

## Performance based design of retaining structures – part II: case study of a gravity quay wall

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### ABSTRACT

This paper presents a brief case study investigating the performance of the Kalamata quay wall that exhibited permanent displacement following a  $M_w$  5.9 earthquake. The aim of the investigation was to attempt to predict the observed damage to the wall using available analysis techniques – chiefly simplified analytical rigid block models. Finally a simple dynamic finite element analyses is considered. Results indicate that simplified models provide useful order of magnitude displacements for carrying out performance based design, but may misinterpret the actual mode of wall displacement. Dynamic numerical analyses provide compelling advantages in terms of modelling dynamic response, however results are dependant on model set-up, and accuracy is dominated by uncertainty with the ground motion.

### 1 INTRODUCTION

This paper presents the second part of an investigation into performance based design (PBD) of retaining structures. Following a review of dynamic earth pressure theory and available displacement methods for assessing wall performance, this paper seeks to apply elements of that theory to a simple case history.

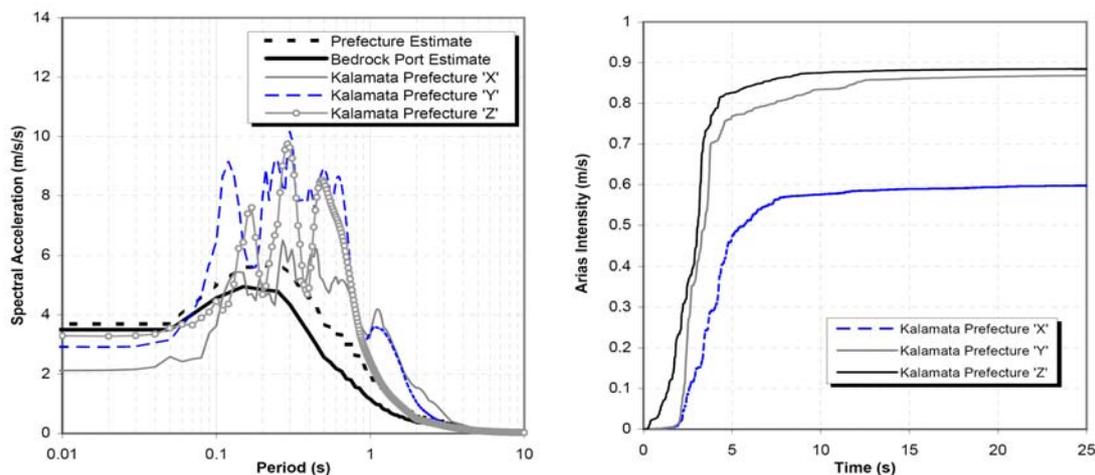


Figure 1: (A) Strong motion and deterministic spectra (5% damped). (B) Husid Plot of Arias Intensity for Prefecture recordings.

### 2 GROUND MOTION

The Kalamata earthquake ( $M_w$  5.9) of 13th September 1986, was caused by a normal fault rupture with an epicentral distance of 9 km from the town of Kalamata (pop. 50,000) in Southern Greece. Response spectra of the accelerograms recorded in the town Prefecture building (over stiff soil profile) are plotted in Figure 1 alongside estimated spectra from the ground motion predictive equation of Akkar and Bommer (2007). The differences between the

predicted and recorded spectra are likely due to dynamic soil-structure interaction, particularly in the medium to long-period range (0.2-2.0 s). Table 1 provides details of the records.

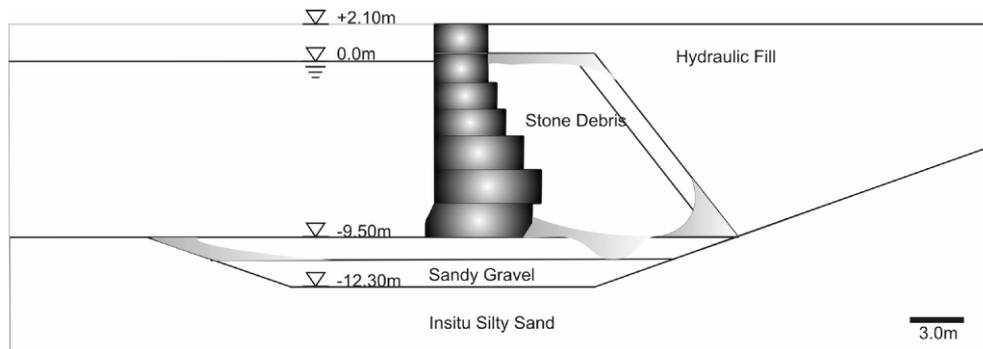
**Table 1: Strong Motion Recordings: Kalamata Prefecture**

Description	PGA (m/s <sup>2</sup> )	PGV (m/s)	PGD (m)	Arias Intensity I <sub>A</sub> (m/s)	Effective Duration (s)
Horizontal (“X”) component (N265)	2.109	0.328	0.079	0.598	3.04s
Horizontal (“Y”) component (N355)	2.911	0.322	0.067	0.868	2.65s
Vertical (“Z”) component	3.251	0.149	0.022	0.885	3.47s

## 2 GEOMETRY AND SOIL PROPERTIES

### 2.1 Wall Geometry and Seismic Performance.

The wall is modelled on a case history of a gravity quay wall at Kalamata Port in Greece, after Pitilakis & Moutsakis (1989), see also PIANC (2002). Construction comprised pre-cast concrete blocks, placed on an excavated platform with placed fill of sandy gravel materials forming foundation layer, prior to backfilling with stone debris and (likely) hydraulic fills. A diagram of the 11.6 m by ~ 6.0 m wall is provided in Figure 2.



**Figure 2: Simplified Model of Retaining Wall at Kalamata Port, Greece (after PIANC 2002)**

The performance in terms of displacement of the wall at the top, was reported to be of the order of  $150 \pm 50$  mm, with rotation of the wall of  $4-5^\circ$ . However, at 11.6 m high, this rotation would result in larger outward displacements at the top of the wall, suggesting either exaggerated reporting, or the tilting did not extend the full height of the wall. No liquefaction was observed.

### 2.2 Design Acceleration

In order to assess ground motions at the base of the wall, in softer deposits at the Port than the Prefecture site, a non-linear 1-D site response analysis (*Oasys* SIREN) on the upper 50 m of the soil profile was conducted. The  $G_0$  profile was derived based on correlation to SPT  $N$  values for backfill and foundation materials, in addition to available shear wave velocity data. Empirical normalised modulus reduction ( $G/G_0$ ) and damping ratio ( $\xi$ ) curves were derived from Stokoe et al. (2004) (accounting for PI, OCR and  $\sigma'_v$ ). Calculated ground motions had PGA of 0.2 g; PGV of 0.244 m/s; and  $I_a$  of 0.58 m/s. A plot of the backfill amplification ratio with depth is provided in Figure 4(A). Average backfill amplification ratio at PGA was 0.77. This is for the free field and ignores any 2D geometric or wall-soil interaction effects. For this reason, dynamic amplification factors in the backfill were taken as unity. Phase affects were ignored.

The  $k_v/k_h$  ratio of 1.1 as observed in the recordings was adopted for the analysis of the wall.

**Table 2: Soil Properties Adopted for Analysis.**

Material Type	Angle of shearing resistance $\phi'$ (°)		cohesion $c'$ (kPa)	Unit Weight (kN/m <sup>3</sup> )	Young's Modulus $E_0$ (MPa)
	LE	FE			
Concrete	E <sup>1</sup>	E <sup>1</sup>	E <sup>1</sup>	23.5	14000
Silty Sand (hydraulic fill)	33	28	0	17	100-200
Sandy Gravel (placed fill)	42	36	0	19.5	180
Silty Sand (in-situ)	-	33.5	2	18.5	200-480
Sandy Gravel (in-situ)	-	31.5	5	19	300-980
Conglomerate (in-situ)	-	E <sup>1</sup>	E <sup>1</sup>	20	1270
Marl (Rock)	-	E <sup>1</sup>	E <sup>1</sup>	21.5	2200-5600

1. Elastic – no failure criterion modelled

LE = Limit Equilibrium

FE = Finite Element

### 2.3 Adopted Soil Properties

For soil properties adopted for the pseudostatic limit equilibrium (PS-LE) analysis, Mohr Coulomb (MC) parameters were estimated based on the soil descriptions and site investigation data, primarily SPT  $N$  values (refer Table 2). For the FE analysis the simple elastic-perfectly plastic Mohr Coulomb constitutive model was used in an undrained analysis. Reduced MC properties were derived to ensure a more appropriate undrained strength,  $S_u$ , would be achieved. If this is not done, the MC model would overpredict  $S_u$  by around 20%.

## 3 PSEUDOSTATIC WALL STABILITY ASSESSMENT

The analysis was carried out following EN 1998 limit state design guidance, but with the recommendations of Matsuzawa et al. (1985) for assessing hydrodynamic pressures. All calculations conservatively assume the hydrodynamic forces in front of the wall and in the backfill are in phase. A “virtual back” was assumed, with an effective wall block  $\gamma$  of 20.4 kN/m<sup>3</sup>, and centroid located 3.27 m back from the toe and 5.64 m above the base. The polar moment of inertia for the wall block was estimated as 9906 m<sup>4</sup>. Friction was assumed to act on the virtual backface at a ratio of  $\delta/\phi = \frac{2}{3}$ , and soil-base interface at a ratio of  $\delta/\phi = \frac{1}{2}$ .

**Table 3: Factors of Safety and critical accelerations for respective failure modes.**

Failure Mode	Static Factor of Safety	Dynamic Factor of Safety ( $k_h = 0.2g$ ) critical acceleration (ratio $k_c/k_h$ )			
		0.33H*		0.6H*	
Overturning	1.8	0.84	0.15g (75-90%)	0.74	0.13g (65-80%)
Sliding	1.3	0.5 0.06g (30-38-%)			
Bearing	77	0 0.146g (73-90%)			

NB: \* Point of action of dynamic earth pressure component from base of wall.

Factor of Safety (FoS) values for both static and seismic stability are presented in Table 3, along with the critical accelerations,  $k_c$ , required to initiate permanent deformation (i.e. when FoS = 1). Sliding was found to be the more critical mode. It is recognised that as the wall slides away from the backfill, wall friction along the virtual back will decrease, providing a reduced contribution to wall stability. In addition, post peak strengths would develop along the soil wedge and basal sliding interfaces tending toward critical state values. Both these effects, as well as cyclic pore pressures generated in backfill and foundation materials would cause the predicted  $k_c$  value to decrease.

Figure 3B presents the rigid block displacement time histories using a Newmark (1965) sliding block analysis, resulting in 115 mm - 175 mm of permanent movement, and are within the range observed. Displacements via outward wall tilt via rotating block analysis (assuming rigid

founding stratum), but due to the wall geometry produced negligible displacements (0-10 mm). Bearing displacements were also considered using a custom approach based on the Newmark sliding block method. Results were sensitive to the input acceleration, with one direction of shaking resulting in 140 mm of permanent outward tilt at the top of the wall.

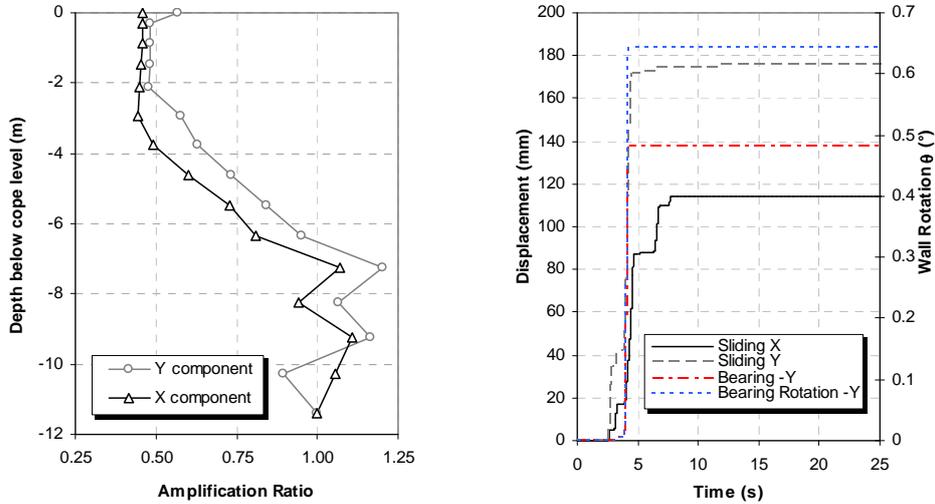


Figure 3: A) Amplification Ratio Profiles from 1-D Site Response Analysis. B) Newmark sliding block displacement estimates including bearing induced displacements.

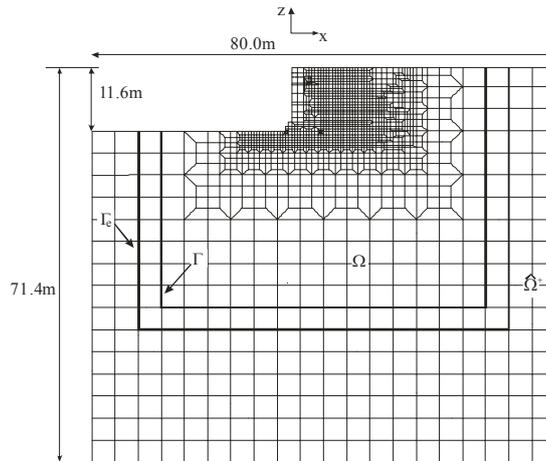


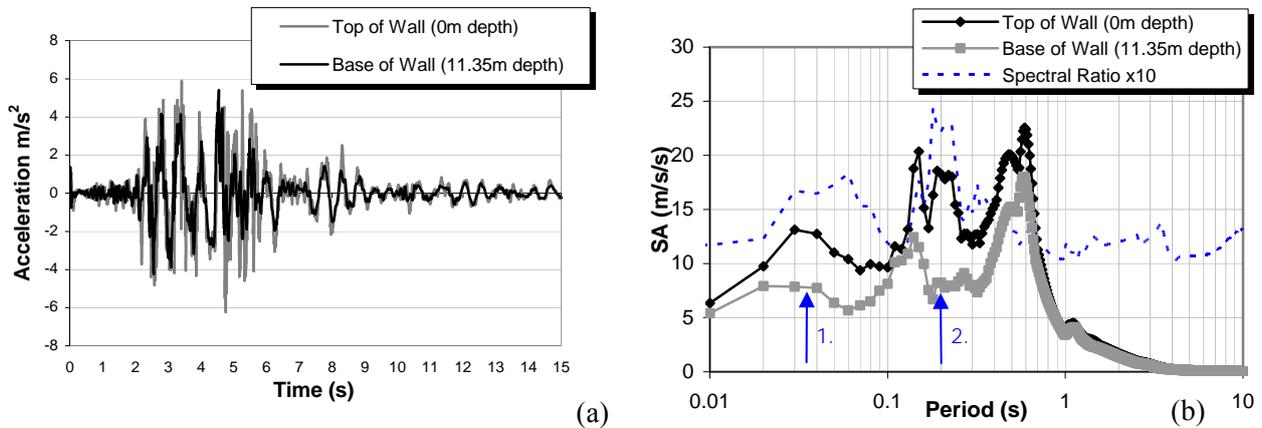
Figure 4. Mesh discretisation (2-D model).

#### 4 DYNAMIC FINITE ELEMENT ANALYSIS

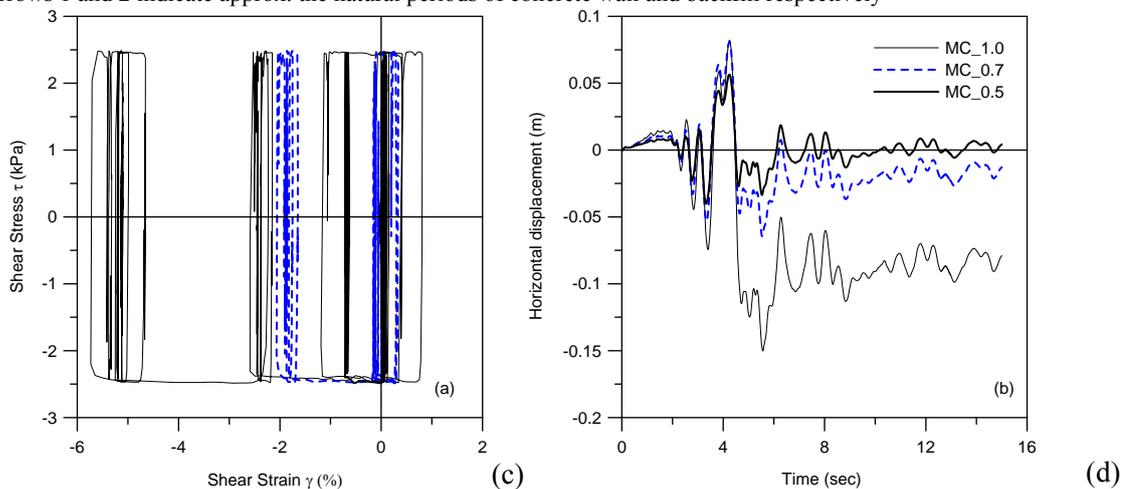
2-D dynamic plane strain analyses were carried out with the mesh discretisation is shown in Figure 4. The mesh generated has 1784 quadrilateral elements with 8 nodes per element. The element size close to the wall of  $0.5 \text{ m}^2$  was used to capture the expected larger soil movements, with as small as  $0.25 \text{ m}^2$  at the corners of the footing. The time integration was performed with the generalised- $\alpha$  method (Chung and Hulbert 1993) and with a time step of 0.01s. The analyses were performed using the Domain Reduction Method (DRM) of Bielak et al. (2003) in conjunction with the standard viscous boundary of Lysmer and Kuhlemeyer (1969). DRM, in conjunction with a conventional absorbing boundary can also be efficiently used in the numerical modelling of geotechnical earthquake engineering problems as an advanced absorbing boundary condition (Kontoe et al. 2008).

The Y component of the record was applied along the bottom boundary of the FE model, and was scaled to 100% (MC\_1.0), 70% (MC\_0.7) and 50% (MC\_0.5) to account for the uncertainty with ground motion attenuation. The soil in the internal area ( $\Omega$ ) was modelled as

elastic perfectly plastic (MC), while the soil in external area ( $\hat{\Omega}^+$ ) and the concrete were modelled as linear elastic materials with Rayleigh damping. The wall was modelled as wished in place with  $K_0$  normally consolidated conditions.



Note: Arrows 1 and 2 indicate approx. the natural periods of concrete wall and backfill respectively



**Figure 5: a) & b) Ground response over height of backfill for the MC\_0.7 analysis. (c) Shear stress-strain curves & hor. displacement histories (d) for three levels of input motion recorded at the ground surface, behind the wall.**

## 5 FE Results

A comparison of ground motion recorded at the top and near the rear base of the wall for the MC\_0.7 analysis and corresponding response spectra (Fig 5a and b respectively) provide an indication of the effect of the modelled soil on the ground response. Long period motion is largely unaffected. Predominant amplifications occurred to medium to high frequency motion (1-100 Hz), particularly at the fundamental frequencies of wall and backfill (marked 1. and 2. in Fig 5b respectively). Figure 5c shows shear stress-strain data close to ground surface in the vicinity of the wall for the three levels of input ground motion. Clearly, the MC model fails to accurately represent hysteretic behaviour of soil under cycling loading. The hysteretic loops should be ovate due to soil non-linearity below yield, not square as shown. Damping in the FE model is much less than in real soils, and thus amplifications predicted are larger than reality, particularly at the higher frequency end of the spectrum. In addition to strain induced damping, reduction in shear stiffness would result in period elongation, not observed in the model.

Figure 5d compares horizontal displacement histories located at the ground surface in the vicinity of the wall, showing strong dependence of residual ground deformations on the amplitude of the input time history.

## 6 DISCUSSION – CONCLUSIONS

A case history of a quay wall that underwent permanent displacements in response to moderate seismic loading has been investigated. Despite limitations with back-analyses, some useful observations can be made for simplified and complex methods for predicting wall performance.

- Rigid block models, commonly used for reducing design accelerations via tolerable displacements for economic limit state design, or prediction of displacement in a PBD methodology, rely presumptively on modes of deformation that may be un-conservative for certain wall geometries; notably where tilting may dominate wall motion –via rotation or bearing failure modes. In addition, strain softening and excess pore pressures reduce critical acceleration,  $k_c$ , from initial assumed values resulting in larger movements.
- Dynamic numerical analysis is a complex tool for modelling performance of geotechnical structures. The simple FE analyses presented showed that uncertainty regarding input motion controls the accuracy of results. Furthermore simple models like Mohr-Coulomb, are suitable for simple FE analyses of static problems, but fail to model dynamic soil behaviour such as; stiffness degradation, hysteretic damping and plastic deformation during unloading. Kinematic hardening or multi-yield plasticity models currently provide the best way forward to model soil dynamics, and research efforts in this direction are ongoing.

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