SEISMIC PERFORMANCE OF BLOCK-TYPE GRAVITY QUAY-WALL: NUMERICAL MODELING VERSUS CENTRIFUGE EXPERIMENT

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Abstract: Numerous earthquakes have demonstrated the high vulnerability of all types of quay walls to strong ground shaking. Motivated by this observation, numerical modeling of a block-type gravity quay wall is performed to simulate centrifuge test data from the University of Dundee. Undrained seismic effective stress analysis is carried out by applying the UBC3D-PLM constitutive model for stress-strain soil behaviour through the use of the finite element code PLAXIS. The soil model is first calibrated against published results from cyclic simple shear tests with the target parameter being the number of cycles required for liquefaction triggering. The computed accelerations, excess pore-water pressures and displacements are shown to be in satisfactory agreement with the measurements, contributing towards an understanding of the mechanisms that govern the seismic response of the studied wall.

Introduction

Sea, lake and river waterfront energy and transportation terminals can be found in most industrialized regions, in Europe and worldwide. A critical component of such facilities is the harbour quay wall, a retaining system separating land from water. 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; Sugano and Iai, 1999; Zarzouras et al., 2010; Tasiopoulou et al., 2013, 2014). A very recent example: the damage of the Lixouri harbour quay wall in the two 2014 Cephalonia (Greece) earthquakes, despite the relatively small magnitude ($M \approx 6$) of the events.

As quay wall are usually founded on soft soil deposits and retain reclaimed land, they are also subject to liquefaction and lateral spreading–induced permanent displacements and large ground settlements. Such phenomena in turn trigger damaging deformations to nearby structures and lifetimes. These effects are thus responsible for a dramatic increase of the overall seismic vulnerability of energy facilities and terminals, compared to the vulnerability of inland industrial facilities. The seismic damage to quay walls may cause a rather pronounced impact on the economy of the affected region in terms of direct and indirect losses.

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 and the foundation soil, produces shear strength degradation which may lead to liquefaction. The above phenomena are further complicated when accounting for soil-structure interaction. Evidently, the deformation modes that synthesize the response of the quay wall at large displacements and near failure conditions can not be realistically assessed by conventional design procedures. The use of suitable constitutive soil models that balance simplicity and effectiveness in conjunction with powerful numerical techniques is a key-step for a successful prediction. However, these more sophisticated procedures need to be verified before used in practice, and centrifuge test databases can play a vital role on this.

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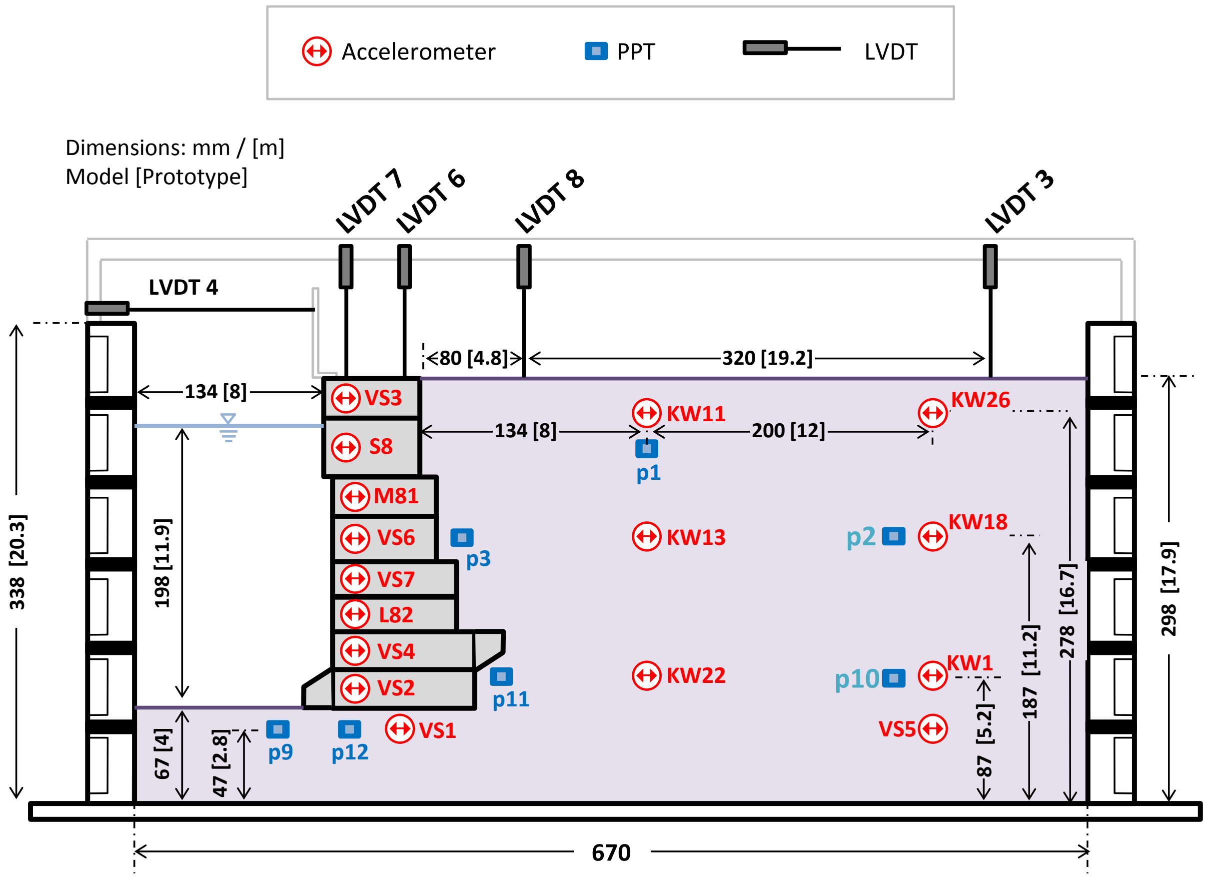

Figure 1. Centrifuge model setup and instrument locations

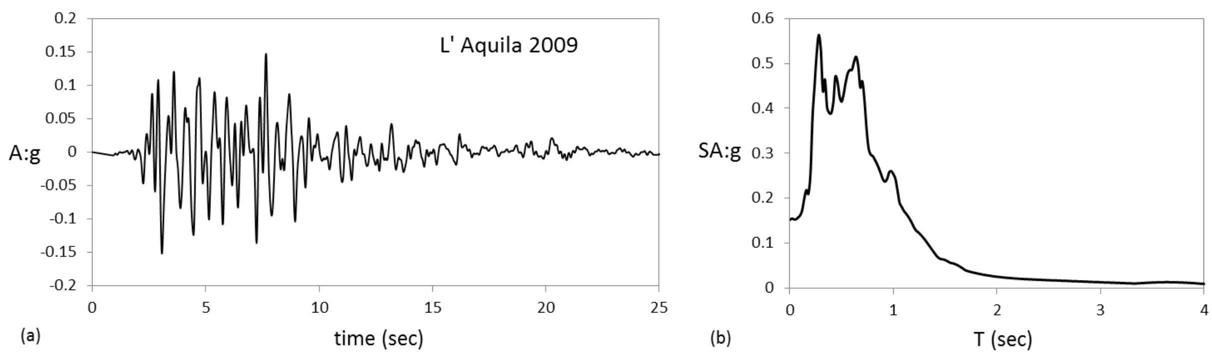

Figure 2. Base input motion: Acceleration time history and corresponding spectrum

In this paper, the seismic response of a block-type gravity quay-wall is investigated by means of an undrained effective stress analysis considering pore water pressure build-up due to cyclic loading. The analysis is performed with the finite element code PLAXIS via the use of the UBC3D-PLM constitutive soil model (Galavi et al, 2012) which is a 3D reformulation of that originally proposed by Puebla et al (1997) and Beaty and Byrne (1998), designated as UBCSand. At first, a calibration procedure for the model parameters is applied involving fitting against published results from undrained cyclic simple shear tests. A comparison of calculated and measured centrifuge model response (Anastasopoulos et al, 2015) is then presented, in terms of acceleration, displacement and pore-water pressure time histories. The analysis reproduces the measured response with satisfactory engineering accuracy.

Centrifuge Testing

A dynamic centrifuge model test conducted at the University of Dundee centrifuge facility is examined. The model used a fine quartz based silica sand (HST 95) and simulated the response of a multi-block gravity quay wall made of aluminum alloy, as a replica of a typical wall at Piraeus port in Greece. The sand bed was formed into an ESB (equivalent shear beam) model container by carrying out air pluviation at a relative density of D_r = 80%. It was then saturated with water and subjected to an acceleration field of 60g. A sketch of the experimental setup (including the instrumentation layout) is illustrated in figure 1. While in flight, a sequence of actual ground-acceleration records were applied at the base of the model as input motion. However, only results for the first acceleration time history (a record from the M_L = 5.9 L' Aquila 2009 earthquake) are presented and compare with numerical predictions (figure 2). For more details on the experimental procedure the reader is referred to Anastasopoulos et al (2015).

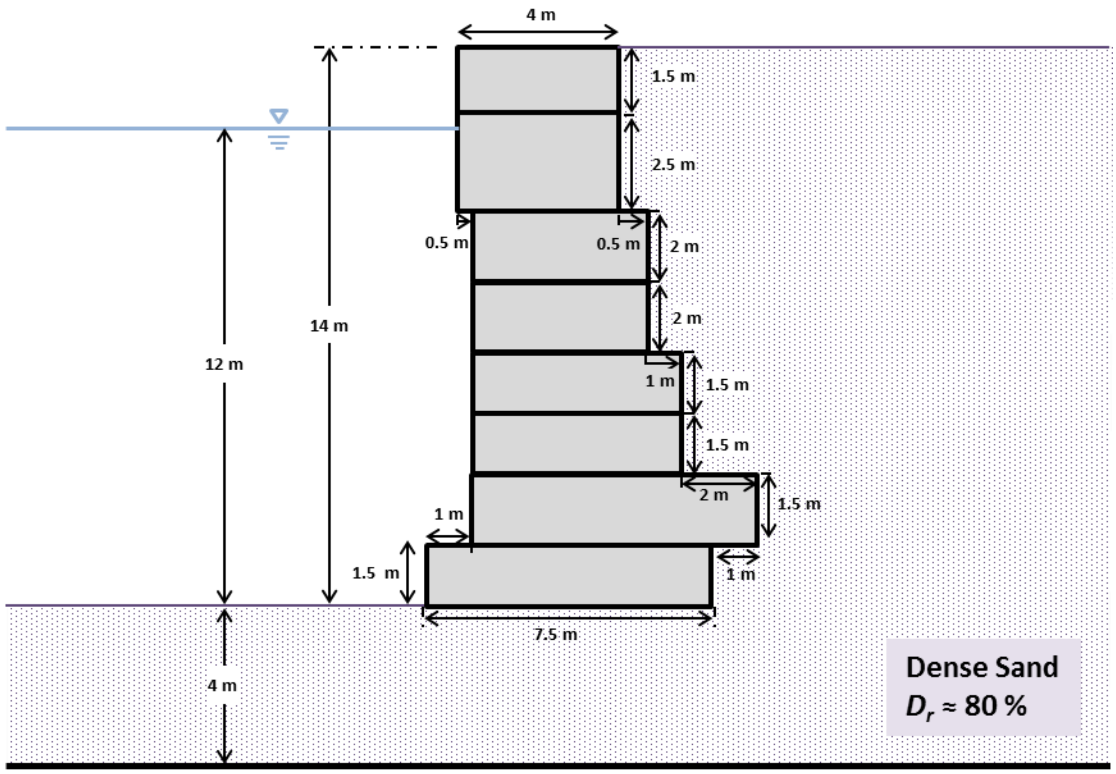

Figure 3. Sketch of the studied multi-block gravity quay wall

Numerical Modelling

A section of the centrifuge model is analysed in prototype scale with the finite element code PLAXIS 2D AE. The analysis is conducted taking into account for material (in the soil) and geometric (interface) nonlinearities. Both the quay wall and the soil are modelled with 15-node triangular plane strain elements, elastic for the former and nonlinear for the latter. The aluminum alloy frames and rubber spacing layers of the ESB model container were also modelled in detail, assuming elastic behaviour. Prescribed displacements were imposed on the horizontal boundaries of each frame prohibiting their movement in the vertical direction, and kinematic constraints were assigned to the external and internal vertical edges of the model allowing it to move as a laminar box (Zienkiewicz et al, 1988, Gerolymos et al, 2008; Zafeirakos and Gerolymos 2013, Galavi et al, 2013).

The contact conditions between the blocks of the quay wall as well as between the quay wall and the adjacent soil were modelled with special interface elements allowing for slippage and gapping via a Coulomb frictional law. Special interface elements were also placed along the inner edges of the ESB model container. The friction interface angles were assumed equal to 18^o between the blocks of the quay wall, 10^o and 14^o at the back and at the base of the wall, respectively, and 10^o for the inner vertical edges of the container. To avoid spurious oscillations at very small deformations and for high frequency components of motion, Rayleigh damping was also introduced into the model, accounting for equivalent hysteretic damping values between 1.5% and 3% in the range of 0.2 Hz and 2 Hz. The initial horizontal effective stresses were set to 0.5 times the vertical effective stresses, while the coefficient of hydraulic permeability was estimated to $k = 3 \times 10^{-4}$ m/s (in prototype scale) and assumed to be constant throughout the analysis. The geometric characteristics of the studied quay wall are detailed in figure 3 whilst the used finite element mesh (3130 elements in total) is portrayed in figure 4.

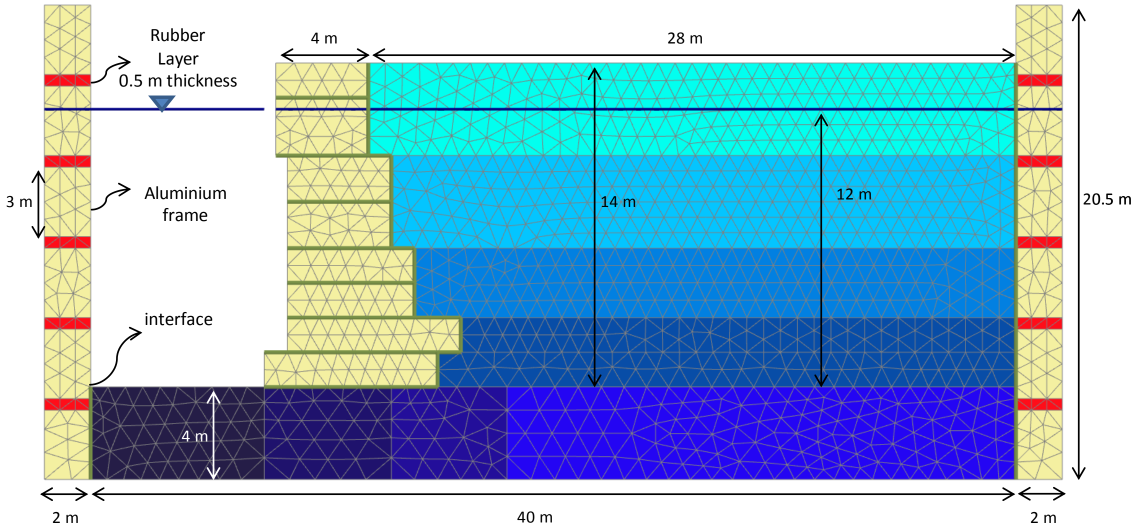

Figure 4. The finite element model

Constitutive modelling

Cyclic soil behaviour is described through a constitutive model (UBC3D-PLM) which is an extension of that originally developed at the university of British Columbia by Puebla et al (1997) and Beaty and Byrne (1998). It involves two yield surfaces (a primary and a secondary one) of the Mohr-Coulomb type. The primary surface evolves according to an isotropic hardening law while a simplified kinematic hardening rule is used for the second yield surface. The plastic flow rule is non-associated and is based on the Drucker-Prager's law and Rowe's stress dilatancy hypothesis. For more details on the formulation and parameters of the UBC3D-PLM model, the reader is referred to Galavi et al (2013). Of special interest for this study are two model parameters: fac_{hard} that controls the evolution (hardening) law of the secondary yield surface to cyclic loading (K_σ effect), and fac_{post} that governs the stiffness of soil after the onset of cyclic mobility when the mobilized friction angle reaches the peak friction angle. The smaller the value of fac_{hard} the greater the excess pore water pressure development and the lesser the liquefaction resistance. On the other hand, the post-failure (at the onset of cyclic mobility) stiffness degradation increases with decreasing fac_{post}. The smaller the fac_{post}, the less stiff the post-failure response.

Calibration methodology

Beaty and Byrne (2011) proposed a set of equations for the calibration of the UBCSand model parameters, with the corrected SPT value $(N_1)_{60}$ being the sole variable. The calibration procedure aimed at matching the cyclic resistance ratio indicated by the NCEER/NSF curve for a given corrected SPT blowcount to induce liquefaction at 15 uniform cycles. At first, by applying this methodology, using the empirical correlation of Idriss and Boulanger (2008) for $(N_1)_{60}$ as a function of relative density D_r:

$$(\mathrm{N}_1)_{60} \approx 46\left(D_r\right)^2 \tag{1}$$

and assuming $\varphi_{cv} = 36^o$ for the phase transformation angle, the values of table 1 were derived for the model parameters.

Then, parameters fac_{hard} and fac_{post} are calibrated against published results (Kammerer et al, 2004; Sriskandakumar, 2004; Tatsuoka et al, 1986) from undrained cyclic simple shear tests in terms of the cyclic stress ratio versus the number of loading cycles required to cause liquefaction (figure 5). The best-fitting values for fac_{hard} are presented in the same figure, while the optimum values of fac_{post} were found to be closed to zero (≈ 0.01). Representative results of computed response are shown in figure 6, for CSR = 0.3, initial effective stress σ'_{v0} = 100 kPa and lateral earth pressure coefficient at rest K_0 = 0.5.

Table 1. UBC3D-PLM model parameters

Parameters	**Unit**	**Description**	**Value**
φ_{cv}	(deg)	Phase transformation angle	36
φ_p	(deg)	Peak friction angle	42
c	kPa	Effective cohesion	0
k^e_B	-	Elastic bulk modulus number	937
K^e_G	-	Elastic shear modulus number	1339
k^p_G	-	Plastic shear modulus number	3581
m_e	-	Power for stress dependency of elastic bulk modulus	0.5
n_e	-	Power for stress dependency of elastic shear modulus	0.5
n_p	-	Power for stress dependency of plastic shear modulus	0.4
R_f	-	Failure ratio	0.662
p_A	(kPa)	Reference stress	100
fac_{hard}	-	Fitting parameter to adjust number of cycles to liquefaction	see Fig 5
fac_{post}	-	Fitting parameter to adjust post liquefaction behaviour	0.01
$(N_1)_{60}$	-	Corrected SPT blow counts	29

Comparison between numerical and experimental results

The predicted and measured performance measures (accelerations, displacements and pore water pressures) are shown in figures 7 to 11. It may be seen that, in general, both the magnitude and the pattern of all time histories are in reasonable agreement. The following observations are worthy of note:

- Negative excess pore pressures are developed near the wall-soil interface decreasing in inverse proportion to the distance from the quay wall. The computed pore pressure time histories are characterized by large spikes indicative of intense dilative soil behaviour, partially attributed to the undrained conditions (dissipation is not allowed) compared to a smoother observed response.

- The measured wall rotation exhibits large-amplitude oscillations contrary to a considerably smoother computed response. The residual rotations, though, practically coincide.
- The predicted and measured maximum wall horizontal displacement and free field settlement (LVDT8) are in satisfactory agreement. The moderate difference between the horizontal displacement atop and at the base of the quay wall implies that sliding prevailed against bearing capacity failure due to the dilative response of sand.
- The inward accelerations are systematically larger than their outward (seaward) counterparts which appear to have been curtailed due to excessive sliding at the base of the wall.
- The absence of long period pulses in both the measured and computed acceleration time histories, reveals that either no or limited soil liquefaction took place.

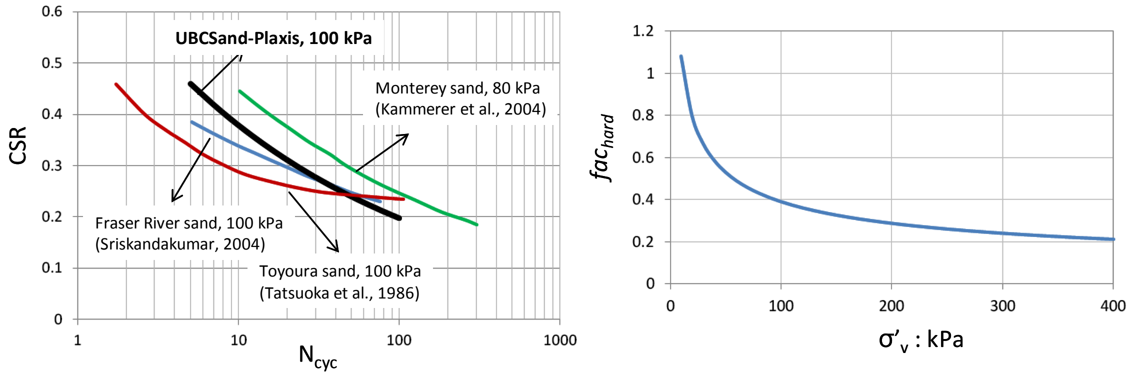

Figure 5. Calculated and measured liquefaction resistance in cyclic simple shear testing (left) and corresponding values of the fac_{hard} parameter (right), for D_r = 80%

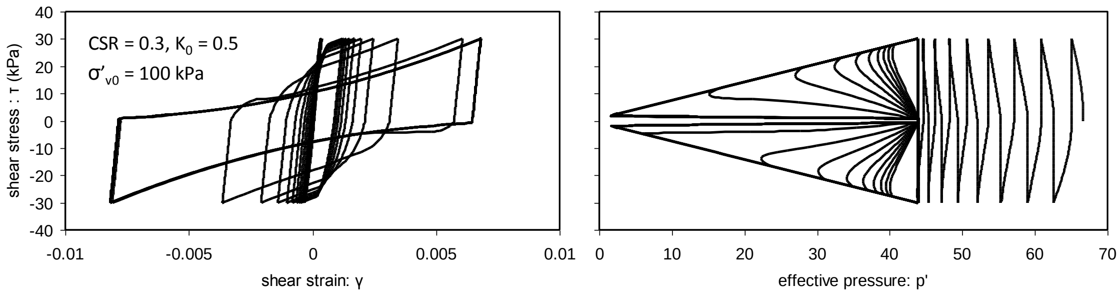

Figure 6. Simulated cyclic simple shear test with the UBC3D-PLM model for D_r = 80%

Summary

An undrained effective stress analysis of a block-type quay wall has been conducted with the use of the UBC3D-PLM constitutive model. A calibration procedure was proposed for the model parameters validated by comparison with centrifuge test results. The predictions were shown to be in satisfactory agreement with the observed response.

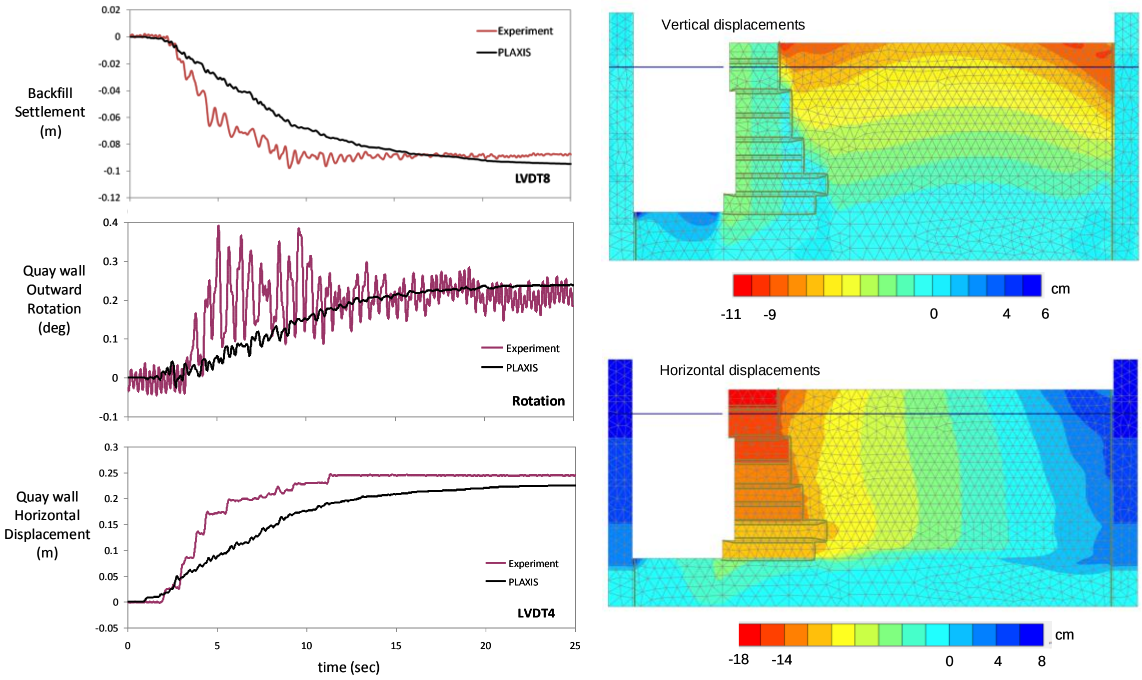

Figure 7. Measured versus predicted response (left), contours of the computed residual horizontal and vertical displacements (right)

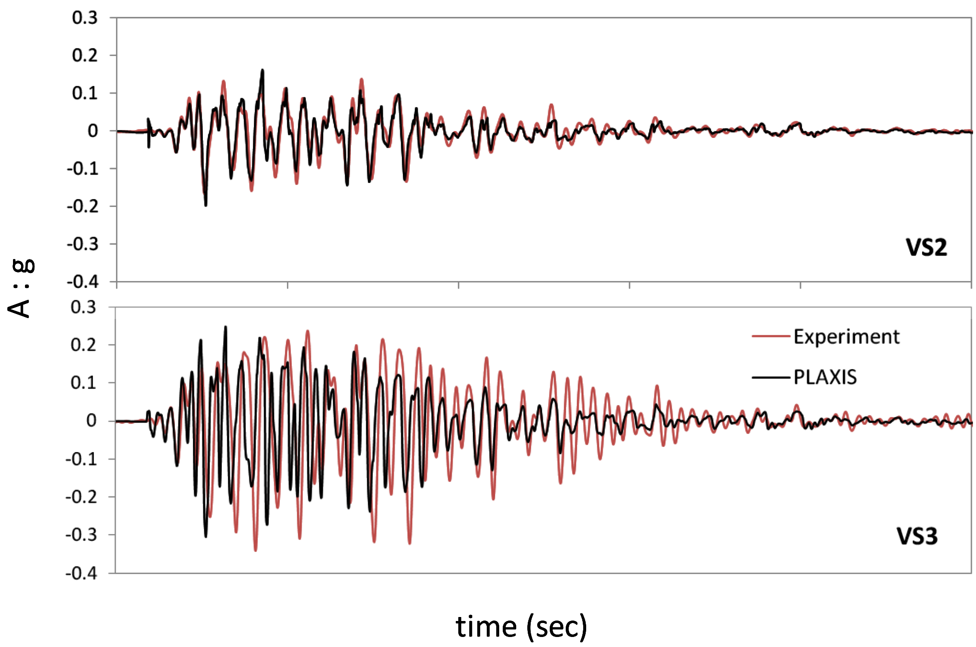

Figure 8. Measured versus predicted acceleration time histories at quay wall

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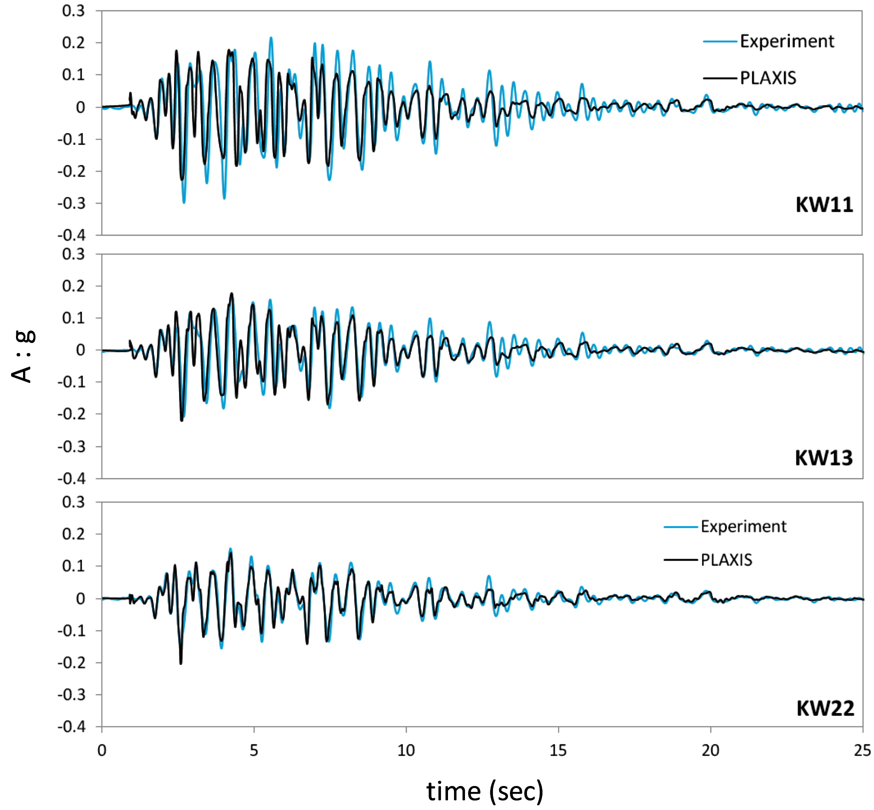

Figure 9. Measured versus predicted acceleration time histories at the soil

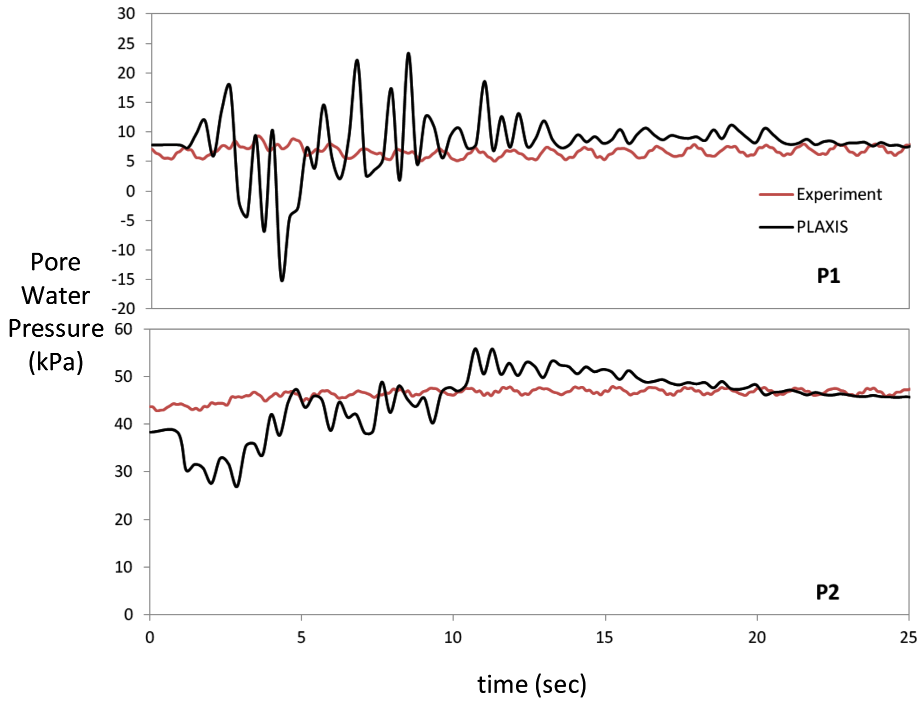

Figure 10. Comparison between predicted and measured pore water pressure time histories (transducers P1 and P2)

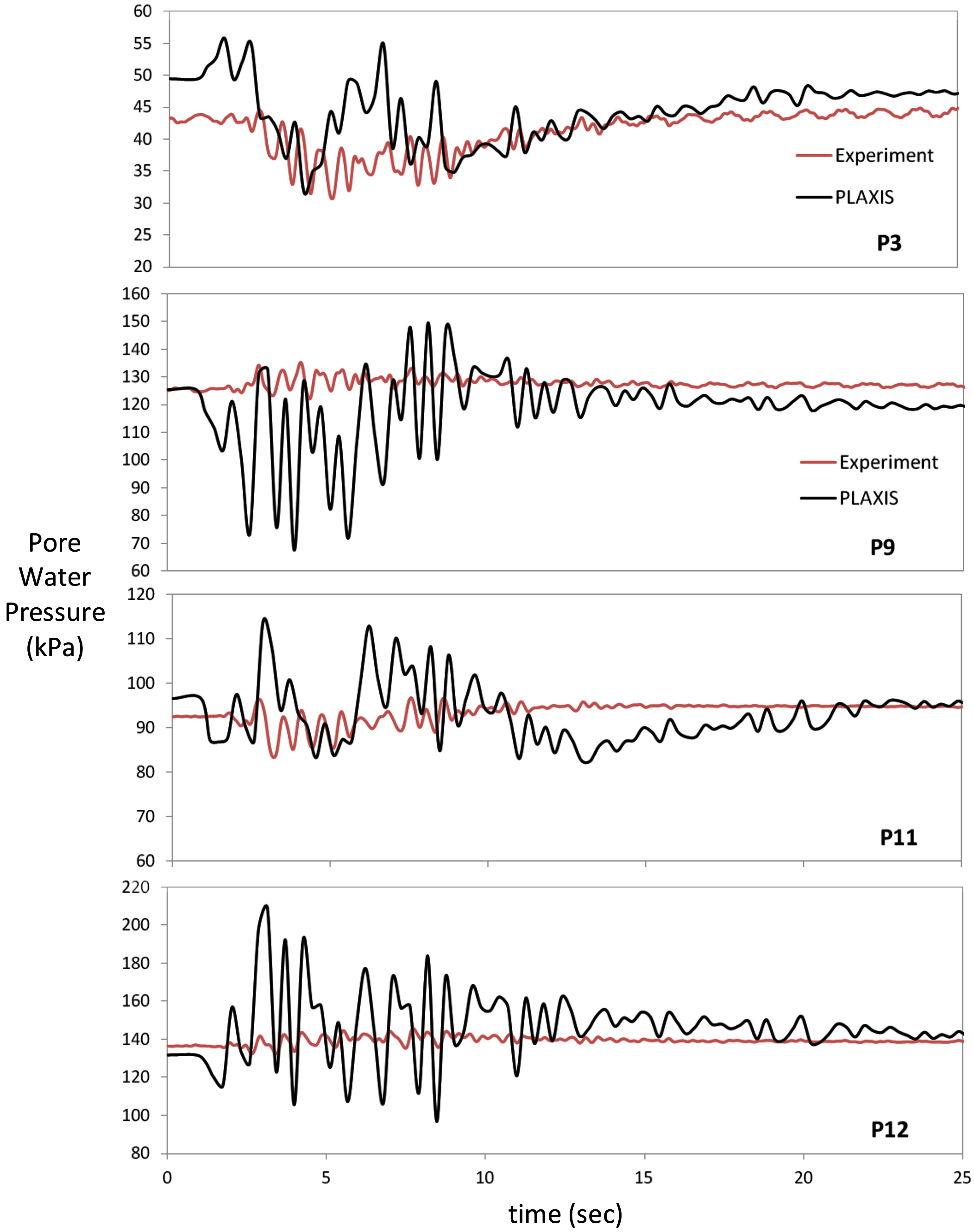

Figure 11. Comparison between predicted and measured pore water pressure time histories (transducers P3, P9, P11 and P12)

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