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SEISMIC EFFECTIVE STRESS ANALYSIS OF GRAVITY BLOCK TYPE QUAY WALLS: APPLICATION TO PIRAEUS PORT

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ABSTRACT

Experience has shown that port facilities, and especially gravity quay walls, are particularly vulnerable to earthquake related hazards. Motivated by numerous observed cases where gravity quay walls suffered large displacement and rotation during earthquakes, even in soils not prone to liquefaction, an illustrative numerical analysis is presented for the response of a typical block-type quay wall section at Piraeus port in Greece. Utilizing the Byrne's elastoplastic constitutive model, an effective stress dynamic analysis is performed using as seismic excitation two recorded strong motions of the seismic environment of Greece. The results emphasize the role of excess pore-water pressure (negative or positive) build-up during shaking on the evolution of the lateral displacement and tilt of the quay wall, shedding light on the potential pitfalls that could emerge from a total stress analysis in which pore-water pressure generation is not directly considered. It is shown that when extensive soil liquefaction does not take place, the negative pore water pressures develop in the backfill mitigate the large displacement and rotation of the quay wall.

INTRODUCTION

Gravity quay wall structures have repeatedly suffered substantial outward displacement and rotation even when subjected to moderate earthquake shaking. (e.g. Pitilakis and Moutsakis, 1989; Egan et al., 1992; Iai et al., 1994; Suganoand Iai, 1999; Elnashai et al., 2010; Zarzouras et al., 2010). Apparently, due to their nature, these structures are extremely vulnerable to liquefaction and lateral spreading. Such phenomena may lead to dramatic horizontal displacements and rotations, resulting not only to the failure of the structural component itself, but also to damage to a number of inter-connected elements: extreme deformation or failure of piping systems and utilities. The strong rocking of quay walls (due only to its inertial forces), when founded on a compliant and weak foundation soil in combination with the one-sided action of the earth pressures leads to the accumulation of horizontal displacement and rotation towards the seaside. This effect is rather amplified due to the current design practice of quay walls: their seismic design against earth pressures unavoidably leads to large dimensions of the walls (Okabe, 1926). The increase of their dimensions is a self-defeating and expensive proposition, as it augments the mass of the quay wall, ultimately amplifying the inertial forces acting on the foundation soil (Zarzouras et al., 2010): a vicious circle that may not serve either the safety or the economy of the project.

Tilt performance criteria related to container crane operations hamper the use of gravity quay walls for container wharf operations imposed to seismic loading. Conventional practice for evaluating

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their seismic stability is based on (i) pseudo-static (force-based) approaches and (ii) oversimplified sliding block (displacement-based) methods of analysis, similar to those applied for embankments! The deformation modes that synthesize the response of the quay wall at large displacements and near failure conditions: sliding, overturning, and bearing capacity mobilization are evaluated separately, totally neglecting the unavoidable interplay with one another. The state-of-practice assigns higher factors of safety for overturning and bearing capacity than for sliding, as these are more critical modes of failure. A key component in the above analyses is the estimation of the seismic active earth pressure. While the Mononabe Okabe limit equilibrium method is widely used in practice to predict active and passive earthquake pressures (Ebeling and Morrison, 1992; PIANC, 2001), the method have limitations, and a generalized limit equilibrium approach has been recommended by the NCHRP 12-70 Project Report 611 (Anderson et al., 2008). In both methods, however, the influence of soil liquefaction on the failure mechanisms of the quay wall is completely ignored.

The vital role that quay walls play in the operational capacity of ports, shipyards and other waterside facilities, increases the pressure to provide more efficient and seismically resilient new infrastructure or improve the existing facilities through rehabilitation and seismic upgrading. Optimizing the seismic performance of gravity quay walls requires a deep understanding of the mechanics that govern their response, and, effective stress analysis is an essential tool that could provide a valuable insight into this. Evidently, the whole problem is very complex. The dynamic response of gravity quay walls is strongly affected by non-linear soil behaviour. Development of excess pore pressures and accumulation of shear and volumetric strains both at the retained soil and the foundation soil, produces the degradation of the shear strength of the soil which may lead to liquefaction. The above phenomena are further complicated when accounting for soil-structure interaction.

The goal of this paper is to investigate the seismic response of block-type gravity quay walls emphasizing the role of pore-water pressure build-up in the soil behind and in front of the wall. Two sub-cases are examined: In the first one (hypothetic case) only the hydrostatic conditions are considered and the possibility of pore-water pressure build up is completely ignored. This case serves as a reference for evaluating the influence of water flow on the system's response (second--realistic case). The comparison is attempted at two performance levels, representing: (a) the contingency-level earthquake (475 years return period) and (b) the ultimate-level earthquake (975 years return period) with application to a typical gravity quay wall section at Piraeus port in Greece. To this end, the paper utilizes the rigorous plain strain finite difference formulation of FLAC2D (Itasca, 2000), along with Byrne's elasto-plastic constitutive model for cyclic stress-strain soil behaviour.

NUMERICAL MODELING

In the framework of the current research program which aims to the upgrade and retrofit of existing piers in ports within Greece, pier II of Piraeus port, built in 1994-1996 was studied. A typical cross section of pier II comprising the geometry of the block-type gravity quay wall and the idealized soil profile is shown in Figure 1. The examined soil profile does not indicate significant liquefaction potential; perhaps apart from the silty sand layer of medium density situated 3 m below the base of the quay wall.

The current 2D section was simulated and analysed numerically using the finite difference code FLAC 2D (Itasca, 2005). The distances of the boundaries from the quay wall are also shown in Figure 1. Two types of models were dynamically analysed: i) model A where the development of negative or positive excess pore pressures, Δu, was allowed and properly simulated, and ii) model B where hydrostatic pressures were applied initially to establish a realistic geostatic field while further development of Δu during the seismic stage was ignored. Both models incorporated Mohr Coulomb plasticity model along with appropriate hysteretic damping. Especially, simulation of model A involves the constitutive law of Byrne (1991) for pore pressure generation which is incorporated in the standard Mohr-Coulomb plasticity model. The waterfront was simulated through constant hydrostatic pressure on the quay wall; thus hydrodynamic effects due to sea-water waves were neglected. “Free-field” conditions were used for the outer boundaries in order to absord wave reflections.
quay wall.

The contact conditions between the blocks of the quay wall as well as between the quay wall and the adjacent soil were modelled with interfaces allowing for slippage and detachement via a Coulomb frictional law. Friction coefficients were assumed equal to 0.5 and 0.7 respectively.

The seismic input motions were chosen among the available records from earthquakes in Greece. The goal was to examine two levels of intensity: a medium and a strong one according to standards of Greece. To this purpose, the chosen records include Kalamata (1986) with PGA 0.26g and Lefkada (2003) with PGA 0.42g, depicted in Figure 2, along with their acceleration spectra. The seismic input motions were applied at the base of the models.

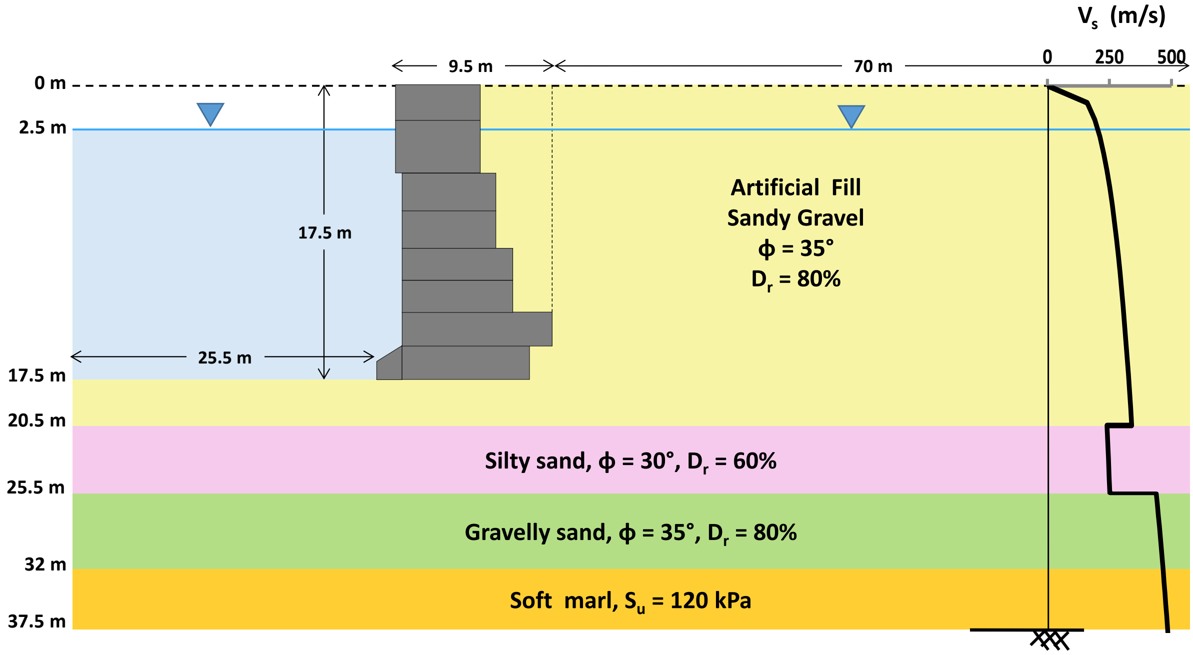


Figure 1. Idealized soil profile and geometry of pier 2 of II of Piraeus port.

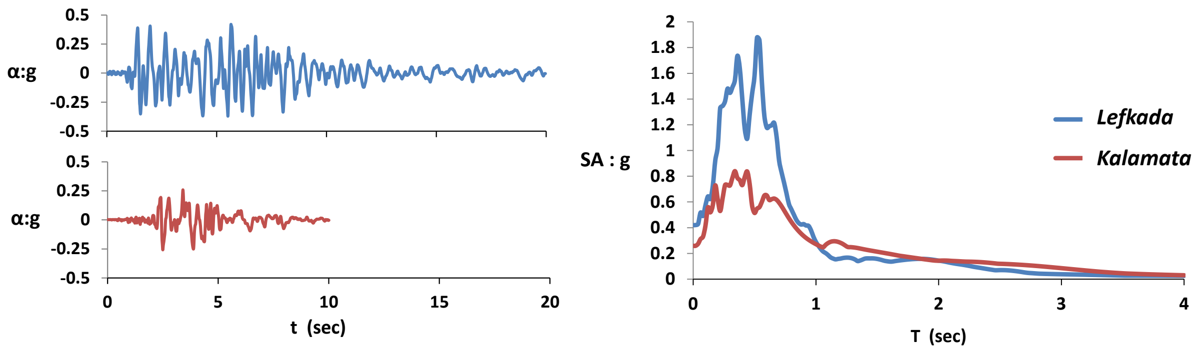


Figure 2. Input motions and acceleration spectra.

RESULTS & CONCLUSIONS

The results are presented in Figures 3-13 through the following graphs: (i) time histories of the quay wall horizontal displacement and rotation, (ii) contours of residual horizontal displacement, shear strain and excess pore water pressure ratio, and (iii) displacement vectors and deformed finite difference mesh.

Numerical results of both models are primarily shown for the higher intensity input seismic motion of Lefkada (2003), in Figures 3 to 7. Examining initially the deformed grid after the end of shaking, illustrated in Figure 3, it is evident that the quay wall sustained greater outward displacement and rotation in case of model (b), where any development of Δu due to seismic loading was ignored. This is due to negative excess water pressure ratio closely behind the quay wall (Dakoulas and Gazetas, 2007), as indicated by the contours of excess pore pressure ratio, r_u in Figure 4. The excess pore pressure ratio, r_u, is defined as the excess pore pressure Δu, over the initial vertical effective stress, σ'_{vo}. Figure 4 also indicates significant generation of positive excess pore pressure within the silty sand layer of Dr =-60%. Nevertheless, it is evident that no liquefaction occurred in the backfill close to the quay wall.

It should be noted that, for all cases considered, no slippage or detachment occurred between the blocks of the quay wall, leading to translation of the quay wall as a rigid body. Interestingly, model B, without Δu, sustained greater outward displacement and rotation compared to model A, with Δu, for all earthquake motions considered. This is attributed to the extensional seaward deformation of the backfill soil adjacent to the quay wall resulting in a geometrical imposed dilation (negative excess pore water pressure), which overshadows the tendency of soil for volumetric contraction (positive excess pore pressure) due to cyclic loading.

Another remarkable observation, for the strong motion record (Lefkada), is that the permanent seaward displacement of the backfill extends all the way to the right boundary of model A, with Δu, approximately 70 m from the quay wall. However, in case of model B, without Δu, the residual displacements vanish rapidly after the midwidth of model, at a distance approximately 40 m from the quay wall. On the other hand, for less strong motions (Kalamata), the distribution of backfill displacements seems to extend to the same distance from the quay wall (Figure 9) despite the larger outward quay wall displacement of model B, without Δu. These discrepancies in the displacement pattern render the problem case specific and they could lead to erroneous design assumptions and displacement-based performance requirements for deformation-sensitive inter-connected elements, such as piping systems and container cranes, when the effect of excess pore water pressure is not explicitly taken into account in the analysis.

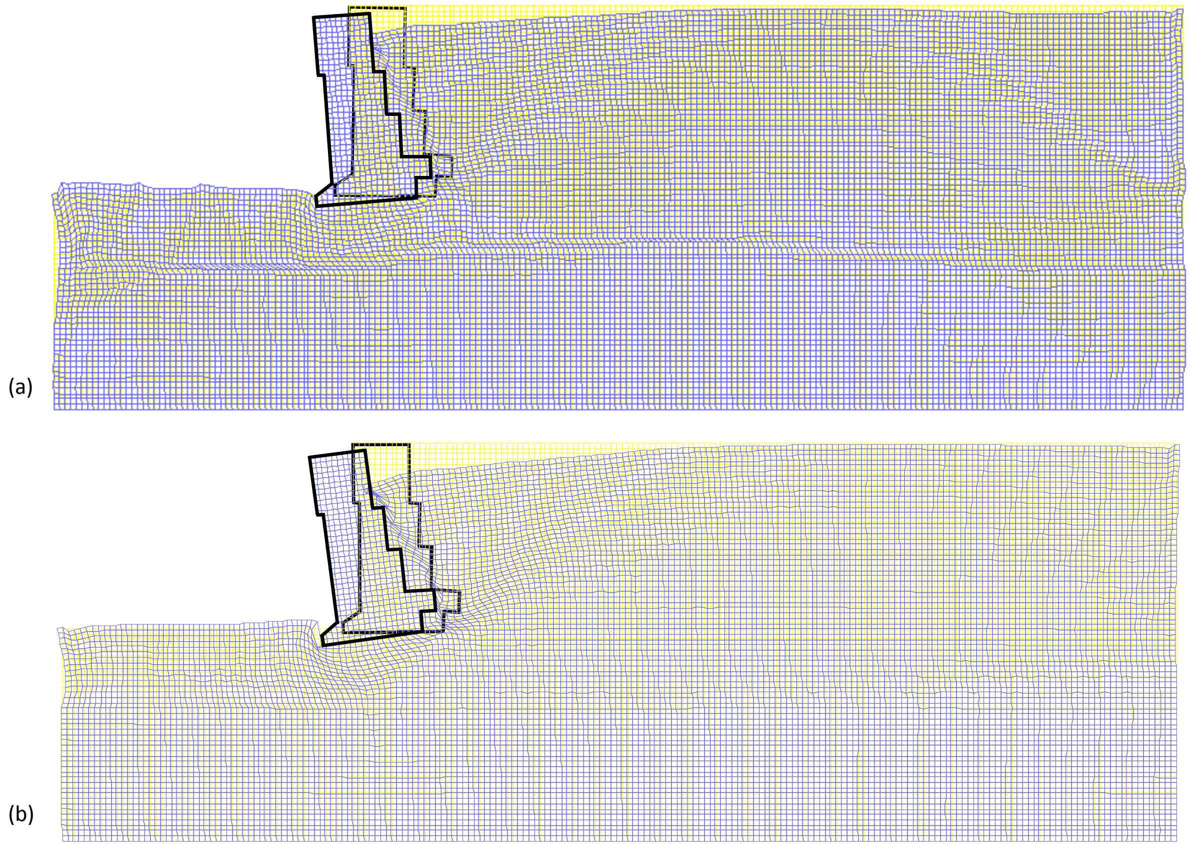


Figure 3. Deformed grid (blue) on top of undeformed grid (yellow) after the end of Lefkada 2003 seismic motion: (a) model A with Δu and (b) model B with no Δu.

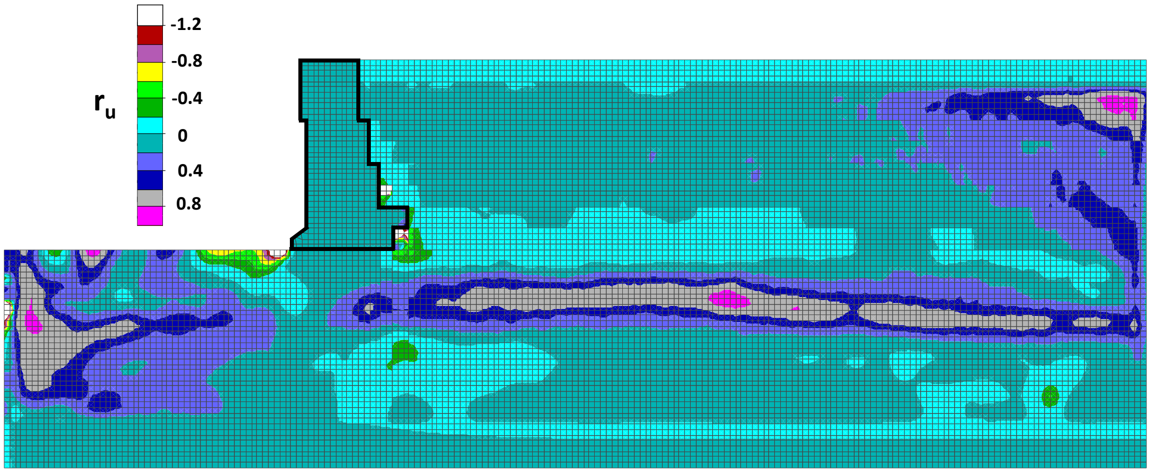


Figure 4. Contours of excess pore pressure ratio after the end of Lefkada 2003 seismic motion in case of the model A where Δu is allowed to develop.

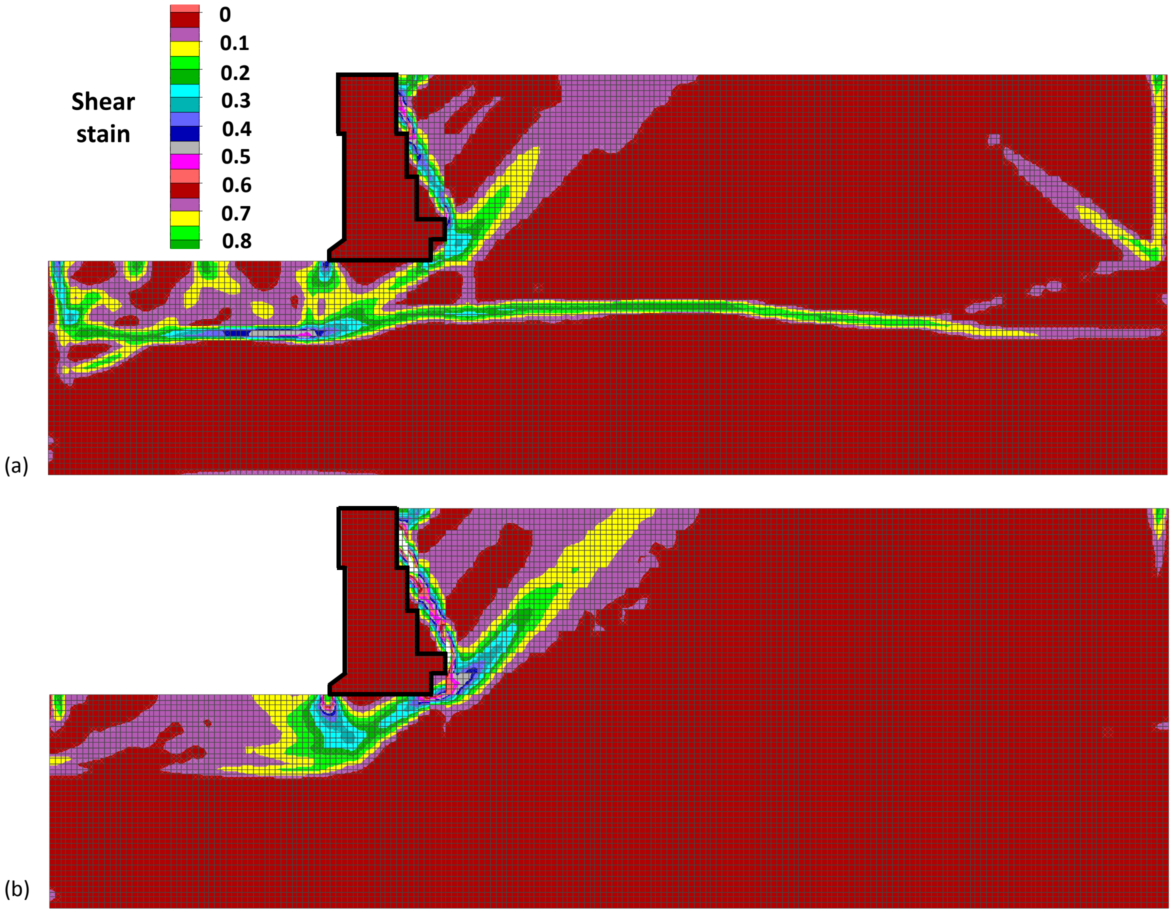


Figure 5. Shear strain contours after the end of Lefkada 2003 seismic motion: (a) model A with Δu and (b) model B with no Δu.

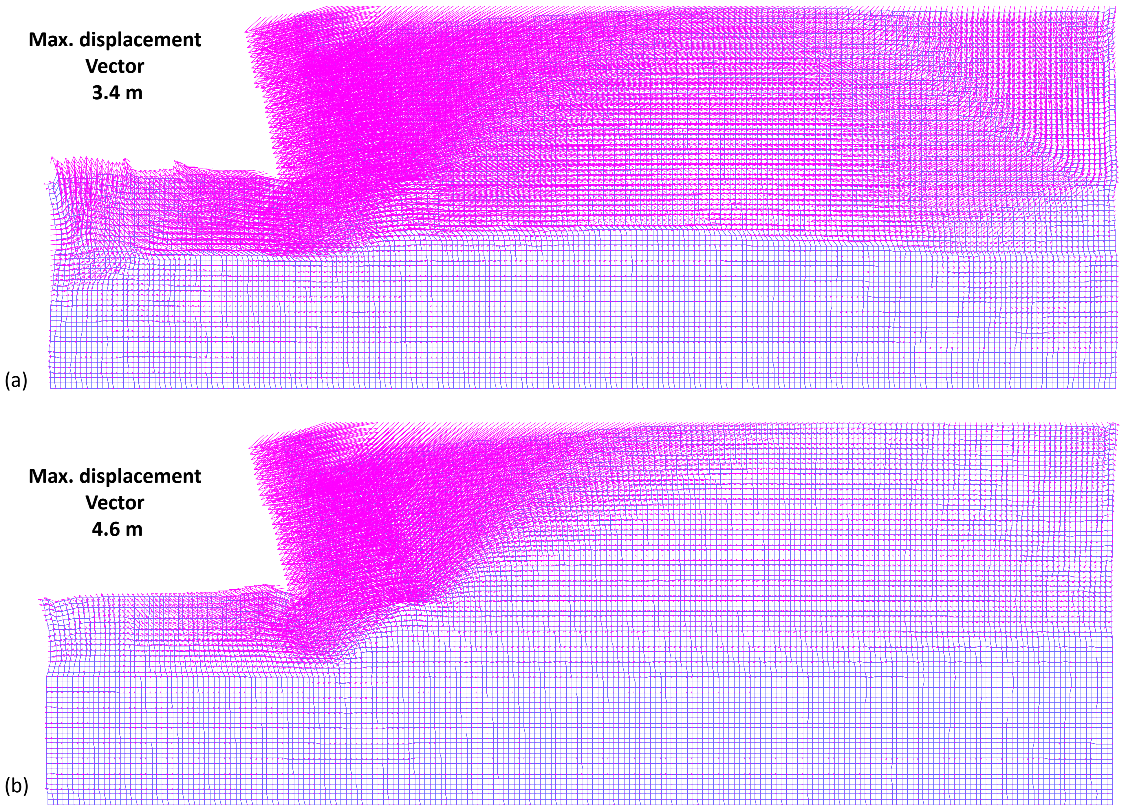


Figure 6. Deformed grid (blue) and displacement vectors after the end of Lefkada 2003 seismic motion: (a) model A with Δu and (b) model B with no Δu.

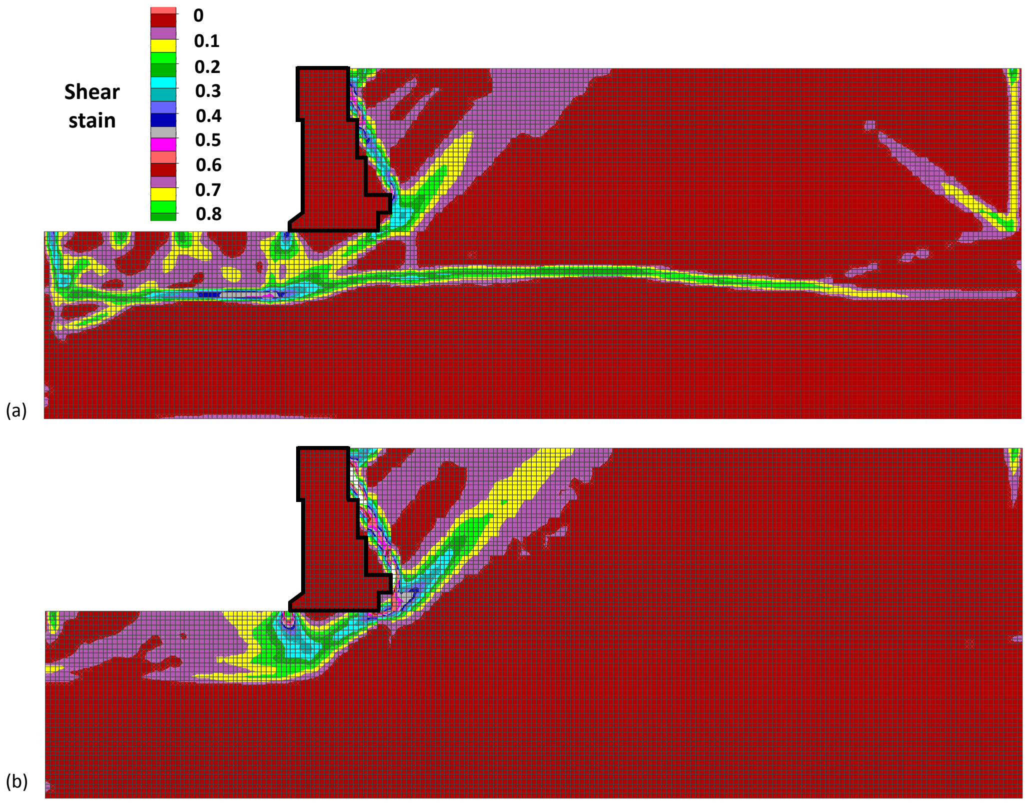


Figure 6. Shear strain contours after the end of Lefkada 2003 seismic motion: (a) model A with Δu and (b) model B with no Δu.

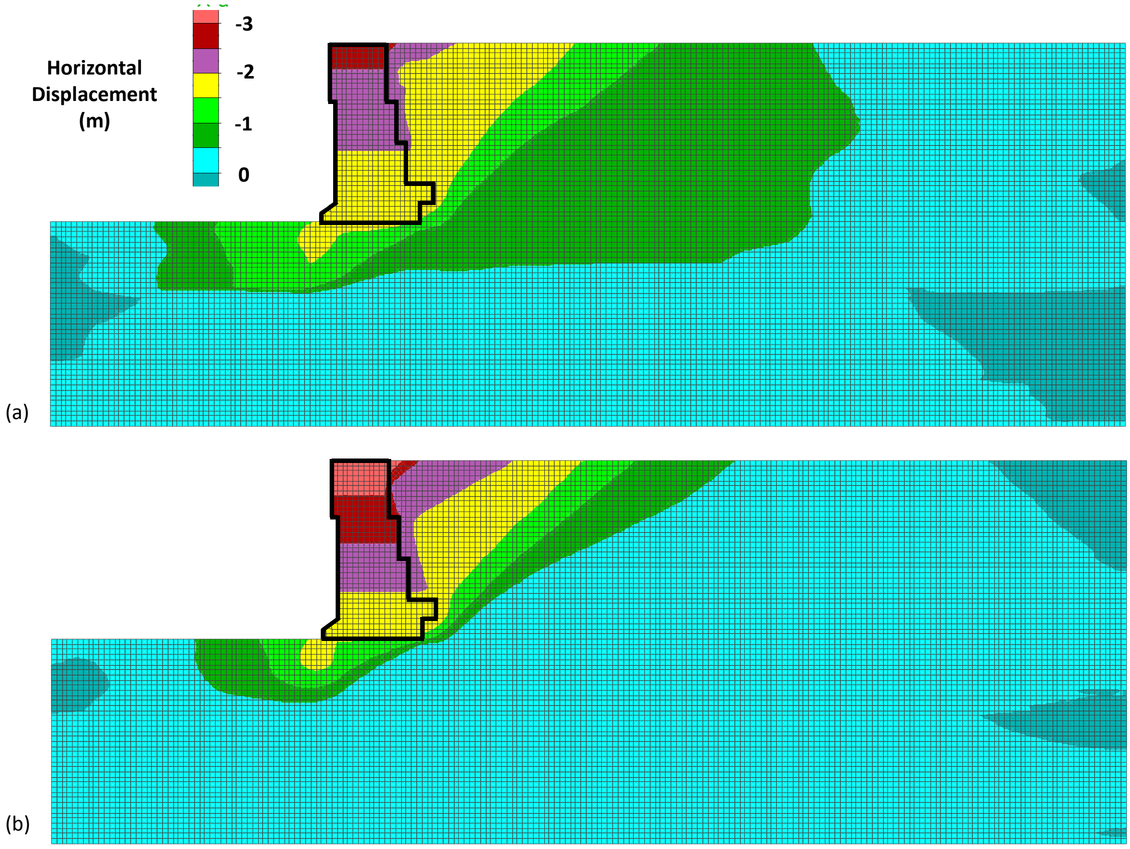


Figure 7. Horizontal displacement contours after the end of Lefkada 2003 seismic motion: (a) model A with Δu and (b) model B with no Δu.

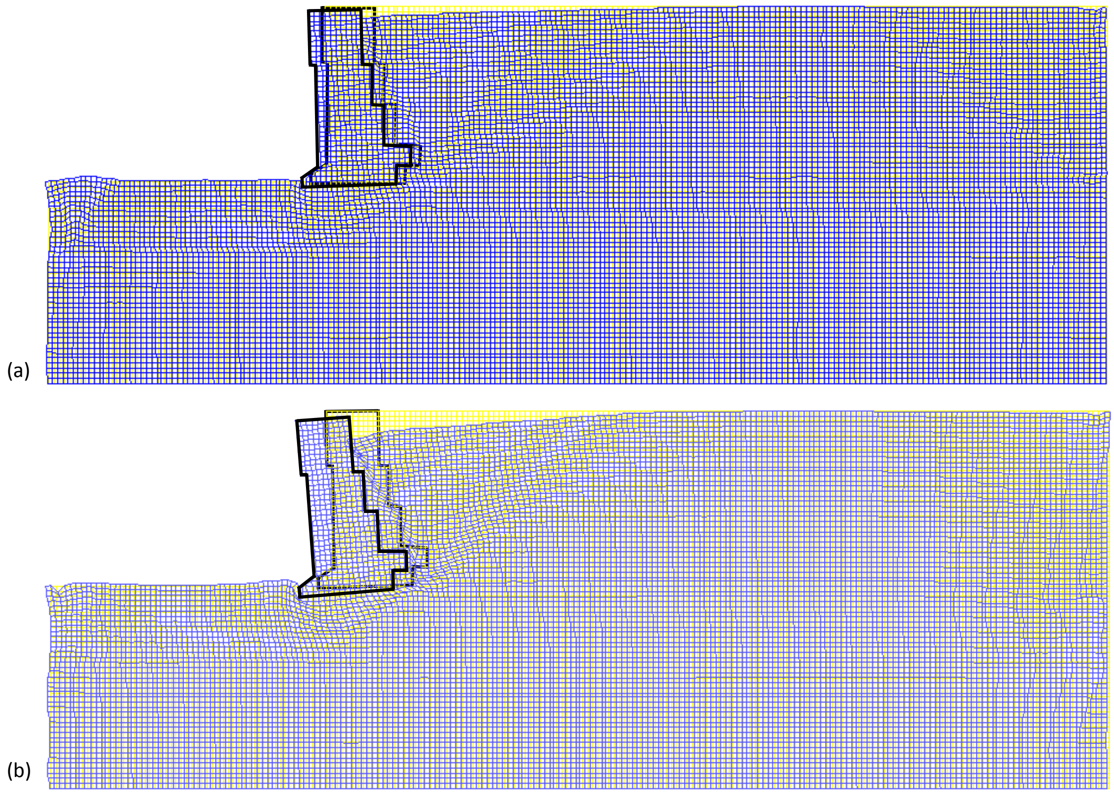
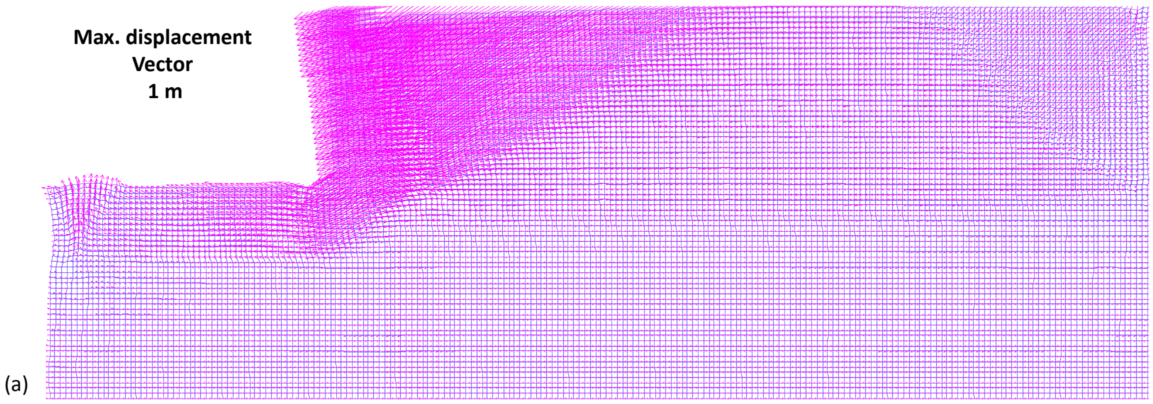


Figure 8. Deformed grid (blue) on top of undeformed grid (yellow) after the end of Kalamata seismic motion: (a) model with Δu and (b) model with no Δu.



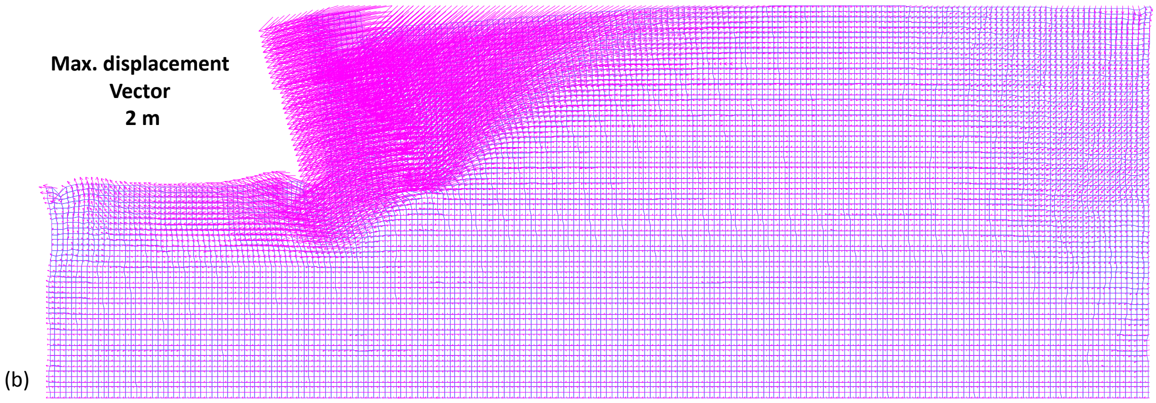


Figure 9. Deformed grid (blue) and displacement vectors after the end of Kalamata seismic motion: (a) model A with Δu and (b) model B with no Δu.

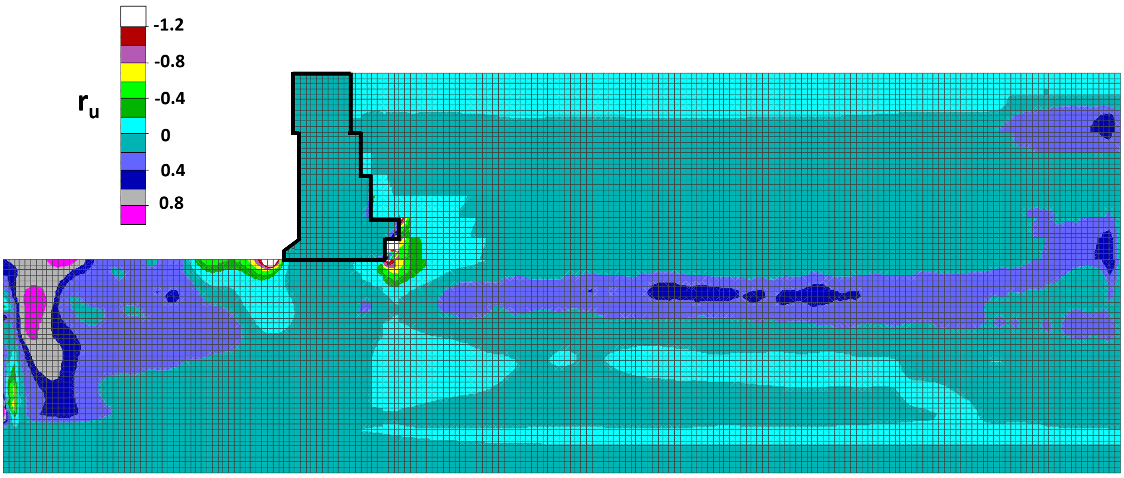


Figure 10. Contours of excess pore pressure ratio after the end of Kalamata seismic motion in case of the model A where Δu is allowed to develop.

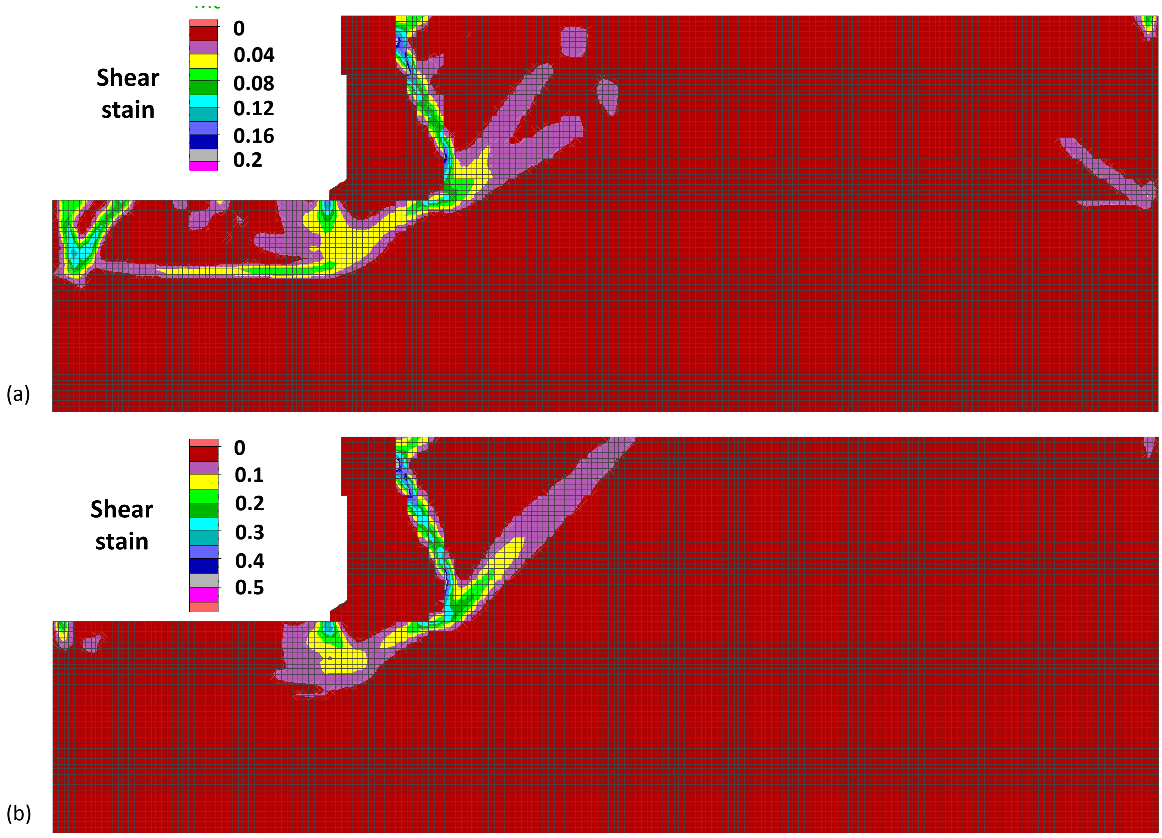


Figure 11. Shear strain contours after the end of Kalamata seismic motion: (a) model A with Δu and (b) model B with no Δu.

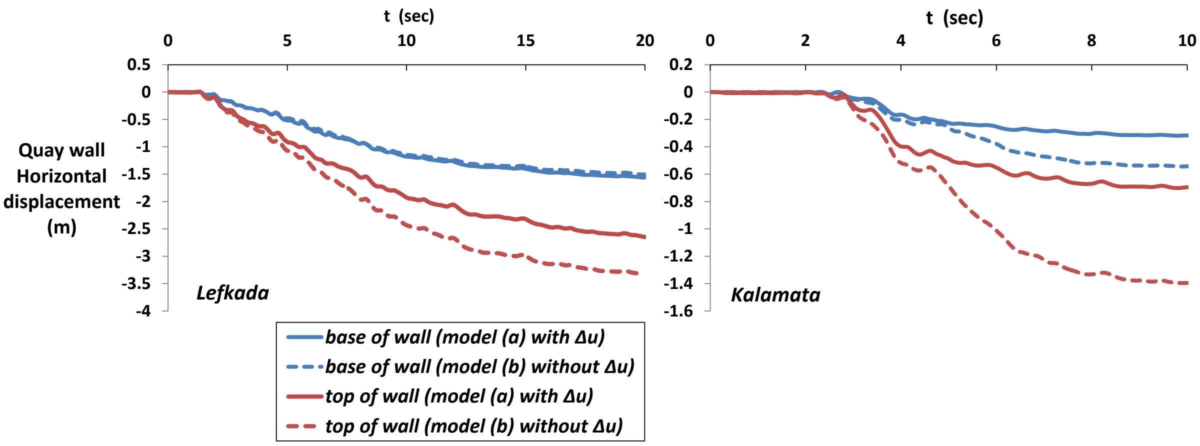


Figure 12. Time histories of quay wall horizontal displacements.

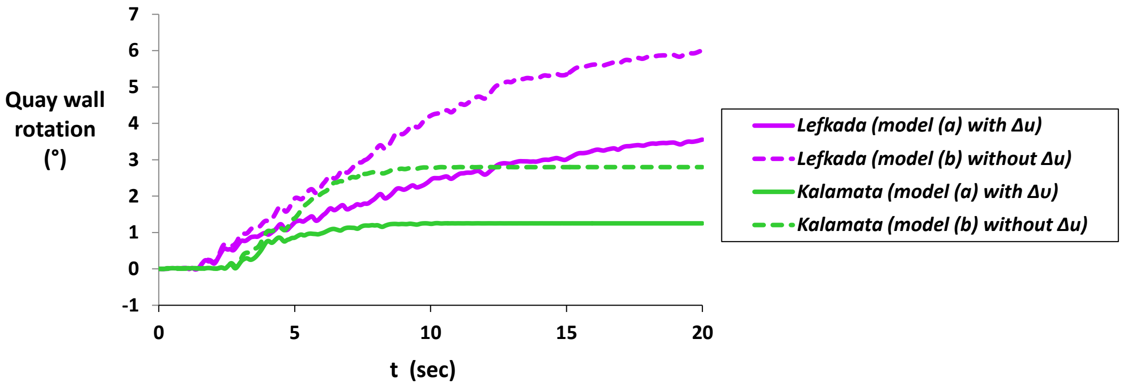


Figure 13. Time histories of quay wall rotation.

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