

A REVIEW OF PERFORMANCE BASED DESIGN PROCEDURES FOR GRAVITY RETAINING STRUCTURES UNDER SEISMIC LOADING

Merrick TAYLOR¹, Stavroula KONTAE², Sarada SARMA³

ABSTRACT

This paper summarises the drivers for performance based design philosophy and implementation in the design of gravity retaining structures subject to seismic loading. A comparison is provided of some of the methods commonly used in practice, and other developments as presented in the literature, from simple pseudo-static limit equilibrium (PS-LE) based tools, through to fully dynamic numerical analyses capable of modelling non-linear soil behaviour. Results of a simple application of some of these PS-LE methods to a selected design problem indicate significant variability between methods, and some ambiguities with regard to the calculation of critical acceleration k_c , when using various formulae in the literature with different design codes. A revised expression for the calculation of k_c for these PS-LE methods is presented. A simple fully dynamic Finite Element Analysis is also considered, using the Mohr Coulomb soil model with Rayleigh damping applied to the input motion. A comparison of resulting displacements against the results of simplified methods indicates that those methods considered provide useful approaches for undertaking PBD, but careful consideration as to the respective methods limitations is required.

Keywords: Retaining Wall, displacement, performance based design,

INTRODUCTION

Regulatory codes and published design guides for seismic design of gravity retaining walls are typically based on simple limit equilibrium (LE) design philosophy. They typically consider the seismic load as an additional pseudo-static (PS) earth pressure. There are a number of shortcomings with this simplified design approach, but its simplicity has a number of perceived benefits. Advanced computational methods (i.e. Finite Element (FE) modelling) have been developing rapidly over the last 30 years and are nowadays a popular analysis tool. However, uncertainties related to the design input parameters (i.e. soil properties under both static and dynamic loading, seismic loading) are such that the numerical precision of advanced numerical methods may be considered with a large degree of scepticism. Hence simplified procedures are generally preferred to computational intensive numerical methods in engineering practice.

On the other hand, the simple design procedures do not provide any indication of deformations occurring to the structure under the design earthquake load (i.e. the *performance* of the structure). In a real world earthquake scenario, movements of the soil and structure will occur under seismic loading, regardless of how over-designed the structure may be. These movements may or may not be acceptable for the performance requirements of the structure. Clearly, the difficulty arises when a critical structure is to be designed (i.e. abutment walls for a bridge designated of 'lifeline' importance, quay walls) where the potential economic loss of serviceability following an earthquake is unacceptably high. In these latter cases rigorous numerical modelling is required.

¹ Geotechnical Engineer, ArupGeotechnics Email: merrick.taylor@arup.com

² Lecturer, Department of Civil & Environmental Engineering, Imperial College, London.

³ Reader, Department of Civil & Environmental Engineering, Imperial College, London.

In the first part of this paper a review of the available methods for seismic design of gravity walls is given. This includes the whole range of methods: from simple pseudo-static limit equilibrium (PS-LE) based tools through to fully dynamic numerical analyses. The second part of this paper details results

SUMMARY OF THE VARIOUS METHODS OF ANALYSIS

The methods for performing seismic analysis of gravity walls presented in the literature are summarised here in tabular format. The methods have been divided into categories based on their mode of analysis – pseudo-static; so called pseudo-dynamic; and fully dynamic. Their derivation, limitations, advantages and shortcomings will be discussed briefly.

Pseudo-static Limit Equilibrium (PS-LE) Methods (Refer Table 1)

Force Based Design

The design procedures in most codes fall into this category, whereby the pseudo-static seismic actions are added to the static problem as external forces, in both horizontal and vertical directions, prior to solving for limit equilibrium with a required Factor of Safety (FoS). The pseudo-static seismic loads are obtained by multiplying the mass of the failing rigid soil block by the design earthquake acceleration (usually PGA, or some fraction of it). Typically a lower FoS than for static design is considered by the design codes as being acceptable for the respective modes of failure of the wall (sliding, tilting/bearing capacity, global, structural). No indication of displacement is possible from the LE method without some further analysis beyond this method (e.g. Newmark (1965) sliding block analyses). Traditional PS-LE methods generally result in '*over designed*' wall sizes to resist the dynamic loads calculated with the specified or required FS under seismic loading, thereby rendering the design method unsuitable from an economic or simply 'practicable' perspective.

Assessing Seismic Displacements

For the calculation of displacements of slope movement when accelerations exceed the critical acceleration for the slope mass k_c , an extension of Newmark's sliding block model (Newmark 1965) has resulted in empirical design formulae fitted to analyses of case histories of slope failures, e.g. Franklin & Chang 1977, Ambraseys & Menu 1988, and others. The Richards & Elms 1979, (R&E) empirical design formulae is derived from selected case studies within Franklin & Chang's database suited to the anticipated small movements (< 1 m) likely to occur to retaining walls. The R&E method relies on Mononobe-Okabe (MO) (Mononobe & Matsuo 1929, Okabe 1926) dynamic active earth pressure theory to determine k_c for the wall block being considered. All these methods are heavily dependent on the value of k_c for the calculation of displacements, and due to their empirical nature, between each other can result in widely varying displacement values (Kramer 1996).

In addition, all these methods prescribe failure to be translational and when applied to wall movements do not consider wall rotation. The rotational mode can however contribute significantly to total wall movements (Nadim & Whitman 1983). From laboratory testing (Lai and Berrill 1982) the R&E method produces reasonable results, albeit consistently over-predicting displacements slightly for translational failures. The Whitman and Liao (1985) reappraisal of the R&E method, in order to take account of the assumptions and uncertainties with the method, developed a probabilistic approach, which produces displacements generally similar to R&E, but of lower magnitude.

Use in Practice:

EC8 and the FHWA design guide for reinforced earth walls (Elias & al. 2001) allow for the reduction of the applied horizontal seismic coefficient based on the concept of acceptable displacements, in a similar manner to the recommendations of R&E, provided that: The wall geometry is not complex; it is not larger than 15 m; nor significant in terms of its importance post an earthquake; and that the design PGA be not greater than 0.3g. Displacements are also limited to greater than 25 mm and less than

300 mm (EC8) or 200 mm (FHWA) due to the fair degree of uncertainty associated with the results of such simplified analyses.

Table 1. Pseudo-static Methods for use in Seismic design of retaining structures.

Mode of Seismic Load Application	Prescribed Soil Behaviour	Analysis Method	Load Derivation	Earth Pressure Theory	Prescribed Failure Mode	Prescribed Point of Action of ΔK_{XE}	Output		Dynamic features modelled				
							Displacements	Forces	Inertia	Cyclic Pore Pressures	Amplifications	Soil Damping	Soil Plasticity
Force based													
<i>Mononobe Okabe (MO) (c. 1929)</i> ¹	P	LE	PGA	MO	T or R	Not spec.	N	Y	N	N	N	N	PP
Typical Design Code ²	P	LE	PGA	MO	T or R	0.3-0.6H	N	Y	Y	N	N	N	PP
<i>Wood (1973)</i> ³	E	LE	PGA	ET	N/A	0.58H	N	Y	Y	N	N	N	N
<i>Wood & Elms (1990)</i> ⁴	E-P	LE	PGA	MO/ ET	N/A	0.5H	N	Y	Y	N	N	N	N
Displacement based													
<i>Franklin & Chang (1977)</i> ⁵	P	LE	PGA/V	N/A ¹	T	N/A ²	Y ¹	Y	Y	N	N	N	PP
<i>Richards & Elms (1979)</i> ⁶	P	LE	PGA/V	MO	T	N/A ²	Y ¹	Y	Y	N	N	N	PP
<i>Whitman & Liao (1985)</i> ⁷	P	LE	PGA/V	MO	T	N/A ²	Y ¹	Y	Y	N	N	N	PP
<i>Ambraseys & Menu (1988)</i> ⁵	P	LE	PGA	N/A ¹	T	N/A ²	Y ¹	Y	Y	N	N	N	PP
<i>Jibson (1994)</i> ⁸	P	LE	I _A	N/A ¹	T	N/A ²	Y ¹	Y	Y	N	N	N	PP
PS FE/ FD	Any	FE/D	PGA	N/A	N/A	N/A	Y	Y	Y	N	N	N	PP

Notes: (Refer Table 2 for Key).

- 1 Enlarged Coulomb wedge based on design acceleration coefficient(s).
- 2 E.g. NZBC B1/VM4. Some codes (FHWA, EC8) allow reduction in hor. seismic coefficient k_h through tolerable displacements, conceptually similar to Richards & Elms (1979) (R&E).
- 3 Derived from Elastic theory.
- 4 W&E (1990) Advised Interpolation of methods 2 and 3 for stiff walls with between 2% and 5% wall deflection with respect to wall height.
- 5 Empirically derived from Newmark analyses of case studies.
- 6 Empirically derived from selected F&C results.
- 7 Modified R&E method taking account of uncertainty probabilistically.
- 8 Solution based on Newmark sliding block, using Arias Intensity instead of PGA.

Pseudo-static Finite Element/ Finite Difference Analysis

The use of FE/ FD numerical methods removes the requirement to prescribe a failure mode in the analysis –as in classic LE. The use of non-linear soil models in the analysis removes the assumptions of full plastic mobilisation as required in the MO theory. The advantage of applying the seismic load as a PS force is the relatively low computational and simpler boundary condition requirements compared to full dynamic analysis. The applied PS force is however based on PGA, and will not represent the true dynamic nature of an earthquake load on the structure. It cannot therefore consider the effects of amplifications, nor soil hysteretic damping, or the development of cyclic pore pressures.

Pseudo-dynamic and Dynamic Limit Equilibrium Methods (Table 2)

The more complex methods (i.e. pseudo-dynamic and dynamic) seek to address the shortcomings of the PS approach to more accurately deduce wall performance. What they both have in common is the use of acceleration time histories to calculate displacements, rather than a PGA-based PS force.

Table 2: Pseudo-dynamic and Dynamic Methods for use in Performance Based Design

Mode of Seismic Load Application	Prescribed Soil Behaviour	Analysis Method	Load Derivation	Earth Pressure Theory	Prescribed Failure Mode	Prescribed Point of Action of ΔK_{YE}	Output		Dynamic features modelled					
							Displacements	Forces	Inertia	Cyclic Pore Pressures	Amplifications	Soil Damping	Soil Plasticity	
<u>Pseudo-dynamic</u>														
Two Block sliding: <i>Zarrabi (1979)</i>	MC	LE	ATH	MO ¹	T	N/A ²	Y ¹	Y	Y	N	N	N	PP	
Rocking block (tilting): <i>Steedman & Zeng (1996)</i>	MC	LE	ATH	MO	R	H/3	Y ¹	Y	Y	N	N	N	PP	
<u>Dynamic</u>														
2D Single block sliding: <i>Newmark (1965)</i>	MC	LE	ATH	N/A	T	N/A ²	Y ¹	Y	Y	N	N	N	P	
3D Single block sliding: <i>Elms (2000)</i>	MC	LE	ATH ^{x3}	N/A	T	N/A	Y ¹	Y	Y	N	N	N	P	
Two block sliding: eg: <i>Miyata & Sugano (2000)</i>	MC	LE	ATH	N/A	T	N/A ²	Y ¹	Y	Y	N	N	Y ⁵	P	
Two block sliding and tilting: eg: <i>Steedman & Zeng (2000)</i>	MC	LE	ATH	N/A	T&R	H/3	Y ¹	Y	Y	N	N	N	P	
Multiblock sliding: <i>Chlimintzas (2003)</i>	MC	LE	ATH	N/A	T	N/A ²	Y ¹	Y	Y	N	N	N	P	
Numerical Analysis:														
Linear	E	FE/D	ATH	N/A	N/A	N/A	Y	Y	Y	N	Y ²	N	N	
Damped Linear	E	FE/D	ATH	N/A	N/A	N/A	Y	Y	Y	N	Y ²	Y ³	N	
Non Linear	MC	FE/D	ATH	N/A	N/A	N/A	Y	Y	Y	N	Y ²	N	P	
Damped Non Linear	MC	FE/D	ATH	N/A	N/A	N/A	Y	Y	Y	N	Y ²	Y ⁴	P	
Fully Non Linear	K*	FE/D	ATH	N/A	N/A	N/A	Y	Y	Y	Y	Y	Y	EwhP	

Key:

LE	Limit Equilibrium.	T	Translation.
FE/D	Finite Element/ Finite Difference.	R	Rotation.
PGA	Peak Ground Acceleration.	E	Linear Elastic.
PGA/V	PGA and Peak Ground Velocity.	P	Rigid Plastic.
I _A	Arias Intensity.	ET	Elastic Theory.
MO ⁽¹⁾	Mononobe Okabe (c.1929) back analysed to consider angle of failure wedge α_{AE} .	MC	Mohr Coulomb Soil Model.
ATH	Acceleration Time History.	PP	Perfectly plastic behaviour only.
Y ¹	Displacements obtained by double integration of excess acceleration above critical k_c .	EwhP	Elastic work-hardening Plastic behaviour.
Y ²	But with excessive amplifications.	K*	Kinematic Soil Model.
Y ³	Rayleigh Damping applied to model and input.	N/A ¹	Dependant on method to obtain critical acceleration k_c as this may depend on MO theory.
Y ⁴	Rayleigh damped input only. (Cannot apply to MC model due to plasticity).	N/A ²	Translation only, no prescription necessary.
Y ⁵	Damping coefficient in equation of motion.	H	Height from base of wall.

Pseudo-dynamic Methods

The pseudo-dynamic methods retain some elements of PS design theory. For example the Zarrabi (1979) two-block model uses MO theory to derive the means to determine the angle of the backfill

wedge, and hence the size and mass of the retained soil block, which changes incrementally throughout the analysis depending on the acceleration at any increment of time Δt during the analysis. This two-block model has been compared to the testing of Lai (1979), and found to provide better agreement than the single-block derived R&E method (Wood and Elms 1990). It does however continue to change the backfill slope angle beyond what is observed in model testing, where a stable failure plane develops and remains at a critical wedge angle during the remainder of the shaking. This perhaps helps to explain why it too tends to over-predict displacements compared to scale model testing (Simonelli & al 2000).

The rocking block model of Steedman and Zeng (1996) requires the calculation of a k_{cr} (critical rotational acceleration) coefficient, which in turn is dependant upon MO theory. It also assumes a constant location of the resultant dynamic lateral earth pressure, which is a large degree of inaccuracy for the model, as the imposed moments are sensitive to the point of action of the dynamic earth pressures. Despite this, they did manage to show reasonable agreement to scale model testing.

Dynamic Methods

Increasingly complex fully dynamic modified Newmark approaches have been developed to incorporate two-block sliding, sliding and tilting (2 degree of freedom blocks), and even multi-block sliding analyses incorporating mass transfer between blocks (Chlimintzas 2003). These developments are research models, and have not as yet translated to commercially available tools for engineering practice. Their limitations centre on the LE analysis method that requires a prescribed mode of failure in order to analyse the problem. For real world performance of a retaining wall, a combination of modes is often witnessed – sliding and tilting combined with the initiation of bearing failure beneath the footing is not uncommon. These models also cannot inherently consider the effects of non-linear soil behaviour, nor cyclic pore pressure generation without some additional means to reduce soil strength during the analysis. However the advantages over more complex numerical methods are that in relative terms the processing and modelling requirements are lower, whilst being more advanced models than the simple empirical formulae as used in current design practice.

Dynamic Numerical Methods

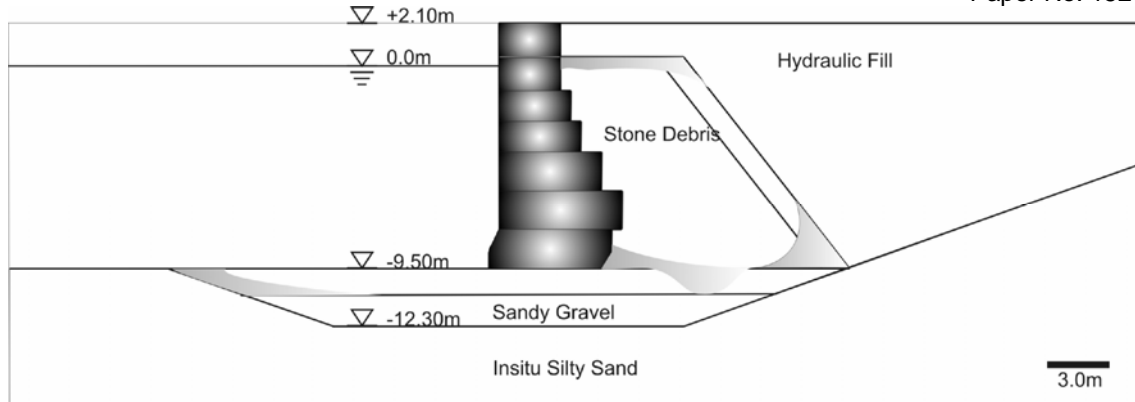
Dynamic numerical analysis by finite element or finite difference methods have become increasingly common in order to predict retaining wall performance under seismic loading, and the advantages are that it need not prescribe the mode of failure of the wall, and thus is capable of analysing for any combination of deformation modes simultaneously. Thus the behaviour of the structure is more correctly modelled than the LE derived approaches. Any constitutive model may be used to model soil behaviour, including kinematic hardening models that more accurately simulate the hysteretic soil behaviour under dynamic loading, with development of cyclic pore pressures, in addition to elastoplastic behaviour below yield. These latter models do tend to be quite complicated and require many input parameters, making their calibration difficult and time consuming. For this reason the use of simpler constitutive models such as Mohr Coulomb are commonly still employed with dynamic numerical analyses albeit with modelling difficulties.

ANALYSIS DETAILS

Description of the Employed LE Model

Geometry

The geometry of the wall is modelled on a case history of a gravity quay wall at Kalamata Port in Greece, reported and previously analysed by Pitilakis & Moutsakis (1989). This wall was selected as, rare among case histories, it had both geometry and soil properties clearly reported. For detail of the wall performance and results of soil testing at the site, the reader should refer to the aforementioned paper. A simplified diagram of the wall is provided in figure 1, with material properties adopted provided in Table 3.



**Figure 1. Simplified Model of Retaining Wall at Kalamata Port, Greece
(after Pitilakis & Moutsakis, 1989)**

Table 3: Parameters adopted for pseudostatic analyses:

Description	Symbol	Value	Units
Backfill Angle of Shearing Resistance	ϕ'	32	°
Foundation Angle of Shearing Resistance	ϕ_b'	36	°
Ratio Mobilised Angle of Shearing Resistance to Soil Shearing Resistance along concrete-soil interface:	δ_b/ϕ_b'	0.67	-
Ratio for friction backface of wall	δ/ϕ'	1	-
Unit weight of concrete blocks	γ_{conc}	24	kN/m ³
Unit weight of soil backfill	γ_b	19	kN/m ³
Water level (below wall crest)	w/l	-2.1	m
Peak Ground Acceleration ¹	PGA	0.334	g
Peak Ground Velocity ¹	PGV	0.33	ms ⁻¹
Arias Intensity	I_a	1.056	ms ⁻¹

Notes: 1. From combined Main shock and Main Aftershock records.

Peak Ground Acceleration

The PGA and PGV values were obtained from the combined records of the **M** 5.9 main shock at Kalamata on the 13th of September 1986, and the **M** 4.9 aftershock of 15th September 1986. The reasons for considering the main aftershock is that, due to proximity of the aftershock epicentre being approximately 10 km closer than the main shock, the resulting record is more of an impulse load with a higher recorded PGA than for the main shock. For these seismic design calculations, 0.67*PGA was adopted as the applied k_h for use in the calculation of MO dynamic earth pressure coefficients. Vertical acceleration coefficient k_v was taken as 2/3* k_h , and for calculation of k_{vc} , 2/3* k_c was assumed.

Calculation of Critical Acceleration Parameter k_c

Formulae in the Literature & observed caveats.

There is one definition for the critical acceleration k_c , but several ways one may use to calculate k_c . From the definition –“The acceleration required to initiate motion in a resting mass”, the obvious way to determine k_c is to back-analyse the horizontal seismic coefficient k_h at which point the wall stability FoS reduces to unity. This can be done readily using a spreadsheet with the formulae involved – either manually or using one of the inbuilt iteration routines. For hand calculation however, a formula was devised by Whitman (1979) that aims to determine k_c through iteration, as k_c is also the acceleration coefficient used to calculate the MO earth pressure coefficient at the point of failure. This method appears to provide quite low k_c values compared to other methods:

$$k_c = \frac{FoS - 1}{\left(\frac{FoS}{\tan \phi_b} \right) + \left(\frac{C_{AE}}{K_A} \right)} \quad (1)$$

Where: FoS is the Factor of safety under static loading condition (using K_0 -as recommended by NAVFAC 1986 - or K_A pressures? –presumably the latter is most appropriate for the relative resistance to forming the active wedge – as this is assumed by MO theory, and as used in the remainder of the expression). ϕ_b is the angle of shearing resistance of the soil/ wall interface at the base of the wall on which the wall will slide. Note that Parameter $C_{AE} = (K_{AE} - K_A)/k_c$. An alternative expression is provided by Kramer (1996). It too relies on M-O theory to determine the loads on the wall at occurrence of failure, and thus requires iteration to obtain k_c . It includes geometry of the wall in the calculation, replacing the calculation of the static FoS as in (1).

$$k_c = \left[\tan \phi'_b - \frac{P_{AE} \cos(\delta + \theta) - P_{AE} \sin(\delta + \theta)}{W} \right] g \quad (2)$$

Where: W is the weight of the wall, and δ is the angle of shearing resistance at the soil-wall interface behind the wall. θ is the wall backface angle. Regardless of the way the geometry is modelled ('virtual back' of wall $\theta = 0^\circ$ vs. sloping $\theta \approx 12.2^\circ$); fairly consistent k_c values result from using this method, provided the mass of the block considered is appropriately proportioned.

However, the formula contains an erratum; it is missing a $\tan(\phi'_b)$ term applied to the vertical component of the applied dynamic earth pressure P_{AE} . The authors propose the following revision:

$$k_c = \left[\tan \phi'_b + \frac{[P_{AE} \sin(\delta + \theta) - u] \tan(\phi'_b) - P_{AE} \cos(\delta + \theta) + P_w}{W} \right] g \quad (3)$$

Where u is the pore water pressure at the base of the sliding block, and P_w is the water pressure acting on the face of the block when P_{AE} calculated with total unit weight (should the soil be sufficiently impermeable or restrained from movement). W remains the total weight of the wall block.

Caveats with respect to Back-analysed k_c values.

One minor ambiguity is with regards to the inertia mobilised. The FHWA design guide for reinforced earth structures (Elias & al. 2001) considers that only $0.5 \cdot H$ of the base width is mobilised by inertia loading of the wall mass, and only $0.5 \cdot P_{AE}$ (dynamic active earth pressure), as they consider it unlikely that the peak dynamic earth pressure will be in phase with the peak inertia load of the wall itself. These assumptions in turn result in a different loading configuration, and hence a different value of k_c if back analysis is performed. This is in contrast to the New Zealand Building Code B1/VM4, which considers the full inertia load of the gravity wall and the full dynamic active earth pressure occurring simultaneously, as for the above iterative formulae for k_c (Equations 1 – 3). For some wall geometries (where $H \approx$ width w) this will be a significant difference.

Results

For the estimation of displacement under the seismic motion that this wall was subjected to in 1986 (M 5.9 main shock and M 4.9 main aftershock), in Table 4 the various methods of pseudo-static analysis are compared against a fully dynamic Newmark sliding block analysis, for various ways of calculating k_c . Equation 1 does not account for total stress condition under seismic scenario resulting in quite different k_c value and hence resulting displacements. Equation 2 produces approximately the same result at Equation 3 only by co-incidence in this example; with a different geometry the results would be quite different. This analysis ignores any effects of cyclic loading on increased pore pressures beneath the foundation, reducing shear strength. If we were to take account of these factors, k_c ought to reduce and increase displacements; however some empirical equations may be based on slope failures

with elevated pore water pressures, and this may explain the large variability between the different empirical equations presented. It is difficult to assess when the dynamic water pressure will be in phase with the inertia of the wall, and it is considered unlikely that the water body will be in phase when PGA occurs. Dynamic water pressures affecting the free face of the wall were thus ignored to simplify the analysis.

Table 4: Results of Pseudostatic Analyses varying k_c calculation method.

$A_{\max} = 0.334g$, $V_{\max} = 0.33 \text{ m/s}$

	k_c^1	k_c^{1*}	k_c^2	k_c^3	k_c^4
k_c :	0.300g	0.295g	0.233g	0.294g	0.298g
k_c/A_{\max} :	0.9	0.88	0.61	0.88	0.89
Method	Wall Displacement in sliding (mm)				
Richards & Elms 1979	4	5	12	5	5
Whitman & Liao 1985	1	1	6	1	1
Ambraseys & Menu 1988	3	4	57	4	3
Jibson 1994	4	4	11	4	4
Newmark (1965) Dynamic Analysis	1	1	7	1	1

Notes: 1. k_c method by Equation 2, after Kramer (1996),
1*. k_c method by Equation 3, revised equation (this paper).
2. k_c method by Equation 1, after Whitman (1979), when FoS calculated using K_A pressures ($FoS = 2.9$).
3. k_c via back analysis of the PS analysis performed to NZBC B1/VM4. Inertia = full mass of wall, dynamic earth pressure = full. Does not consider vertical component of P_{AE} .
4. k_c via back analysis of the PS analysis performed to FHWA (Elias & al. 2001). Inertia over width $0.5H$, $0.5*$ dynamic earth pressure. Does not consider vertical component of P_{AE} .

Description of the Employed FE Model

Spatial and temporal discretisation

This section details a series of 2D dynamic plane strain analyses of the quay wall illustrated in Figure 1. The adopted FE discretisation is shown in Figure 2. The mesh generated has 1784 quadrilateral elements with 8 nodes per element. The element size close to the wall of 0.5 m^2 was used to capture the expected larger soil movements, with as small as 0.25 m^2 at the corners of the footing. The time integration was performed with the Generalised- α method (Chung and Hulbert 1993) and with a time step Δt of 0.01s.

Boundary conditions and Input Ground Motion

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). The DRM is a two-step sub-structuring procedure that was originally developed for seismological applications to reduce the domain that has to be modelled numerically by a change of governing variables. However, the 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 2006, Kontoe *et al* (2006)).

In the first step, a 1D FE model was used to calculate and store the free-field response in terms of effective forces at various depths. In the step I analysis the excitation was applied along the bottom boundary of the 1D model, while all nodes of the same elevation on the two side boundaries were constrained to deform identically. In the step II analysis, the stored effective forces were applied to the corresponding nodes of the step II 2D model (Figure 2), which are located between the boundaries Γ_c and Γ . It is important to note that the fictitious boundary Γ separates the computational domain into two areas Ω and $\hat{\Omega}^+$. The formulation of the DRM is such that the perturbation in the external area $\hat{\Omega}^+$ is only outgoing and corresponds to any deviation of the step II 2D model from the step I 1D

model. The standard viscous boundary was applied along the lateral boundaries of the step II model and both horizontal and vertical displacements were restricted along the bottom boundary.

The N80E component (PGA=0.291g) of the Kalamata Prefecture record was incrementally applied in the horizontal direction along the bottom boundary of the FE model. Since there is no bedrock strong motion record in the vicinity of the quay wall, the surface accelerogram was scaled to 100% (MC_1.0), 70% (MC_0.7) and 50% (MC_0.5) to account for the uncertainty regarding the strong motion attenuation with depth.

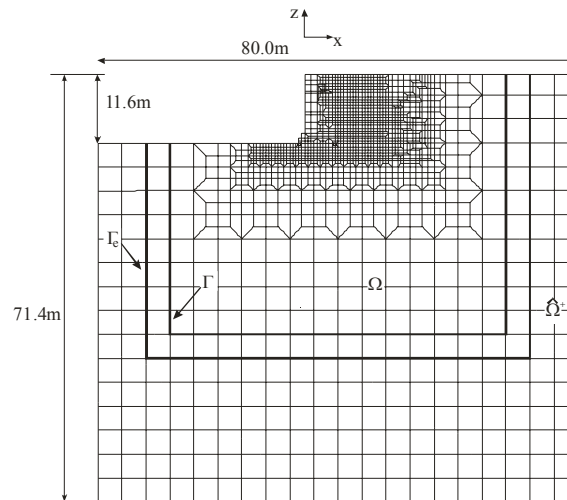


Figure 2. Mesh discretisation (2D model).

Table 5: Soil Properties Adopted for F.E.A.

Material Type	Angle of shearing resistance ϕ' (°)	Apparent cohesion c' (kPa)	Unit Weight (kN/m ³)	Young's Modulus E_0 (MPa)
Concrete	LE ¹	LE ¹	23.5	14000
Silty Sand (hydraulic fill)	28	0	17	100-200
Sandy Gravel (placed fill)	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)	32	10	20	1274
Marl (Rock)	38	20	21.5	2200-5600

1. Linear Elastic – no failure criterion modelled

Constitutive model

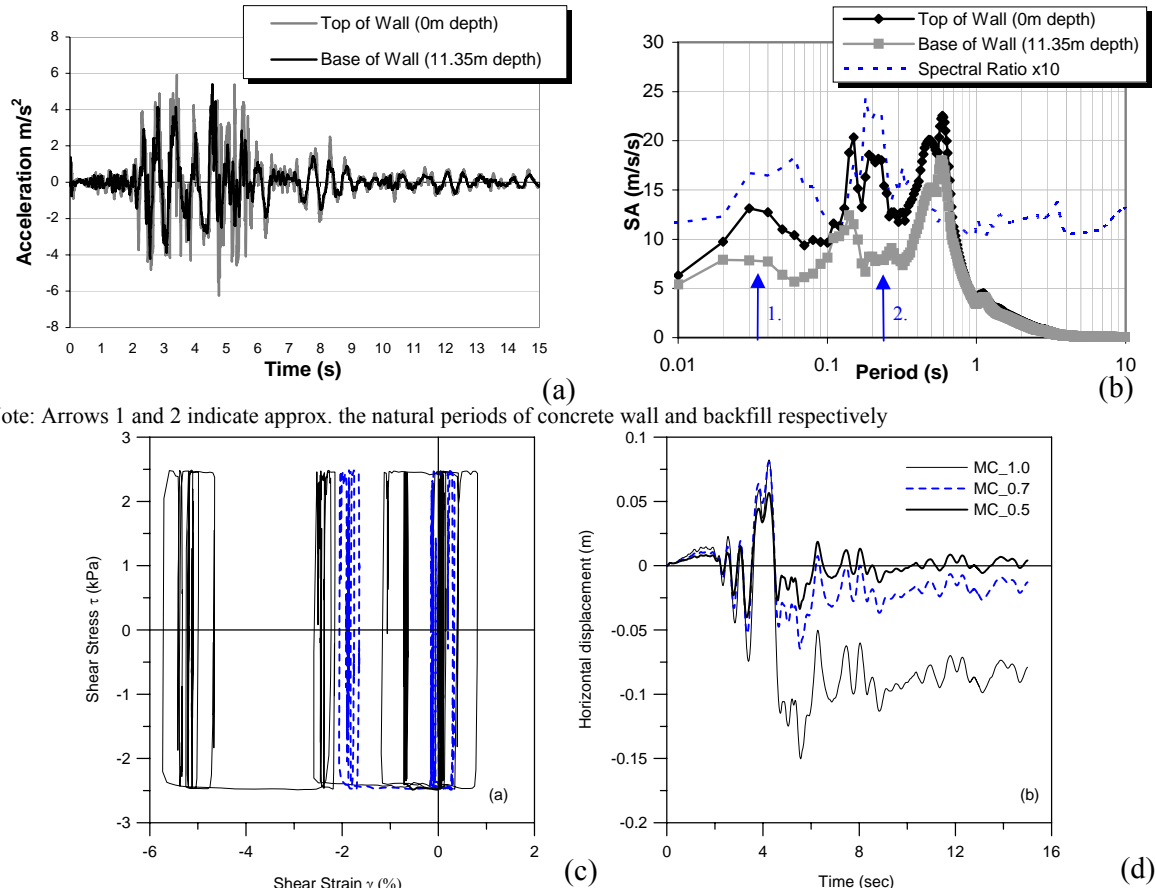
For the 1D analysis, the soil was modelled as linear elastic material with 8% Rayleigh⁴ damping. For the 2D model, the soil in the internal area (Ω) was modelled as elastic perfectly plastic Mohr – Coulomb (MC) material, while the soil in external area ($\hat{\Omega}^+$) and the concrete were modelled as linear elastic materials with 8% Rayleigh damping. Undrained conditions were assumed in all the analyses. Table 5 presents the soil properties used for the analysis. The wall was modelled as wished in place with K_0 pressures determined assuming normally consolidated conditions for the natural marine deposits and hydraulic fills. The material properties adopted for the analysis are present in Table 5. E_0 and ϕ' values were determined from interpreted shear wave velocity and SPT test data obtained after the earthquake.

⁴ The Rayleigh damping coefficients were taken as $A=1.15$ and $B=3.092E-3$ resulting in a target damping ratio of 8% for the first and the third mode of the model.

FE Results

Ground Response

A comparison of the horizontal accelerations recorded at the top ($y = 0$ m) and base ($y = -11.35$ m) of the wall (at $x = 7$ m) for the MC_0.7 analysis and the corresponding response spectra (Fig 4a 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 (1-100Hz range), particularly at the fundamental frequencies of the wall and backfill.



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

Figure 4: a) & b) Comparison of ground response over height of backfill for the MC_0.7 analysis. Shear stress-strain curves (c) and horizontal displacement histories (d) for three levels of input motion recorded at the ground surface, behind the wall.

Figure 4c shows the shear stress-strain curve for an integration point ($x=6.66$ m, $z=-0.394$ m) close to ground surface in the vicinity of the wall for the three levels of input ground motion. Clearly, the MC constitutive model fails to accurately represent the hysteretic behaviour of soil under cycling loading. The hysteretic loops should be ovate due to soils elasto-plastic behaviour below yield, not square as shown. Damping is therefore much less than in real soils, and thus amplifications predicted by the FE analyses are larger than reality. Predictably, the mobilised plastic shear strains appear highly dependant on the intensity of the applied earthquake record. In a similar way, Figure 4d compares the horizontal displacement histories for the three input records computed at a node ($x=7.0$ m, $z=0.0$ m) located at the ground surface in the vicinity of the wall. The displacement time histories indicate that most of the displacement that occurs during the seismic event is recoverable for the MC_0.5 analysis,

but not for the MC_1.0 analysis.

Mode of Movement

As mentioned earlier, one of the advantages of the FE method over the simple limit equilibrium procedures is that there is no need to prescribe the mode of failure of the wall, and thus is capable of analysing for any combination of deformation modes. Indeed, the plots of displacement vectors in Figure 5 obtained at different time instances from the MC_0.7 analysis illustrate both translational and rotational modes of oscillation.

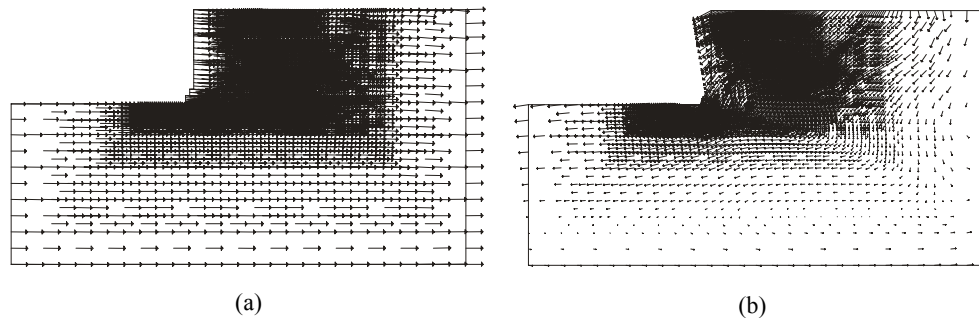


Figure 5. Snapshots of displacement vectors showing translational mode (a) and rotational mode (b) (MC_0.7 analysis).

DISCUSSION – CONCLUSIONS

The advent of performance based design procedures for retaining walls during the late 1970s has led to the development of methods of varying complexity for either; reducing design loads for economic design by taking account of tolerable displacements, or for assessing actual movements under a design seismic event. For reducing design loadings, PS-LE methods have been developed and whilst conservative for assessing sliding displacement, rely on a number of assumptions that may render the method un-conservative for particular wall geometries; particularly where tilting mode of failure may dominate wall motion, and cyclic pore pressures will reduce k_c significantly. Varying methods of calculating k_c for these PS approaches have been considered in this paper, and a revised expression is presented (Equation 3). Where wall designs deviate from the standard wall geometry used to derive these expressions, it is recommended a simple derivation be carried out to check appropriateness of the equation being adopted, as PS-LE methods are heavily dependant on the k_c parameter.

Dynamic numerical analysis brings with it a complex suite of requirements for modelling performance of geotechnical structures accurately. The simple FE analyses presented in this paper showed that the uncertainty regarding the input motion controls the accuracy of the numerical results. Furthermore simple elasto-plastic models like Mohr-Coulomb are generally adequate for simple FE analyses of static problems, but fail to capture important features of the soil behaviour when subjected to cyclic loading such as; stiffness degradation, hysteretic damping and plastic deformation during unloading. For these situations a kinematic hardening model currently provides the best way forward to model soil dynamics numerically, and research efforts in this direction are ongoing.

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