Left) Maximum rotations along the columns. (Right) Maximum roof displacement.

This in turn produces a similar response of the structures in terms of shear forces and moments, i.e. lower values for the first floors and slightly higher forces and moments at the upper floors when the displacement is towards the backfill as compared to those when the movement is in the same direction of soil pressure (Fig. 8). This general trend is observed for both buildings and both soil types, and is especially appreciable at the lower half of the structure.

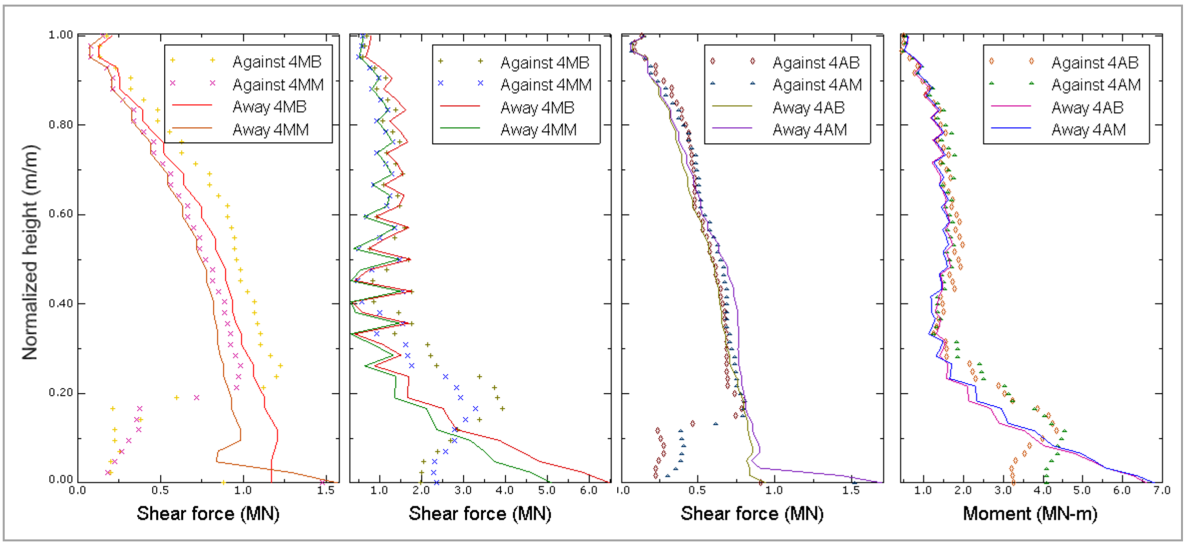

Figure 8. Maximum shear forces and overturning moments for both directions of the structure's movement.

Thereafter, variations were looked for between the response obtained from the first study case in the sequence (response spectrum analysis) and the more detailed model corresponding to the fourth case study, in order to establish if current practices (1st case) are appropriate enough so that structures designed based on their results could safely withstand demands arising from the refined analysis, or if on the contrary there exists any improperly considered aspect when using such methodology.

Fig. 9 shows the maximum displacement envelopes along the columns, with displacements divided by the corresponding total height of the structure. Shapes of these drift curves do not present major differences regarding distribution between the two models contrasted. But they do make it evident that displacement values for the more detailed 4th case are considerably higher (up to 100% increase) than those estimated by current practices. This behavior appears perfectly reasonable as a natural consequence of the inclusion of a flexible foundation in replacement of the fixed base used for the response spectrum case, because in this way the movement of the structure is less restricted, thus being able to rotate at the base and reach greater amplitudes.

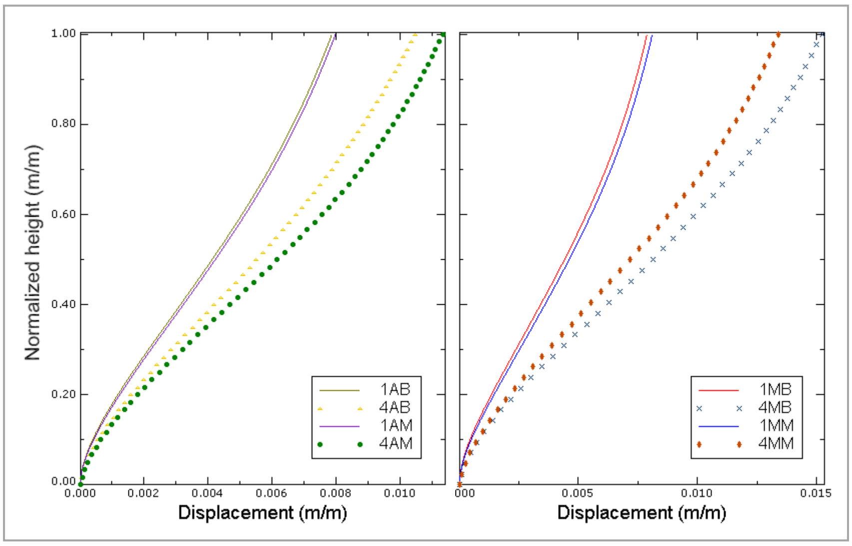

Figure 9. Maximum drift envelope along the columns.

Regarding shear forces, comparative results, presented in Fig. 10, indicate that, although base shear is practically the same for both models, it is their distribution along the height of the building which exhibit great contrasts. Thus, while case 1st show a slow increase of the shear force envelope from top to bottom until the surface of the backfill and from then on it grows at a higher ratio as derived from the rectangular earth pressure distribution imposed, case 4th do not present such abrupt increase beginning at the top of the backfill but instead changes gradually all along the height. This contrast, which is basically explained because of the variability of soil pressure through time that tends to have an inverse triangular distribution at the time of maximum shear forces (Fig 9 – right), leads to shear magnitudes for the 4th case that considerably exceed those of the spectrum response analysis, except for the lower two-to-four floors.

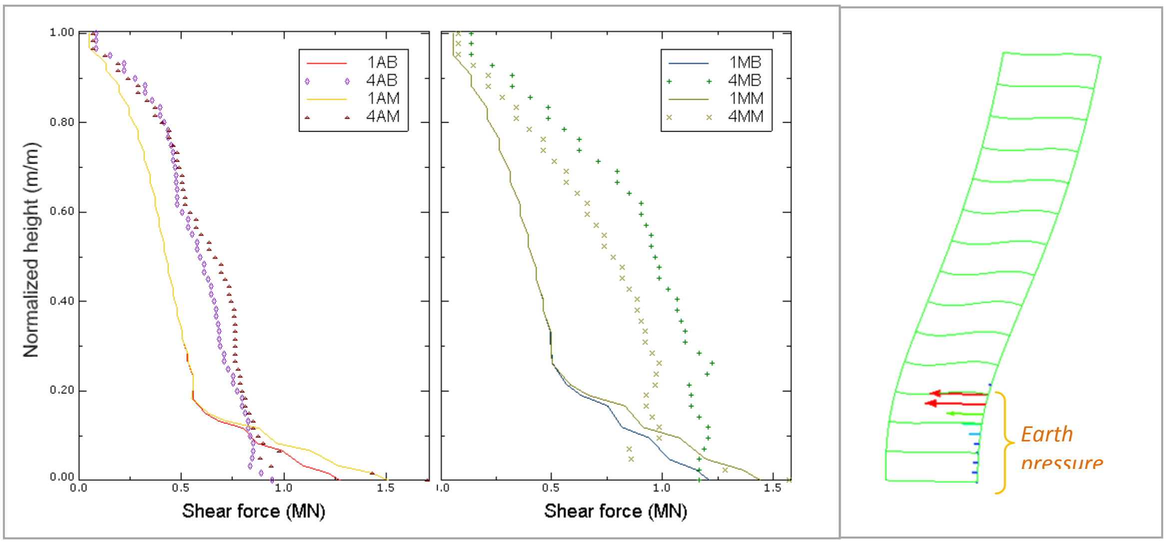

Figure 10. (Left) Maximum shear force envelope along the columns. (Right) Lateral earth pressure distribution at time of maximum shear.

Finally, coinciding with what was discussed about shear forces, when compared with those of the response spectrum analyses, overturning moment envelopes for the refined model, as displayed in Fig. 11, show a very similar base moment, present a localized increase near the backfill surface and consist of clearly higher values from the top of the backfill and up to the roof.

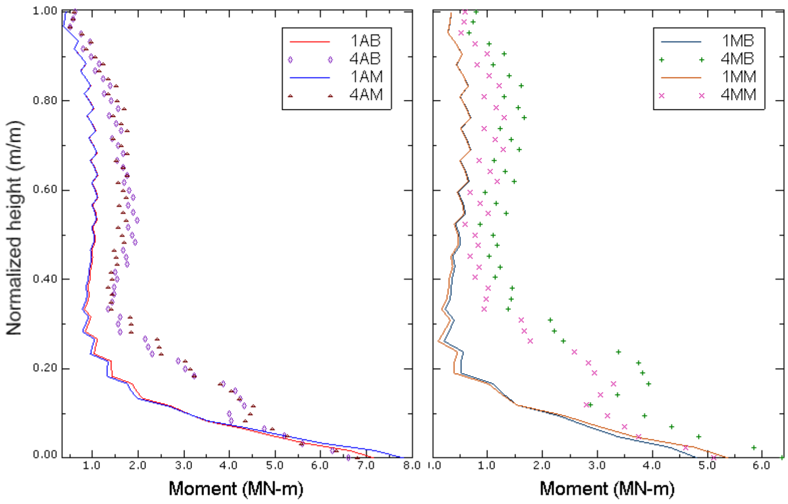

Figure 11. Maximum overturning moment envelope along the columns.

5. CONCLUSIONS

Numerical analyses were developed in order to determine the effects of SSI between a structure built on sloping ground and a backfill supported against one of its sides. The most important findings can be synthesized in two ideas. First, compared with its movement away from the slope, when the structure drifts towards the backfill the deformation of the building in the lower levels is reduced due to the restriction imposed by the slope, which in turn decreases shear and overturning moment at these levels. But given that roof displacements remain almost unaltered, deformations that do not occur in the first levels are translated towards the upper floor, which ends up increasing the demands of curvature, shear forces and moments for elements located above the backfill surface. Second, when contrasting estimates obtained as is currently done for routine design projects of this type (response spectrum analysis with fixed base and imposing a lateral uniform line load to account for earth pressure) with results from a more complex, detailed and realistic model (time-history nonlinear analysis, including flexible foundation and dynamic soil-structure interaction at the structure-backfill interface), it was noted that, even for a similar base shear, drift, shear and moment envelopes for the refined model were higher from the top of the backfill to the roof, which opposes to the traditional idea that SSI is beneficial and that ignoring its effects should lead to improved safety margins while simplifying the analysis. As a consequence, although this is still an exploratory study and results are only examined from a qualitative point of view, evidence consistently points out that, maybe except for the lower floors, the method currently used for analysis of structures with slopes resting on one of its sides is not the most appropriate so as to provide for sufficiently resistant elements able to withstand the demands to which they can be exposed.

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