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IMPROVEMENT SCHEMES FOR ANCHORED SHEET-PILE BULKHEADS UNDER STRONG SHAKING

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ABSTRACT

The seismic response of anchored steel sheet-pile walls subjected to strong earthquake shaking is investigated through a typical tall quay-wall embedded into a relatively dense sandy "foundation" soil. The system is triggered by seismic motions of various intensities at rock-outcrop level (PGA = 0.15 g, 0.30 g, and 0.50 g). The long-established simplified design methods of pseudo-static limit equilibrium in conjunction with the Mononobe-Okabe method, are shown to lead to results for bending moments that in some cases are larger than, but in other cases quite similar to, those computed with the commercially available finite element codes of ABAQUS and FLAC. For the case of strong excitation with PGA = 0.5 g the horizontal deflection of the wall suffers excessive values. To limit these displacements, several improvement schemes are tested. Response in case of soil improvement in three "crucial" locations: around the anchor-wall, in front of the main wall, and in both regions, is presented. Soil improvement leads to significant reduction of the anchor wall deformation, but the induced bending moments of the sheet pile wall are slightly larger than those of the non-improved cases.

Keywords: anchored wall; sheet-pile; improvement schemes; strong excitation

1. INTRODUCTION

Anchored Steel Sheet-Pile (SSP) walls are sometimes prefered as retaining structures in wharves and quays mainly due to their easy installation, while the soft or loose soils that usually underline such waterfront structures could hardly support the additional weight of gravity concrete walls. Nevertheless, anchored sheetpile quay-walls have experienced seismic failures [e.g. Kitajima & Uwabe (1978), Agbabian Associates (1980), Dennehy (1985)].

The following conclusions emerge from a study of the performance of anchored bulkheads in very strong earthquake shaking: (i) most of the observed earthquake failures have resulted from large-scale liquefaction of loose cohesionless soils mainly in the backfill, but sometimes also in the supporting foundation soil; (ii) anchored bulkhead damage by excessive seaward "bulging" and tilting of the sheet-pile wall, accompanied by excessive movement of the anchor block; and (iii) development of detrimental excess pore-water pressures in the backfill next to the wall, once thought to be a contributor to large deformations and failure, is now recognized as unlikely to occur when seaward bulging takes place [Al Atik and Sitar (2010), Gazetas et al. (1990), Dakoulas and Gazetas (2008), Towhata et al. (1996), Ebeling and Morrison (1992)].

Table 1. Fundamental properties of retaining and foundation soil.

	c (kPa)	φ (deg)	E (MPa)	γ (kN/m^3)
Backfill	1	32.5	100	18
Soil 1	1	35	200	19
Soil 2	1	37.5	300	19

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2. SEISMIC BEHAVIOR OF SSP WITHOUT ANY IMPROVEMENT

2.1 Geometry and F.E. discretisation

An 32 m long SSP wall (18 m free and 14 m embedded), shown in Fig. 1, is analyzed dynamically with the finite element codes ABAQUS and FLAC. The wall is embedded into a dense sandy layer (Soil 2), while the retained backfill soil (Soil 1) comprises also of medium density–but not liquefiable–sand. In addition, the backfill is overlain by 4 m of cohesionless fill. The strength and stiffness parameters of the three layers are given in Table 1. The main SSP wall has a rigidity that ranges (depending on the excitation intensity level) from EI ≈ 1 x 10^6 to 2.3 x 10^6 kNm^2/m, ultimate moment capacity from M_{ult} ≈ 4000 kNm/m to 9000 kNm/m, and distance to the anchor wall, L = 45 m to 55 m.

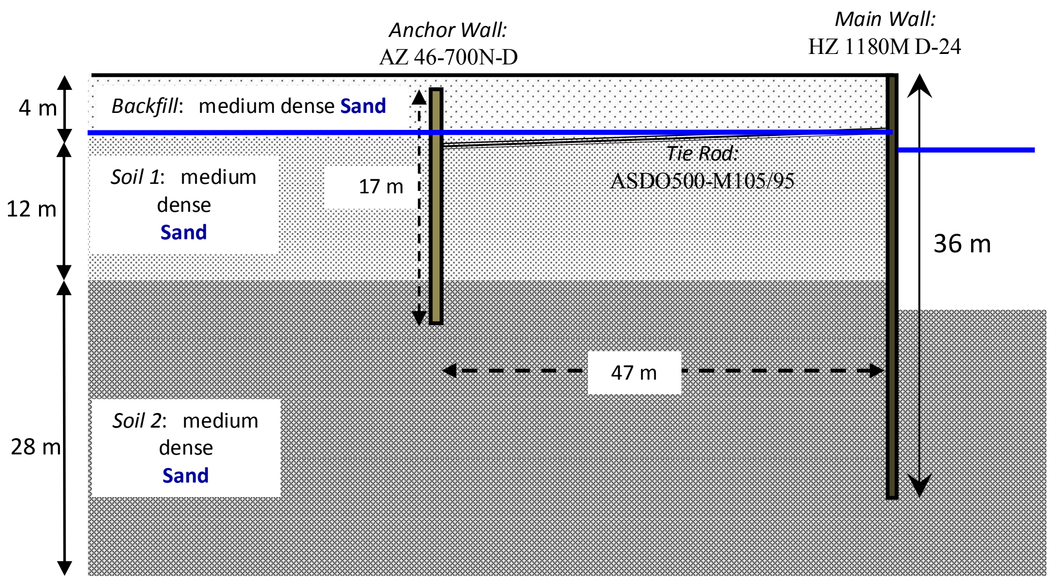

Figure 1. Sketch of the original wall configuration (with no improvement).

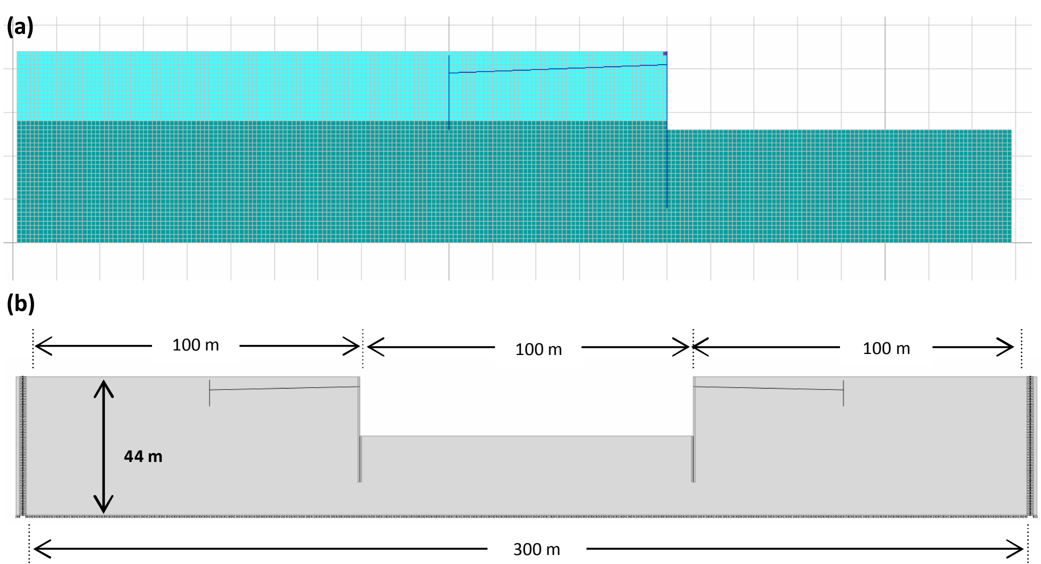

Figure 2. Finite element mesh in: (a) FLAC and (b) ABAQUS.

A description of the two FE models is as follows:

- FLAC: Coupled dynamic-groundwater flow calculations are performed by a two-dimensional explicit finite difference model, illustrated in Fig. 2(a). The built-in constitutive model (named the "Finn model") is employed as it can capture the basic mechanisms that can lead to liquefaction in sand. The lateral boundaries of the main grid of the model are coupled to the free-field grid by viscous dashpots to simulate quiet boundaries.
- ABAQUS: The FE domain shown in Fig. 2(b) is discretized using quadrilateral solid plane-strain fine elements 0.5 x 0.5 m^2. Interface between wall and soil is tension-less and frictional; it is modeled with special elements that allow both separation and sliding, the latter controlled by coefficients of friction μ. To capture radiation damping normal and shear viscous elements ρV_S and ρV_P (per unit area) are placed at the vertical boundaries between the soil domain and the vertical free-field column which is introduced in order to avoid the box effect. A refined constitutive soil model is utilized here through a subroutine attached to ABAQUS, which was developed by Gerolymos and Gazetas (2006) and Anastasopoulos et al (2011).

2.2 Excitation

Determining the seismic ground motions (excitation) which an anchored SSP wall must resist safely is an important first step in the analysis/design. In our study, three levels of seismic intensity are defined regarding their PGA: 0.15 g, 0.30 g, and 0.50 g. For each category, several characteristic accelerograms are selected from earthquakes around the world recorded in sites with $V_{S,30} = 500 \div 800$ m/s, corresponding to EC8 type B Soil Category. These ground motions are suitably modified in the frequency domain, so that their response spectra fit the appropriate seismic design spectra of EC8 (see Fig.3). The fitted to code spectrum records of Category III: Chavriata, Los Gatos, and Jensen Filtration Plant, are applied at the base of the model.

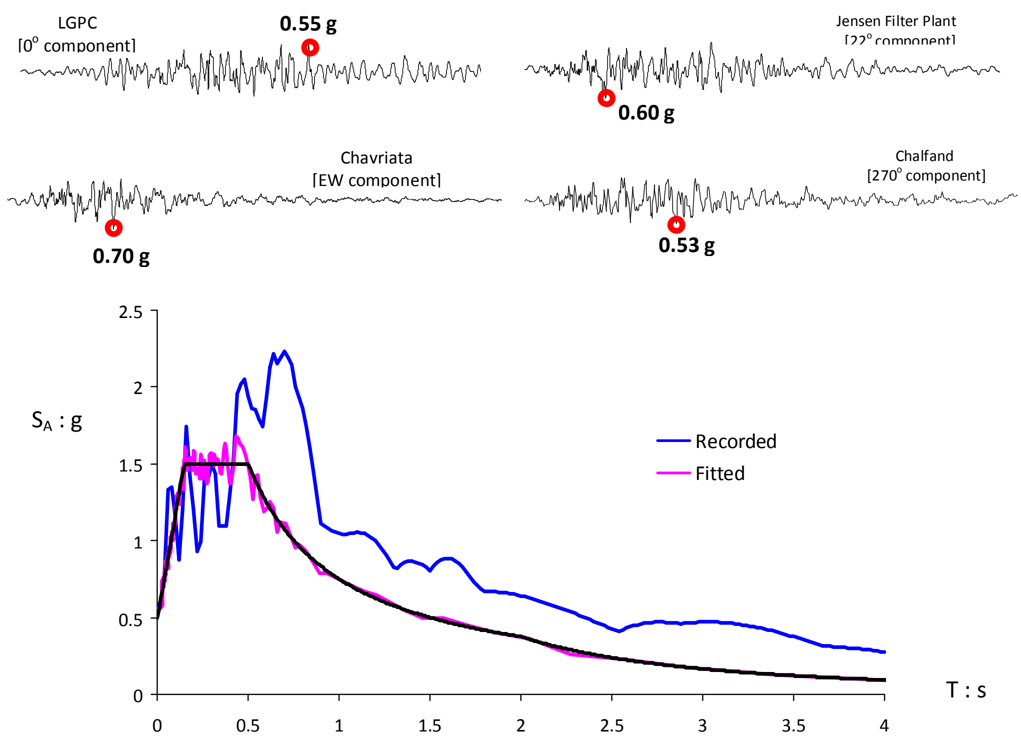

Figure 3. Recorded strong ground motions selected as intense seismic excitations (top). Fitting of the Los Gatos record to EC8 design spectrum for soil category B (bottom).

2.3 Seismic response of SSP wall

Dynamic effective-stress analyses in terms of maximum bending moments on the SSP wall and on anchor wall along with SSP wall's horizontal deflections are summarized in Table 2. Soils are sensitive to pore-water pressure buildup and are subjected to an extreme earthquake motion with a PGA equal to 0.5g.

Table 2. Results of ABAQUS and FLAC codes for strong seismic response of an 36 m height SSP wall.

Excitation	**ABAQUS**		**FLAC**	
	M_{SSP} : kNm/m	U_{SSP} : m	M_{SSP} : kNm/m	U_{SSP} : m
Jensen	8650	0.790	9005	1.260
Los Gatos	8940	0.930	9145	1.080
Chavriata	8805	0.926	9080	1.410

In accordance with Table 2 the following conclusion can be drawn: SSP wall develops large horizontal displacements as large as 1.5 m, almost uniformly distributed along the waterfront. In line with the technical standards in Japan for port and harbor facilities OCDI 2002 (see Table 3) as well as PIANC 2001, such large displacements on the main wall are unacceptable, and they basically imply restricted use of the wall. Hence, to reduce the main wall's deformation, soil improvement schemes are tested and are discussed in the next section. A detailed version on the seismic response of anchored SSP walls in several soil deposits can be found in Gazetas et al. (2016).

Table 3. Quaywall deformation standards from the viewpoint of Temporally Service [from the Technical Standards and Commentaries for Port and Harbour Facilities in Japan, OCDI 2002].

Suffered deformation: maximum swelling or settlement		
Quawall Depth	Less than 7.5 m	Greater than 7.5 m
Normal Use	0–20 cm	0–30 cm
Restricted Use	20–30 cm	30–50 cm

3. SEISMIC BEHAVIOR OF SSP WITH IMPROVEMENT

In the light of the above, soil improvement was chosen at two "crucial" locations: (i) the ancor-wall and (ii) the front of the main wall, leading to three different scenarios:

- Scenario 1: soil improvement around the anchor wall
- Scenario 2: soil improvement in front of the SSP wall
- Scenario 3: soil improvement both around the anchor wall and in front of the SSP wall.

Figure 4 portrays the dimensions and location of the aforementioned remedy measures. The properties of the improved soil are: E = 500 MPa, v = 0.2, and γ = 23 kN/m^3. These are typical values of improved soil which could be achieved with state-of-practice techniques such as soil compaction, soil mixing: lime soil admixtures, cement admixtures, fly ash admixtures, etc.

3.1 Soil Improvement on horizontal deflections and bending moments

The results from soil improvement Scenario 1, shown in Figures 5 and 6, exhibit a significant reduction of the anchor wall deformation leading to negligible values of bending moments compared to the no-improvement case. The almost non-deformed anchor-wall constrained the main wall displacement at the height of the cable connection, reducing its overall horizontal movement. However, the shape of deformation, led to slightly higher bending moments compared to the no-improvement case. The effectiveness of this scenario of soil improvement is attributed to the diversion of the shear zone of the active failure wedge to greater depths below the anchor wall/improved soil system.

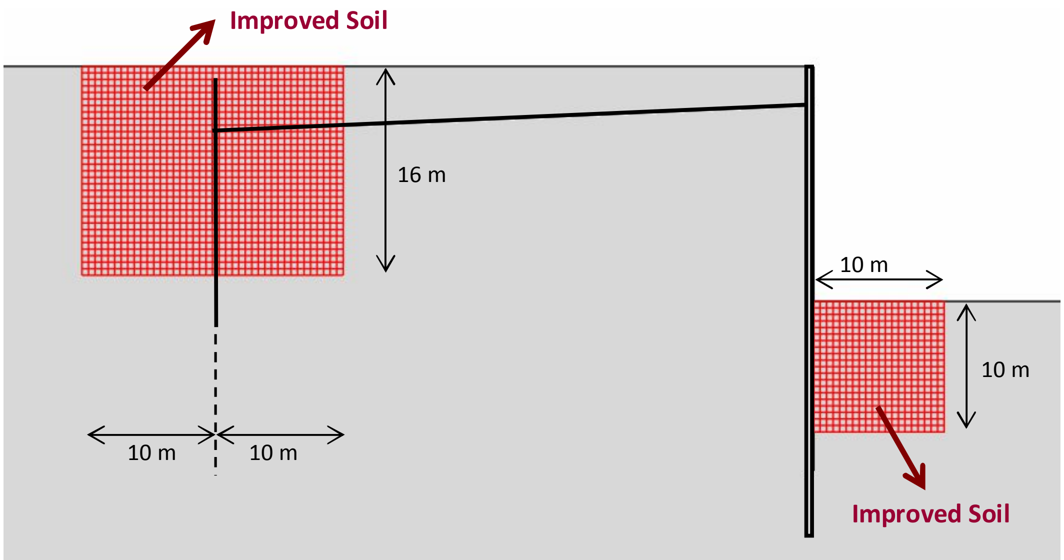

Figure 4. Location and geometry of the soil improvement regions in case of a 36 m main wall and an 18 m anchor wall.

The results for the case of scenario 2, show a decrease of the main wall displacements compared to the no-improvement case, but the effectiveness of this soil improvement scenario is not as high as in case of scenario 1, due to no diversion of the shear zone of the active failure wedge. The comparison between scenarios 1 and 2 proves the critical importance of the anchor-wall region. Even the horizontal displacements of the main wall below -14 m are almost the same between scenario 1 and 2, the top deflection in case of scenario 1 is considerably smaller: 0.847 m for scenario 1 compared to 1.104 m for scenario 2. The explanation for this is straight-forward: the improvement of soil around the anchor makes it stiffer, thus the main wall is retained stronger at the top.

In case of scenario 3 significant decrease of the horizontal movement of the anchor-wall as well as the main wall occurs. The combination of the scenario 1 and 2 leads to the smallest horizontal displacements of the main wall in the order of 0.6 m at the top and 0.9 m at mid-height, and to the smallest bending moments in the wall. As a consequence of the substantial bending moments reduce on the main-wall, the required stiffness of the main wall is smaller therefore its section can be reduced as well.

3.2 Effect on pore water pressures

As portrayed in Figure 7, the water pore pressure ratio, r_u , behind the main-wall at a depth of 8 m is building up to a value of $0.9 < 1$ [where $r_u = 1$ signifies the initiation of soil liquefaction]. However, no liquefaction occurs. Also, at the depths of 21 m and 36 m the pore pressure ratio is low and the liquefaction is not possible. In case of scenario 1 (soil improvement around the anchor wall), water pore pressures are smaller especially for the deeper soil layers. The soil close to the surface (depth 8 m) does not indicate any substantial difference than before. This response is as expected because the improvement around the anchor results in reduce of main wall and soil displacements therefore the limitation of system's deformation does not allow the water to dissipate, therefore the pore water pressures are almost the same than before. At deeper levels, the main wall can be deformed and water dissipates more easily. In case of scenario 3 (soil improvement around the anchor wall and in front of the main wall), for all the depths behind the main wall the pore water pressure ratio is nearly the same with the case of no improvement at all. The explanation is straight-forward: soil improvement on both regions lead to restriction of horizontal displacements thus in limiting dissipation of water pressures.

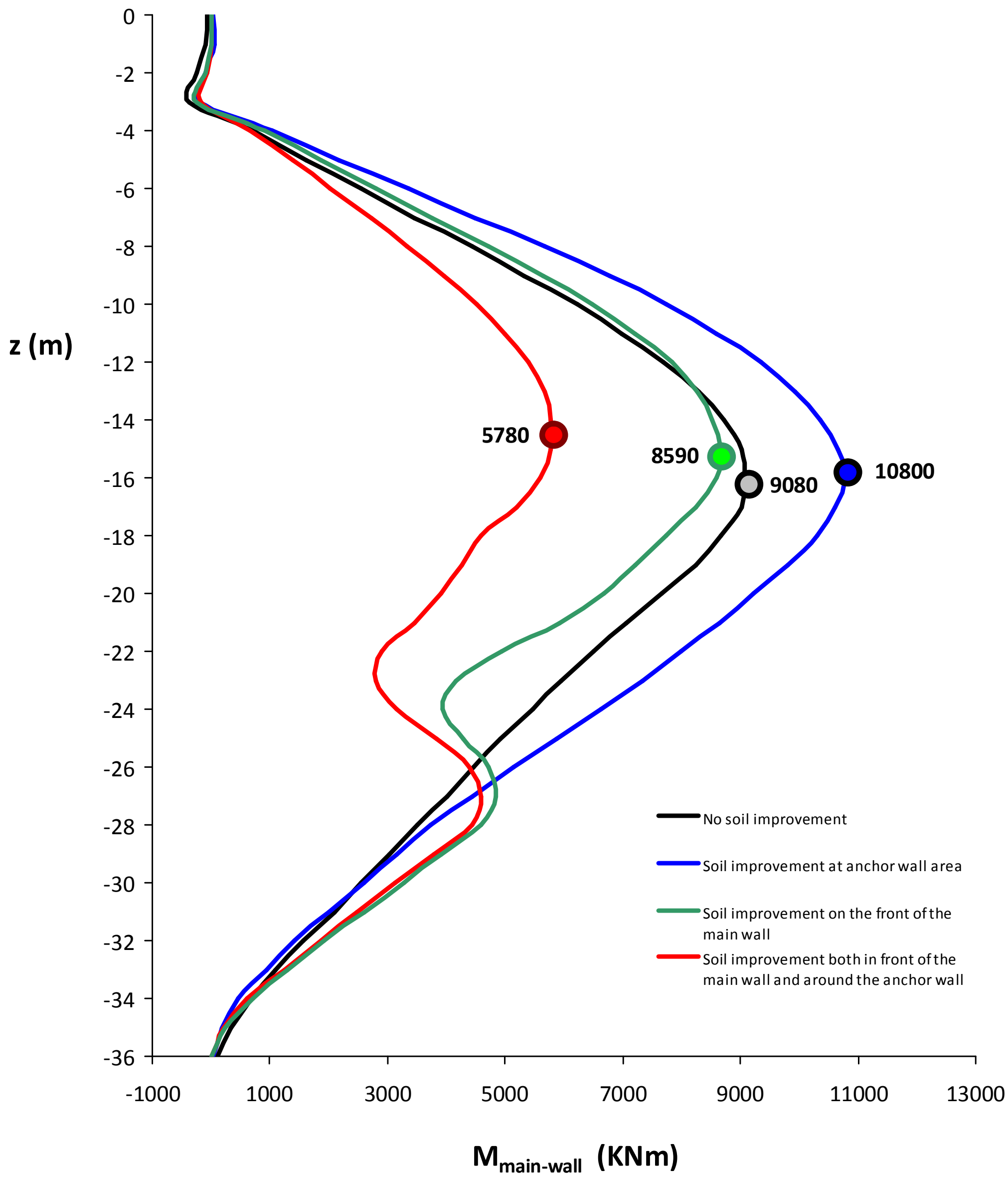

Figure 5. Bending moment distribution of the main wall at the instant of maximum moment for the different cases of soil improvement and comparison for the unimproved case. Excitation: Jensen Filtration Plant.

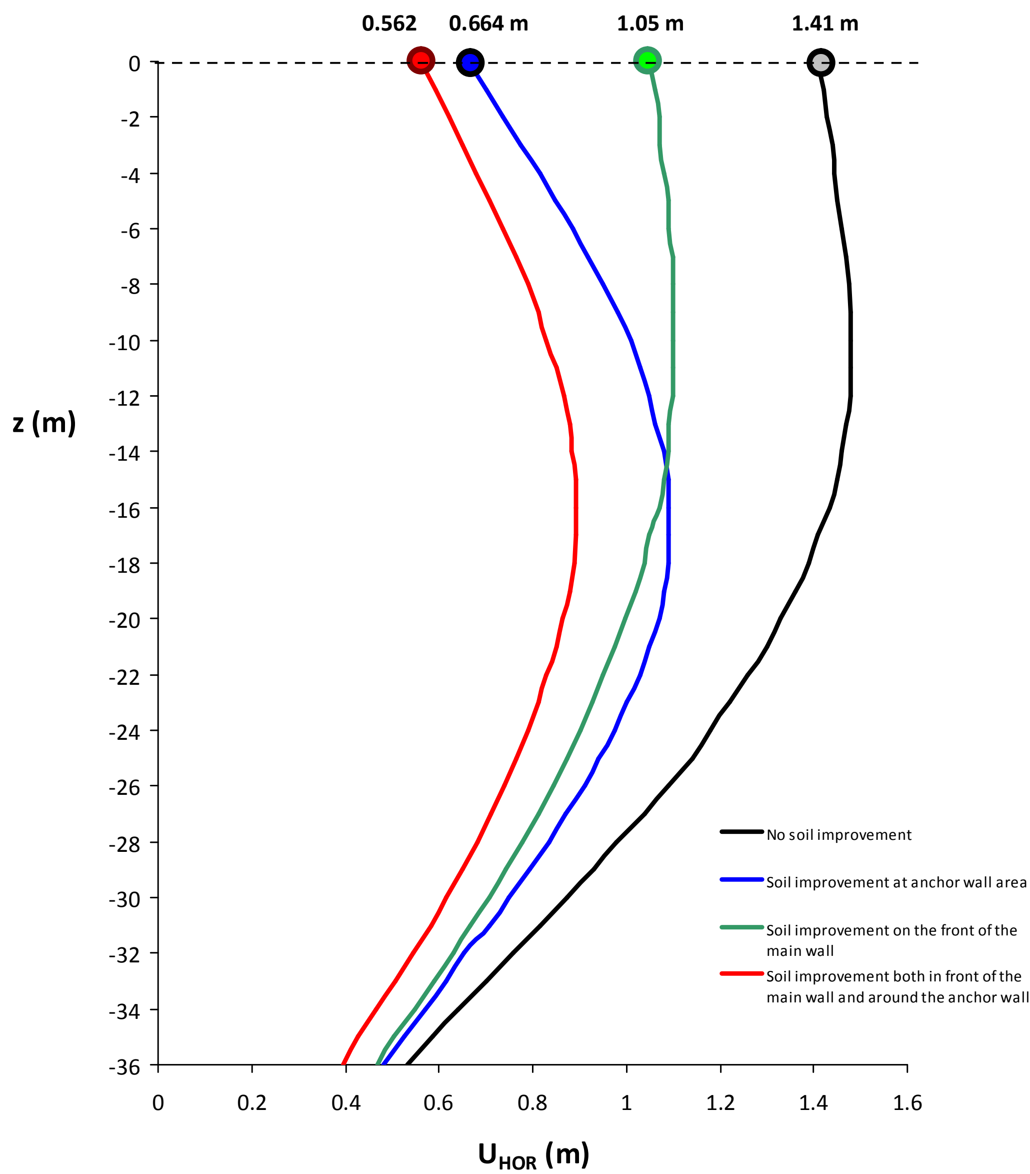

Figure 6. Distribution of horizontal displacements of the main wall at the instant of maximum moment for the different cases of soil improvement and comparison for the unimproved case. Excitation: Jensen Filtration Plant.

4. CONCLUSIONS

Horizontal displacements induced on the main SSP wall when soil improvement includes the anchor zone (scenario 1 and 3) are of the order of 0.5 m. Such a deflection is absolutely normal for a flexible type of wall with 18 m free height which is subjected to an extremely strong excitation of PGA = 0.5 g. However, soil improvement only in front of the SSP wall (scenario 2) seems not to be effective in reducing lateral deflections. To sum up, improvement either around the anchored wall or both in the anchored and main wall regions proved to be adequate measures while improvement only in front of the SSP wall does not suffice. In our analyses only a limited excess pore water pressure was allowed to develop. Thus, liquefaction did not take place behind or at the base of SSP wall.

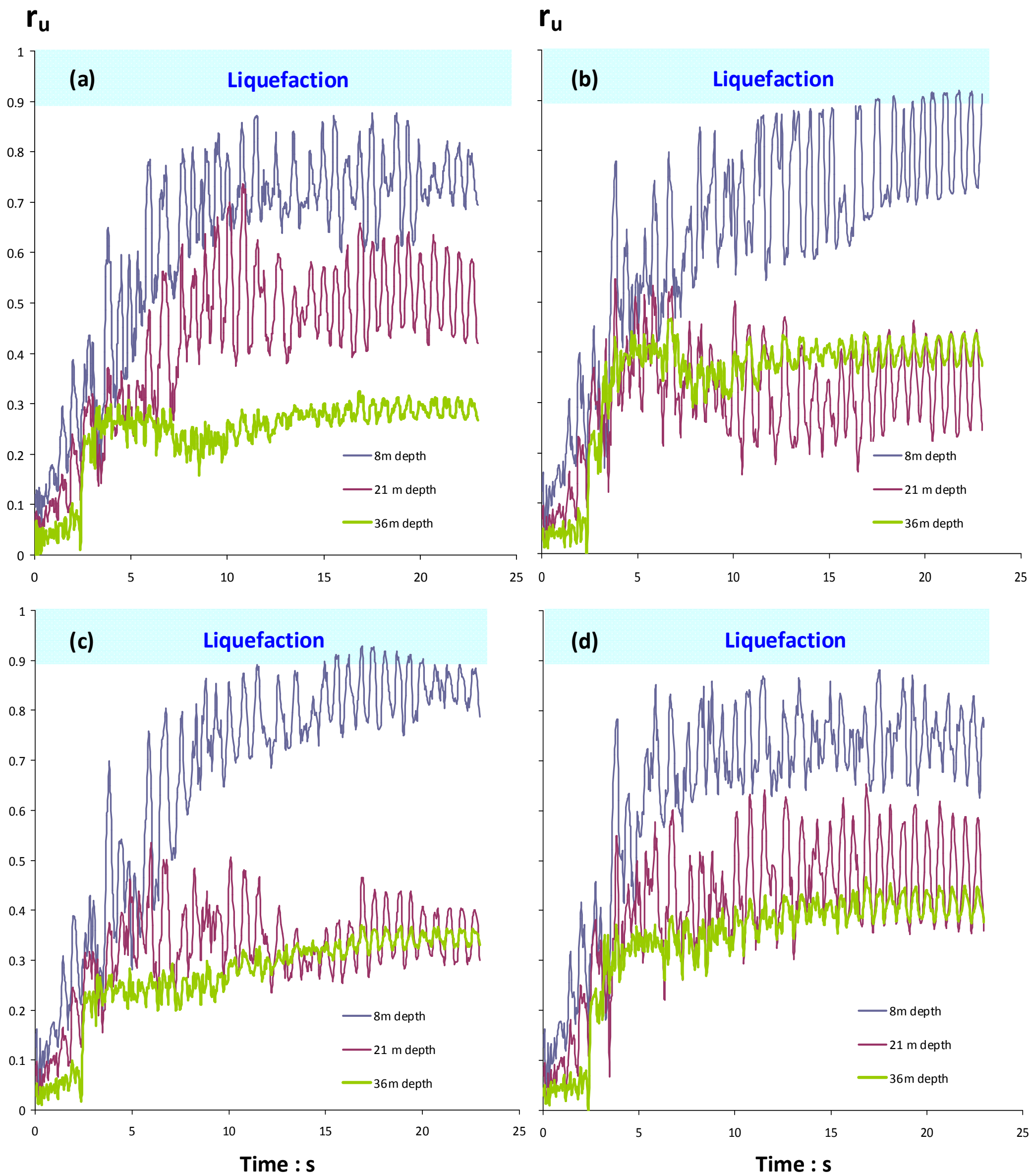

Figure 7. Time histories of pore water pressure ratio, ru , in three different depths at the free field far left from the anchor wall when: (a) soil is not improved, (b) soil around the anchored wall is improved, (c) soil in front of the main wall is improved, and (d) soil improvement occurs in both regions.

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