

# DISPLACEMENT BASED SEISMIC DESIGN FOR LARGE GRAVITY RETAINING WALLS

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## ABSTRACT

The use of factor of safety (FOS) design approaches based on limiting equilibrium is conventional practice for designing retaining structures, but experience has shown that there can be significant uncertainty and a wide range of performance achieved when using force-based design approaches for seismic design. Such shortcomings were highlighted during the design of large counterfort wall retaining structures for the Port Botany Expansion project in Sydney, Australia. Extending 21.5 metres in height, the counterfort structures posed a significant challenge when evaluating stability against bearing failure under seismic loading, based on FOS criteria alone. In order to better understand the apparent stability concerns, recourse to dynamic finite element modeling was made to more accurately assess potential failure modes and wall performance.

The dynamic modeling provided greater insight into more realistic failure modes and helped to dispel fears of seismically induced bearing failure in the traditional sense of a global mechanism. It also served to justify the use of Newmark sliding block analyses to characterize dynamic wall behaviour. Based on equivalency of the dynamic finite element and Newmark sliding block results, an appropriate pseudo-static force to represent seismic loading was established. This rationalized approach to seismic design demonstrated overall adequacy in design, overcoming the initial shortcomings of the FOS design approach.

## 1 INTRODUCTION

Port Botany is Australia's second largest container port, located on the northern shore of Botany Bay approximately 12 km south-east of Sydney's city centre (Figure 1). A billion dollar expansion of the port facility has been underway since 2008, namely the Port Botany Expansion (PBE) project. This design-build project involves reclamation covering an area of approximately 63 hectares to construct an additional container terminal with 1850 m of wharf face to accommodate five new post-panamax shipping berths.

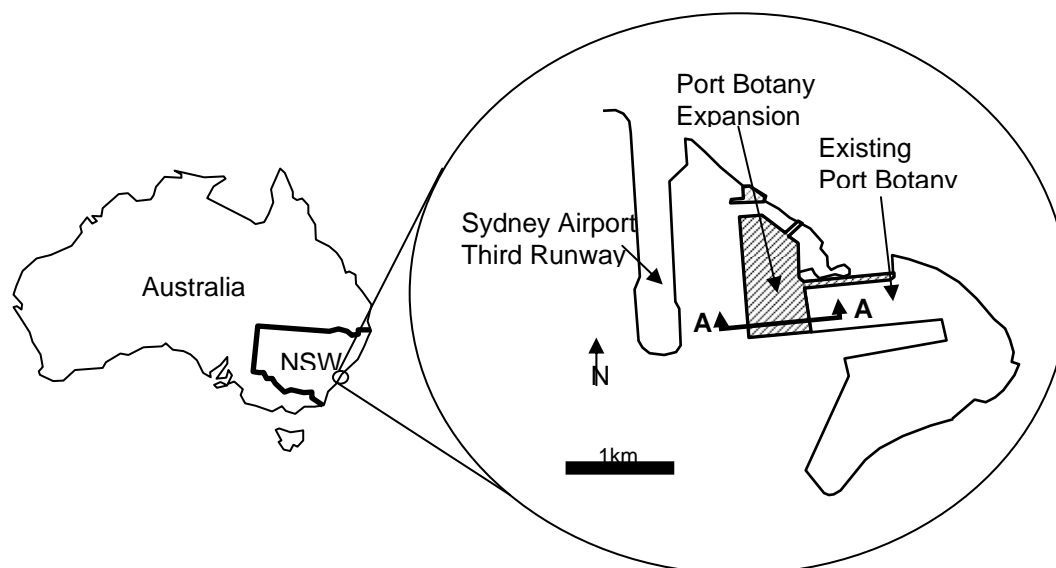


Figure 1: Location and Plan Layout of Port Botany Expansion Project

Construction of the new wharf face has been undertaken using large precast concrete, free-standing retaining wall structures, referred to as counterfort units. These are placed side by side and fixed at their crest by a 1.5 m deep continuous reinforced concrete cope beam, to form the solid edge of the reclaimed land against which vessels will berth.

Given the scale of the counterfort units, each measuring 20 m high by 15 m deep by 9 m wide, assessment of both stability and movement under imposed loadings were prominent aspects of the design. Prescriptive design

requirements included consideration of bearing capacity under seismic loading and its effect on stability. The owner required seismic assessment design both on the basis of a FOS (force-based) approach and using finite element (displacement-based) methods. Results using these approaches were not immediately compatible, necessitating a more refined design approach to address stability and movement considerations in a more consistent manner. This aspect of the design is the focus of this paper, wherein an approach to bridge the gap between force-based and displacement-based design methods is presented using the Newmark sliding block model. Commentary on the seismic coefficient used for pseudostatic analysis is also provided, being a key design parameter.

## 2 COUNTERFORT UNITS

An isometric view of a counterfort unit relative to a person of typical height is shown in Figure 2, together with a corresponding finite element model used for structural design. Immediately apparent is the significance of the counterfort units from both a physical and design standpoint. Buttress components transfer the forces from lateral earth pressures acting on the wall down to the base, and the base in turn is restrained from moving by vertical earth pressures derived from the self weight of the retained material above it. In addition to retaining reclamations, the wall elements, linked by a substantial cope beam, also serve to support vertical crane loading as illustrated in the general arrangement illustration shown in Figure 3.

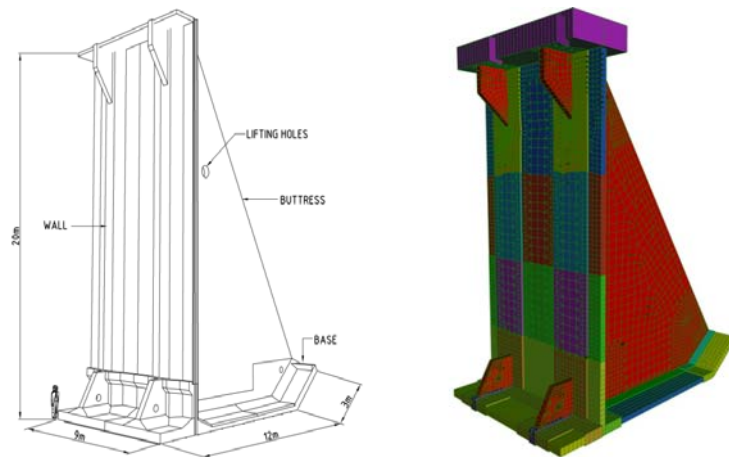


Figure 2: Individual counterfort unit on the left and a corresponding Strand7 computer model on the right. Total weight of unit is 640 metric tons.

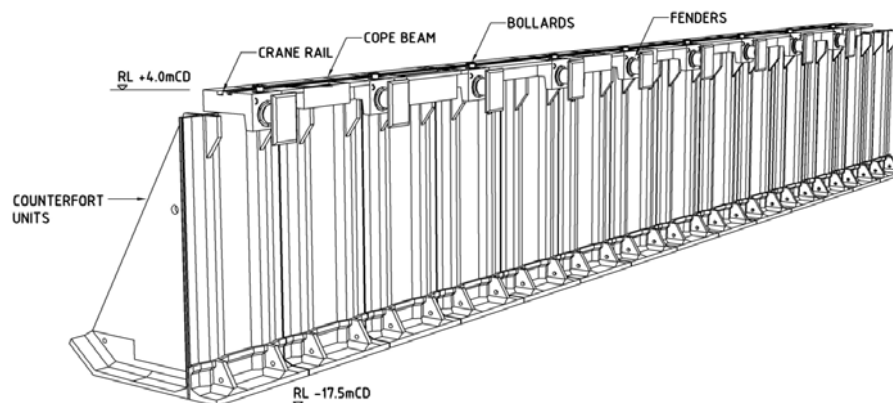
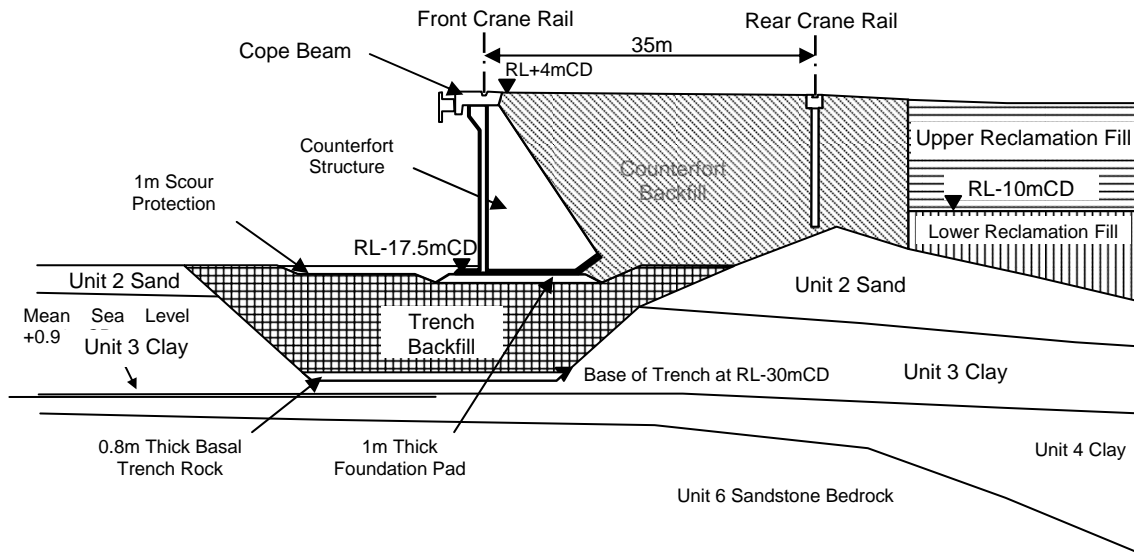


Figure 3: Illustration of counterfort berth structure with cope beam, fenders, bollards and container crane rail.

Backfilled trench foundations are required under most of the counterfort units due to the presence of unsuitable *in situ* fissured clay material close to founding levels. Dredged sand is used for the replacement trench backfill and is vibro-compacted to a dense condition following placement to provide sufficient strength and stiffness for supporting needs. Vibro-compaction of dredged backfill material is also undertaken in the zone immediately inland of the counterfort wall to satisfy stability and serviceability requirements. A typical cross-section of the counterfort berth structure illustrating the general design configuration is shown in Figure 4.



Note: 0m Chart Datum = - 0.925m Australian Height Datum

Figure 4: Typical cross-section of the counterfort berth structure illustrating the general design configuration.

### 3 DESIGN OVERVIEW

The owner’s design requirements for the PBE project works were defined in the Project Scope and Technical Requirements (PSTR), providing minimum technical, operational and performance requirements. Seismic requirements relevant to counterfort design included:

- A minimum limiting equilibrium FOS of 1.1 for potential global, sliding, overturning failure modes under seismic loading. FOS for seismic bearing capacity was not specified, but was reviewed during design development;
- Completion of a site-specific seismic hazard study;
- Design for stability and liquefaction under the ultimate design earthquake event of 1 in 1000 annual exceedance probability (AEP);
- Design for a serviceability design earthquake event of 1 in 100 AEP;
- Dynamic analyses considering spectral shape factor, soil dynamic and inertial loading; and
- Prediction of displacements and levels of damage for the 1 in 1000 AEP and 1 in 100 AEP design earthquake events.

Elements of the counterfort berth structure were also subject to vertical settlement and horizontal movement limits at various times following construction completion. Most important was maintaining the crane gauge within a static  $\pm 30$  mm front-to-rear gauge tolerance at 20 years post-handover. The absolute horizontal front crane rail movement (at the top of the counterfort unit) and rear crane rail (offset 35 m inland) was limited to a static value of 40 mm at 20 years post-handover. Following discussions with the designer about the spread of likely seismic displacements, the owner specified that the port facilities should be able to be brought back into full operation after a 1 in 1000 year earthquake event with minimal repairs being required.

### 4 SITE SPECIFIC SEISMIC HAZARD STUDY

A site-specific seismic hazard study was carried out by a specialist sub-consultant, who produced site hazard curves for peak ground acceleration (PGA) and response spectra values for design earthquake events. The hazard curves were derived using historical seismicity for earthquake magnitudes between 5.25 to 7.25, and site to earthquake source distances of less than 300 km. Design PGA values are summarised in Table 1.

Table 1. Design Values for Peak Ground Acceleration

AEP	Horizontal Peak Ground Acceleration (g)	
	At Rock Level	At Ground Surface Level
1 in 100	0.039	0.045
1 in 500	0.088	0.099

1 in 1000	0.125	0.133
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In order to quantify the effect of local soil profile variations within the site and fully assess inertial effects on the counterfort structures, dynamic analyses were performed using input motion at rock level compatible with the rock PGA and acceleration spectra for the 1 in 100 AEP and 1 in 1000 AEP design earthquake events, as described further below.

## 5 DEVELOPMENT OF DESIGN APPROACH

Following standard practice, the limiting equilibrium analyses simulated the earthquake as a statically applied horizontal force, considered as a dimensionless horizontal acceleration coefficient, expressed as a fraction of gravity, which equates to the horizontal force when multiplied by the weight of the potential failure mass. Results of this pseudostatic approach indicated the counterfort unit as being most vulnerable to bearing capacity failure rather than sliding or overturning failure modes. Somewhat lower bearing FOS values (in some cases near-unity) were apparent compared to more favourable sliding and overturning cases, provoking discussion on tolerable FOS and potential modes of bearing capacity failure. In deliberating the meaning of bearing capacity in the context of the counterfort structure, it was argued that full bearing failure could not develop during an earthquake without invoking fully global mechanisms (shown conceptually in Figure 5) that were not realistic. It was therefore considered that local bearing pressures, toe yielding and corresponding wall movements were more rationally assessed using dynamic analyses, rather than a conventional global bearing FOS.

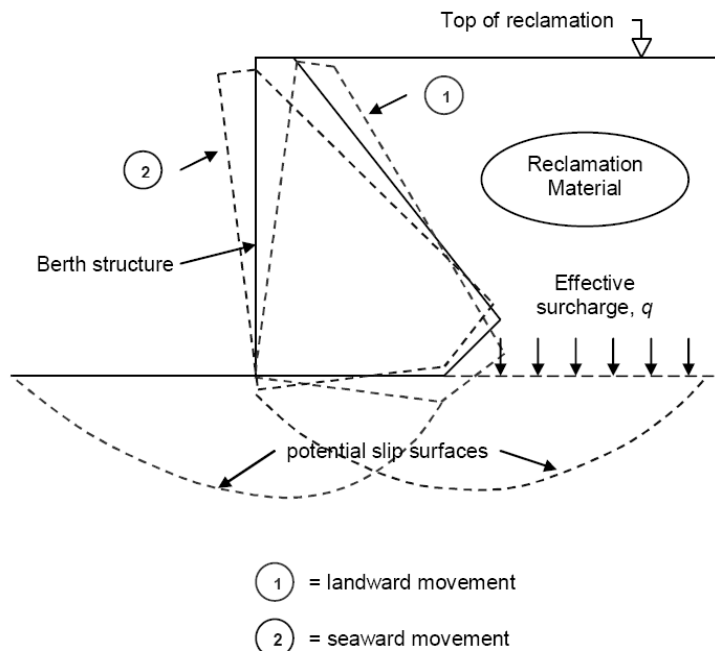


Figure 5: Conceptual failure mechanisms for conventional bearing capacity shear failure considered unrealistic.

## 6 DYNAMIC ANALYSIS

Dynamic numerical modelling was performed using the program PLAXIS. Only a very brief description of the modelling work is provided here. The design analysis cross-section is shown in Figure 6. Given the infrequent seismicity in Australia and poor instrument coverage, a range of appropriate U.S. sourced earthquake records were provided as a basis for developing appropriate design input motion at the base of the model. Earthquake time-history records were all scaled to have a PGA of 0.125 g at rock level (base of the model), corresponding to the 1 in 1000 AEP event. Dynamic shear moduli and Rayleigh damping parameters were determined through calibration between PLAXIS and SHAKE models comprised of the same, representative horizontal material layering.

Assessment of seismic displacements utilized a Mohr-Coulomb soil model in addition to the elastic soil model employing the calibrated dynamic shear moduli and Rayleigh damping parameters. This quantified permanent deformation and was considered to provide a reasonable approach for assessing dynamic behaviour under the level of shaking imposed. Dynamic degradation of soil strength and stiffness was not considered given that compaction requirements were specified to safeguard against potential earthquake-induced liquefaction.

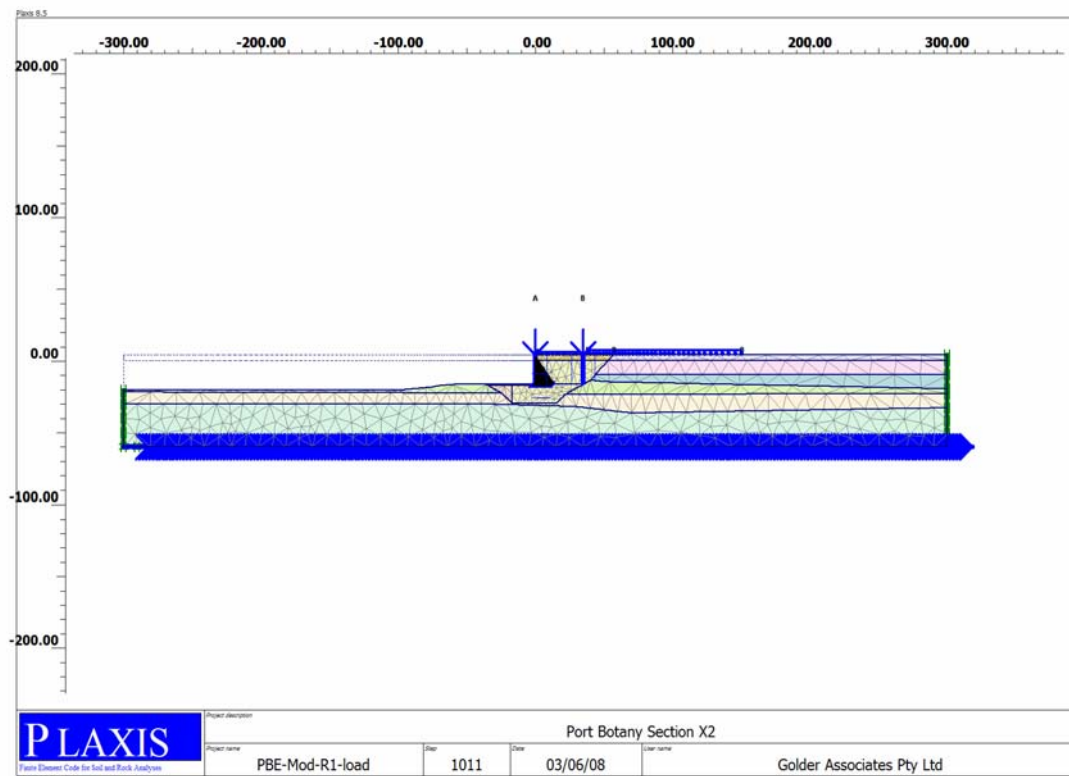


Figure 6: PLAXIS dynamic model.

Typical results of the PLAXIS modelling are shown in Figures 7 and 8. These indicated that dynamic movement was associated with localized exceedence of soil strength at the toe and heel regions of the counterfort structure, as characterized by the occurrence of plastic failure denoted by the open circles in Figure 7. The extent of plastic failure shown in Figure 7 was typical of the maximum extent predicted during the 1 in 1000 AEP event. Rotational (tilting) movement associated with this localized yielding was small, such that a predominantly translational mode of movement behaviour was observed. This observation showed that the seismic response of the wall involved sliding (as shown in Figure 8), rather than a global rotational bearing failure mechanism.

## 7 ADOPTED SEISMIC DESIGN APPROACH

The dynamic analysis results reinforced the notion of more localized bearing behaviour that had been suspected of the counterfort structure. A method was required to reconcile the more onerous design implications of the pseudostatic analysis work with the dynamic analysis. This required a shift in focus to consideration of the value of the seismic coefficient assigned and a means of rationalizing its reduction. This was resolved by depicting the counterfort unit as a Newmark sliding block model (Newmark, 1965), prompted by the predominantly translational mode of movement exhibited in the dynamic analysis and reasoning that the self-stabilizing nature of the counterfort unit would also act to enforce such behaviour. It is also noted that use of the Newmark sliding block model is not unfamiliar to retaining wall design, as apparent from the review of commonly adopted seismic coefficient recommendations provided in Table 2.

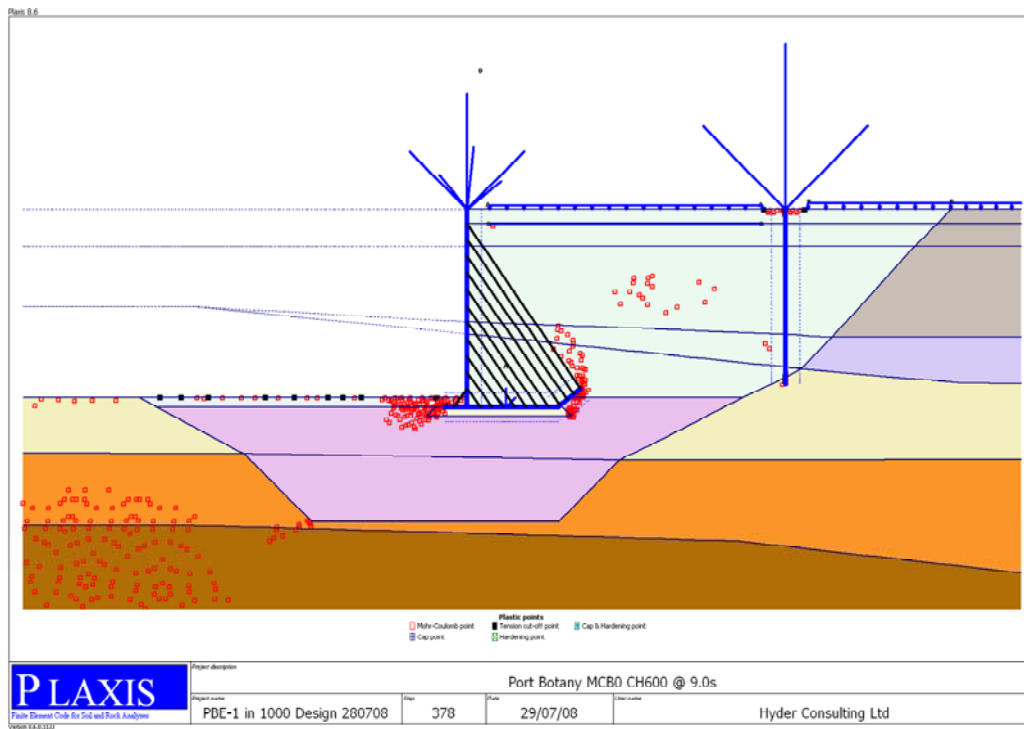


Figure 7: PLAXIS results showing typical plastic failure (denoted by open circles) following 1 in 1000 AEP event.

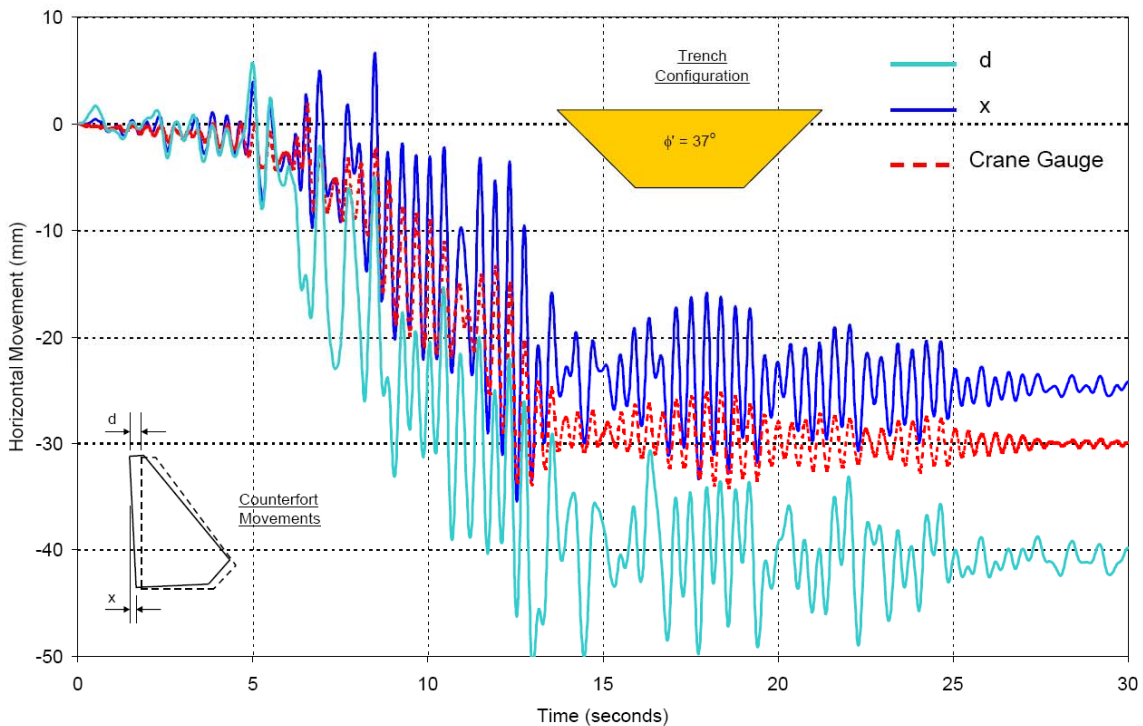


Figure 8: Typical horizontal movement versus time behaviour in PLAXIS for 1 in 1000 AEP event.

Table 2. Review of horizontal seismic coefficient recommendations

Reference	Horizontal Seismic Coefficient	Comments
PIANC (2001)	1.0 PGA 0.33 PGA	Earthquakes with PGA up to 0.2 g Earthquakes with PGA above 0.2 g
Hynes-Griffin and Franklin (1984)	0.5 PGA	Tolerable displacements where FOS > 1 using Newmark analyses
Wood and Elms (1990)	1.0 PGA	Reduced loading is acceptable if the structure is designed to move outward (can be assessed using Newmark)
Eurocode 8 (1998)	0.5 PGA 0.67 PGA 1.0 PGA	Up to 300 mm displacement Up to 200 mm displacement Displacement not possible
AS4678 (2002)	0.5 PGA	Where EQ PGA is more than 0.3 g, the likelihood and potential displacement of structure should be assessed using Wood and Elms (1990)

Note: PGA = peak ground acceleration.

The Newmark approach depicts relative movement between a block and the base material upon which it is supported, when the base material is subjected to earthquake excitation. Relative movement occurs when a threshold acceleration of the block is exceeded, defined as the pseudostatic acceleration coefficient required to reduce the sliding FOS to unity. Pseudostatic analyses indicated that the threshold acceleration values for sliding, overturning and bearing failure modes were 0.2 g, 0.2 g and 0.09 g, respectively. A design PGA = 0.15 g was assigned to the counterfort units for the 1 in 1000 AEP event, based on PLAXIS results which indicated a higher surface acceleration in the vicinity of the berth structure compared with rock level acceleration.

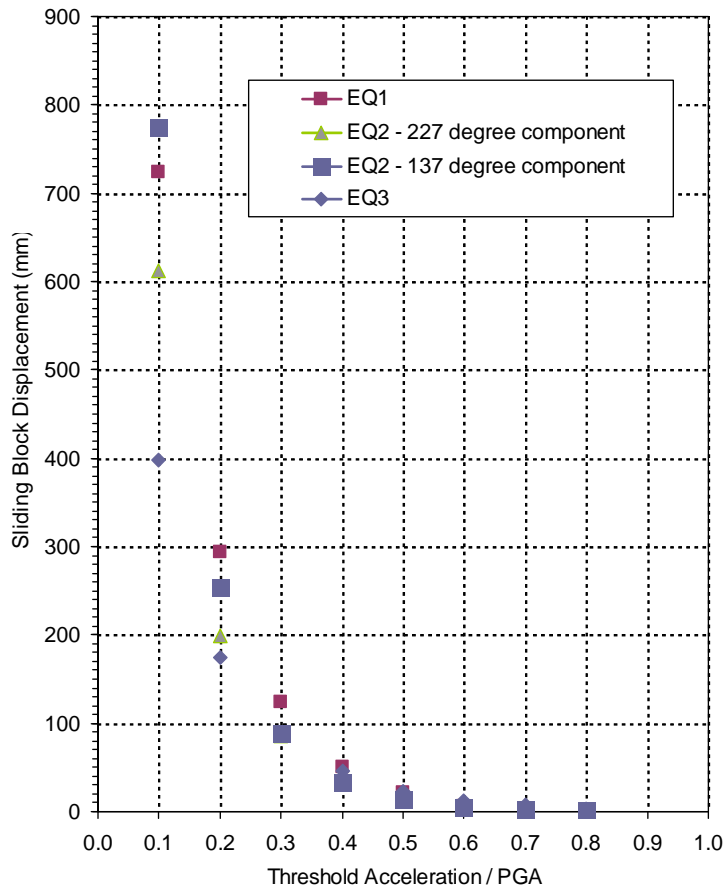


Figure 9: Newmark displacements for varying threshold acceleration and 3 different time-history acceleration inputs (EQ1, EQ2, EQ3)

Using horizontal movement of the counterfort unit as an indicator of bearing-related movement, Newmark sliding block analyses using prescribed design earthquake time-histories and varying threshold acceleration values were undertaken as shown in Figure 9. These indicated that a 50% reduction (corresponding to 0.5 on the horizontal axis of Figure 9) in PGA resulted in horizontal displacements in the order of 30mm, compatible with the movement indicated by the PLAXIS dynamic analyses so a 50% reduction in PGA was considered equivalent to designing the counterfort unit using pseudostatic limit equilibrium methods with the knowledge that momentary exceedance of resistance would occur during the 1 in 1000 AEP event, culminating in permanent movement in the order of 30 mm. This provided the basis for adopting a 50% reduction in PGA for pseudostatic design of the counterfort berth structures.

## 8 DISCUSSION AND CONCLUSIONS

The Newmark sliding block analysis work justified a 50% reduction in PGA to achieve a consistent and rational design to the satisfaction of both force-based and displacement-based design requirements. This required dynamic modeling to quantify soil response and inertial wall behaviour, together with an appropriate and sound basis for using a Newmark sliding block model. The outcome was that the designers argued successfully for adoption of a performance specification in terms of displacement-based design for critical seismic events. The benefits of this displacement-based seismic design include providing a logical, defensible alternative to poorly suited conventional pseudostatic limit equilibrium bearing analyses and providing a tangible measure of seismic wall performance, bench-marked against established design methods. This displacement based approach provides a robust tool to define performance objectives and enables owners to specify performance and manage operational risk by nominating crane gauge (or other critical movement parameters) under specific earthquake events.

## 9 ACKNOWLEDGEMENTS

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