

## DOCKSIDE CONTAINER CRANE-QUAY STRUCTURE INTERACTIONS

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### ABSTRACT

Container traffic continues to grow worldwide at about 8 percent a year and large feeder ports will load and unload the entire ship's cargo at one berthing. As a result, the operational sophistication of the terminals that serve the largest ships is particularly critical. The front line of a terminal is its container handling quay cranes, which require rigorous structural design and stringent serviceability. New quays must be designed to carry the new cranes and allow for uninterrupted use of the port and harbour facilities. The paper presents important issues of time domain seismic analyses of port structures. The dockside crane is simplified as a 2DOF (two-degree-of-freedom) simple stick model, which possesses approximately the same dominant frequencies and participating masses specified by the crane manufacturer. The analysis shows noticeable amplification effects by the retained soils and local soil stratum. Responses of the crane, walls, and tie rod are discussed.

Keywords: seismic analysis, crane-quay structure interactions

### INTRODUCTION

Container traffic continues to grow worldwide at about 8 percent a year (Jordan, 1995). To keep up with this growth, ports and shipping lines are ordering larger ships. Post-panamax ships with 16 containers abeam and 4800 TEU are now standard. Large feeder ports will load and unload the entire ship's cargo at one berthing and, as a result, the operational sophistication of the terminals that serve the largest ships is particularly critical. The front line of a terminal is its container handling quay cranes, which require rigorous structural design and stringent serviceability. New quays must be designed to carry the new cranes and allow for uninterrupted use of the port and harbour facilities.

The operational requirements revolve around the berthing of large container vessels and holding them along side to enable containers to be accurately placed on to or lifted from the vessel. Container operations are undertaken 24 hours per day while vessels are on the berth. This requires a design that involves low maintenance for the quay structure and particularly the crane rails on which the quayside cranes operate. In the event of an earthquake the crane rails have limited flexibility to be easily re-aligned provided the permanent movement in the quay structure is limited.

Pseudo-static method has been widely adopted by practicing engineers to design quay structures under seismic conditions (EAU, 1996; USACE, 1992). However, as the mass of a dockside crane is generally significant, seismic design of port facilities ought to consider crane-quay structure interactions, which can not be easily considered by the pseudo-static approach. Indeed in the performance-based design of port facilities, structures of higher performance grade should be assessed by using more sophisticated methods. PIANC (2001) recommends that time domain seismic analysis should be carried out in the standard/final design stage for performance grade A or S quay structures.

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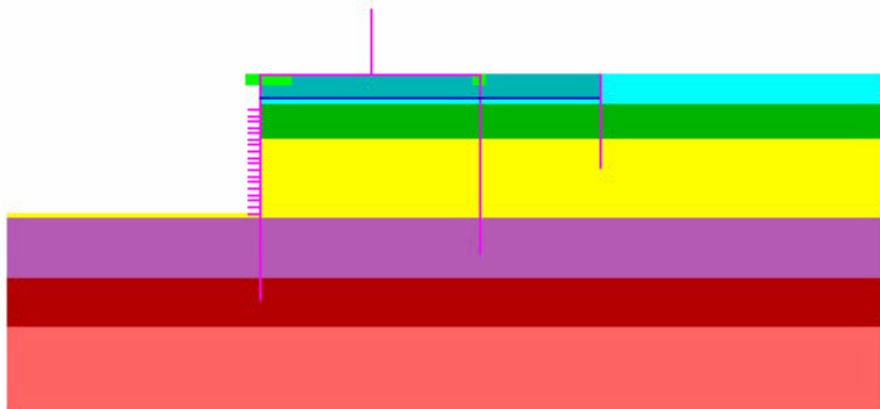
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In practice, recent advancement of computation techniques has enabled engineers to carry out rigorous seismic analysis without difficulties.

This paper presents a rigorous assessment of interactions between a dockside crane and an anchored quay structure. Important issues of time domain seismic analyses of port structures are discussed. The dockside crane has been simplified as a 2DOF simple stick model, which possesses approximately the same dominant frequencies and participating masses specified by the crane manufacturer. Noticeable amplification effects on the crane and quay structure are discussed.

## ANCHORED QUAY WALL SYSTEM

A typical quay wall system (a quay wall, anchor wall, rear crane pile, and tie rod) is schematically given in Figure 1. As shown in the sequence, the dockside crane can be represented by a simple stick model (a horizontal beam and a vertical column), supported by the quay wall and crane pile. The hydrodynamic effect can be represented by added masses, which have been modeled by a series of simple-supported beams that are free to move in the horizontal direction. One of the frequent reasons of anchored quay wall failures is excess seaward displacement of the quay wall, caused partly by inadequate passive soil resistance to restrain the anchor wall. It is therefore of practical importance to locate the anchor wall at a sufficient distance from the main wall so that the stability of both the quay wall and anchor wall is secured.



**Figure 1: Anchored Quay Wall System**

Port facilities are often located in areas susceptible to liquefaction. Seismic design of quay walls are therefore strongly influenced by liquefaction hazards. In general, there are three different types of liquefaction effects that can damage the quay structure; namely, passive wedge liquefaction, active wedge liquefaction, and liquefaction below base of the wall (Day, 2002). Liquefaction-induced settlements can also damage the port facilities (e.g. aprons, storage facilities). When liquefaction is an issue, the geometrical extent of liquefaction must be considered in the design. Indeed, it is a general requirement that the construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake. As a result, an assessment should be carried out in early stages to determine the liquefaction susceptibility of the project site. If liquefiable soils exist at the project site, remediation of liquefiable soils can be an effective technique to improve the performance of quay wall structures to earthquake loading. PIANC (2001) has given design approaches for determining the range of soil improvement and techniques to assess the influence of treated zones on the existing structures. However, the choice of ground improvement method should

take the depth and thickness of liquefiable soils into account. The current study assumes that liquefaction in the retaining soils and the foundation is unlikely to occur.

The inertia forces induced by an earthquake are of short duration and alternate in direction many times. Experimental work on dynamic earthquake pressure (Kawamura et al., 1987) showed that the thrust and distribution of the lateral earth pressure vary mainly with the seismic magnitude, angle of wall friction, magnitude of wall movement and different modes of wall movement. Clearly, the earthquake-induced loading on the retaining wall is closely related to soil-structure interaction, which can not be easily considered by the pseudo-static method. For a realistic analysis it is necessary to consider the time history of the ground motion.

## **NUMERICAL MODELLING AND INPUT MOTIONS**

Established calculation methods for analyzing retaining structures subject to seismic loading are the analytical type pseudo-static approach and numerical techniques. In the pseudo-static analysis the seismic-induced earth pressure is usually calculated by the MO approach (Mononobe and Matsuo, 1929; Okabe, 1926), which considers only force equilibrium and assumes a hydrostatic distribution of the earth pressure, uniform acceleration, and no resonance. Because the pseudo-static approach for anchored quay walls can be carried out by hand calculation only in very few highly idealized situations, there has been a great emphasis placed on the use of numerical techniques for either pseudo-static type analyses or time domain analyses. PIANC (2001) recommends that, for the standard/final design, pseudo-static analyses can be used only for Grade C structures. Port structures of higher performance grade (A or S) should be assessed by using time domain seismic analysis.

In the current analysis the response of an anchored quay wall system to earthquake is calculated by the FLAC software (Itasca, 2002), which is a two dimensional explicit finite difference program for engineering mechanics computation. In the numerical model soils are modelled as Mohr-Coulomb materials. Interface elements are used to represent the potential separation/sliding between the walls and retained soil. Viscous boundaries and free field boundaries are adopted to provide the necessary radiation condition and eliminate outward propagating waves back into the model. Indeed, the numerical model can rigorously calculate the earthquake-induced earth pressure, which is directly related to the displacement of the soil-wall interface. Of course, dynamic amplification by local site condition and phase effect have also been taken into account. Important modeling considerations are given below.

### **Element Size**

Both the frequency content of the input motions and the wave velocities of soil materials will affect the numerical accuracy of wave transmission. In practice, for accurate representation of wave transmission through a model, the spatial element size should be smaller than approximately one-tenth to one-eighth of the wave length associated with the highest frequency component of the input motions (Kuhlemeyer and Lysmer, 1973). ASCE (2000) recommends that the cut-off frequency for geotechnical earthquake engineering problems should be no less than 10Hz.

### **Boundary conditions**

Many geotechnical problems can be idealised by assuming that the regions remote from the area of interest extend to infinity, the problem of soil-structure interaction is a typical example. As the capability of computer is limited, the unbounded theoretical models have to be truncated to a manageable size by using artificial boundaries. In practice, numerical models for the anchored quay wall system should be extended to a sufficient depth below the ground level and to a sufficient width to consider local site effects and soil-structure interactions. The quiet (or absorbing) boundary developed by Lysmer and Kuhlemeyer (1969) has been adopted in the FLAC model to eliminate wave reflections from the truncated boundaries.

### Hydrodynamic effects

Based on Westergaard's approach (1931), effects of hydrodynamic pressure can be approximately considered as added masses acting with the quay wall. The added mass increases parabolically with depth and is defined by

$$m(y) = \frac{7}{8} M_w \sqrt{hy} \quad (1)$$

where  $m(y)$  is the variation of mass with depth  $y$ .  $M_w$  is the mass density of water and  $h$  is the overall depth of the water. As shown in Figure 1, the added mass can be reasonably modeled as a simple-supported beam, which possesses the corresponding mass  $m(y)$  and is free to move in the horizontal direction.

### Damping

Material damping in soils is generally caused by its viscous properties, friction and the development of plasticity. Indeed, the role of the damping in the numerical models is to reproduce in magnitude and form the energy losses in the natural system when subject to a dynamic load. A 5% frequency dependent Rayleigh damping is used for soil materials. However, a 4% mass only Rayleigh damping is assigned to the crane, wall structures, and tie rod.

### Time steps

To complete the numerical solution, it is necessary to integrate the governing equations with respect to time in an incremental manner. The time step of the solution procedure should be sufficiently small to accurately define the applied dynamic loads and to ensure stability and convergence of the solution. In the current analysis the timestep is approximately  $10^{-6}$  second.

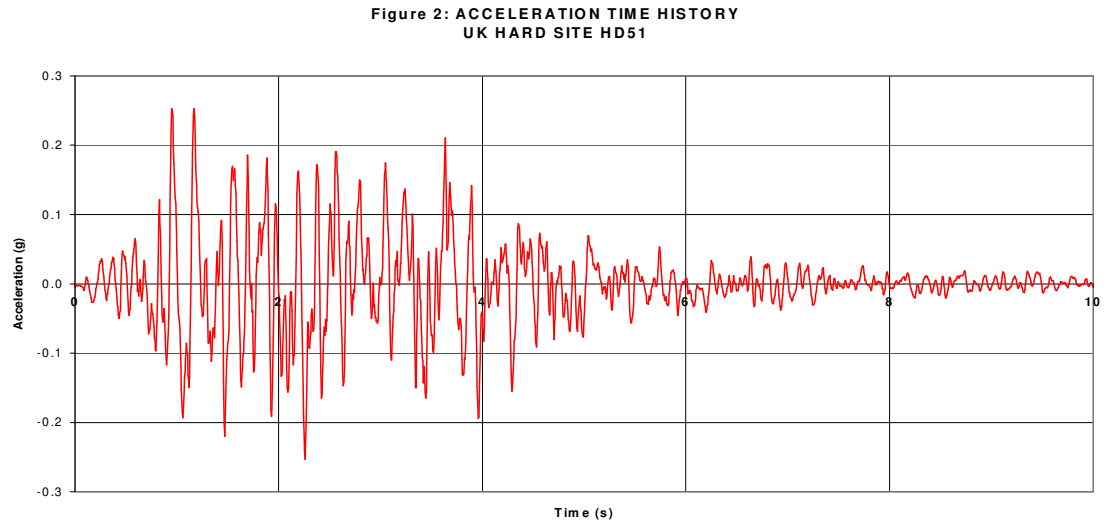
### Deconvolution process

In order to consider soil structure interactions, numerical models for dynamic geotechnical problems should include a sufficient depth of foundation soils. Input motions are then applied at the lower boundary of the model. An important step of time domain seismic analyses is to determine the input motions at the model base from the available ground motions. This process, known as a deconvolution process, can be carried out by analyzing a 1D soil column, with identical geological strata, by using for example SHAKE package (Schnabel et al., 1992).

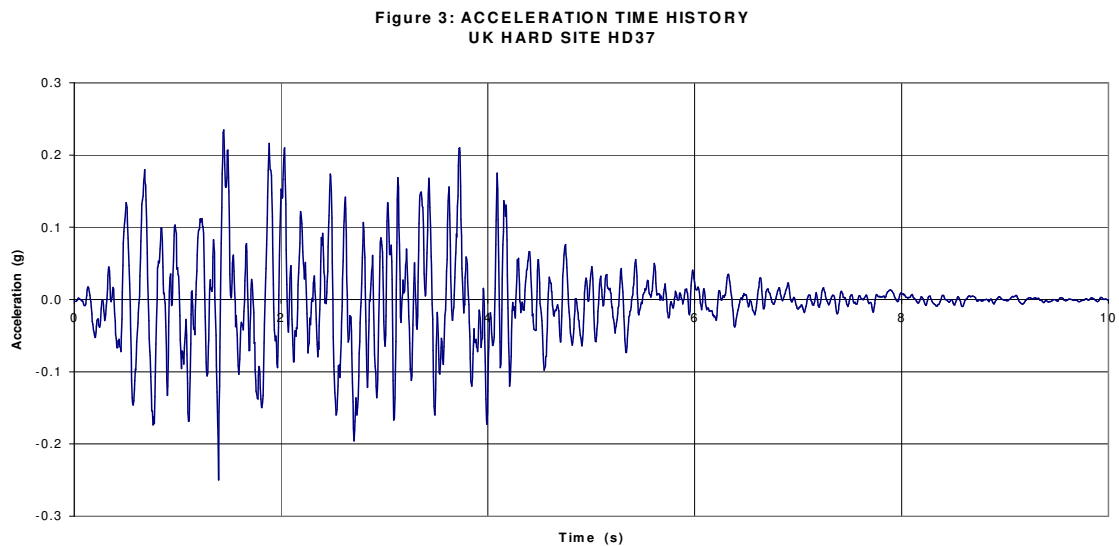
### Uncertainties in input motion

Two of the seismic input motions considered in the current analysis are given in Figures 2 and 3. They are derived from the statistically independent synthetic acceleration time histories with 5% damping (BNFL, 1995). These time histories have a duration of 10 seconds with strong motion occurring between approximately one and five seconds. The horizontal input is 0.25g while the vertical input equals two-thirds of the horizontal input, as recommended by ASCE (2000).

The seismic response of a soil deposit depends mainly upon the natural frequency of soil stratum, the peak acceleration of input motion, and the frequency contents of the dynamic loading, among others. Normally, the soil deposit will respond predominantly in the fundamental mode of vibration in each direction. Because different acceleration time histories possess different dynamic characteristics (frequency contents, peak magnitudes, etc), a soil deposit can resonate differently with respect to different time histories. In practice, more than one set of acceleration time histories should be used to ensure that the analysis results achieve a given confidence level (ASCE, 2000).



**Figure 2: Horizontal acceleration time history (UK Hard Site HD51)**



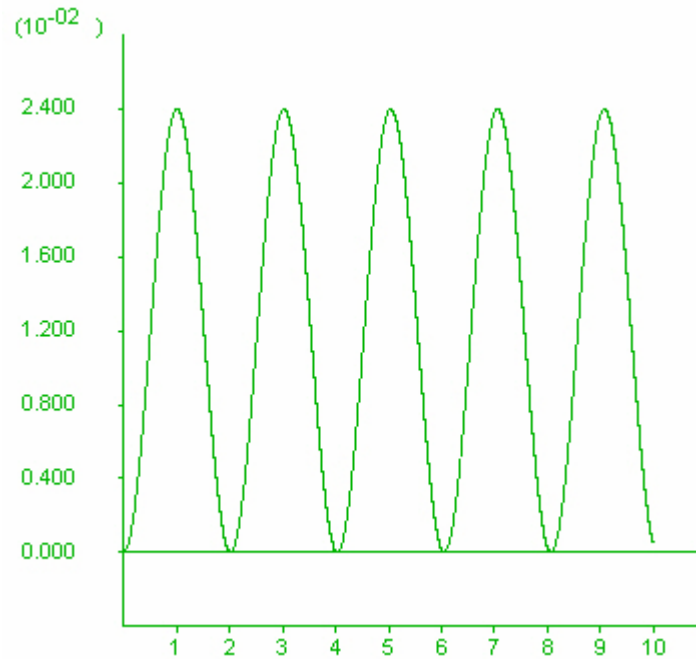
**Figure 3: Vertical acceleration time history (UK Hard Site HD37)**

### Modelling of crane

In the current analysis crane dynamic properties specified by the crane manufacturer (Kobayashi, 2004) are:

	Natural frequency	Participating mass
Horizontal direction	0.492 Hz	466 kg/m run
Vertical direction	1.677 Hz	933 kg/m run

As shown in Figure 1, dynamic effect of the crane has been taken into account by a simple 2DOF stick model, which should have approximately the same dominant frequencies and participating mass specified by the crane manufacturer. For example, horizontal natural frequency of the simple model can be obtained by applying gravity in the horizontal direction and observing the displacement oscillations with zero damping (Figure 4).



**Figure 4: Horizontal response of the crane (natural frequency=0.5Hz cf 0.49Hz)**  
Horizontal axis – time (second), vertical axis – horizontal crane tip displacement (m)

### ANALYSIS PARAMETERS

The height and embedded depth of a reinforced concrete diaphragm quay wall are 23 and 15m, respectively. The wall cross section area is 2.0m<sup>2</sup>/m run and moment of inertia is 4.3m<sup>4</sup>/m run. The distance between the quay wall and the anchor wall is 54m. The length of the anchor wall is 14m. The area of tie rod cross section is 0.002m<sup>2</sup>/m run. The pre-stress in the tie rod is 850 kN/m run.

The example focuses mainly on the seismic behaviour of the quay wall system. It is assumed that the seismic-induced excess pore water pressures will not dissipate completely during the design earthquake. As a result, un-drained soil properties are considered. The soil profile and geotechnical parameters are given in Table 1.

**Table 1. Geotechnical parameters**

Soil profile (thickness)	Mass density Kg/m <sup>3</sup>	Shear modulus (MPa)	Bulk modulus (MPa)	$\phi$ (°)	c (kPa)
Upper Sand (6m)	2200	90	2000	38	-
Upper Clay (5.7m)	1750	100	5000	-	36
Lower Sand (13m)	2200	100	2000	40	-
Rock (~30m)	2800	12000	18000	-	-

### ANALYSIS RESULTS

The model was analyzed under static condition and subsequently followed by a time domain seismic analysis. In the post-earthquake conditions the viscous boundaries and free-field conditions were removed but the seismic-induced displacements at the truncated boundaries were maintained. Post-

earthquake conditions were analyzed under the dynamic mode for a reasonable duration. Typical analysis results are given in the sequence.

### Response Spectrum at cope Level

Figure 5 presents response spectra of the input motion and the corresponding accelerations, respectively, at the top of the quay wall and at the cope level, 15m behind the quay wall. Obviously, cope level seismic responses have been amplified by the local soil stratum and retained soils. Frequency shifts can also be observed. The maximum amplification effect seems to occur at the top of the quay wall. This may be caused by the combined quay wall-retaining soil interaction.

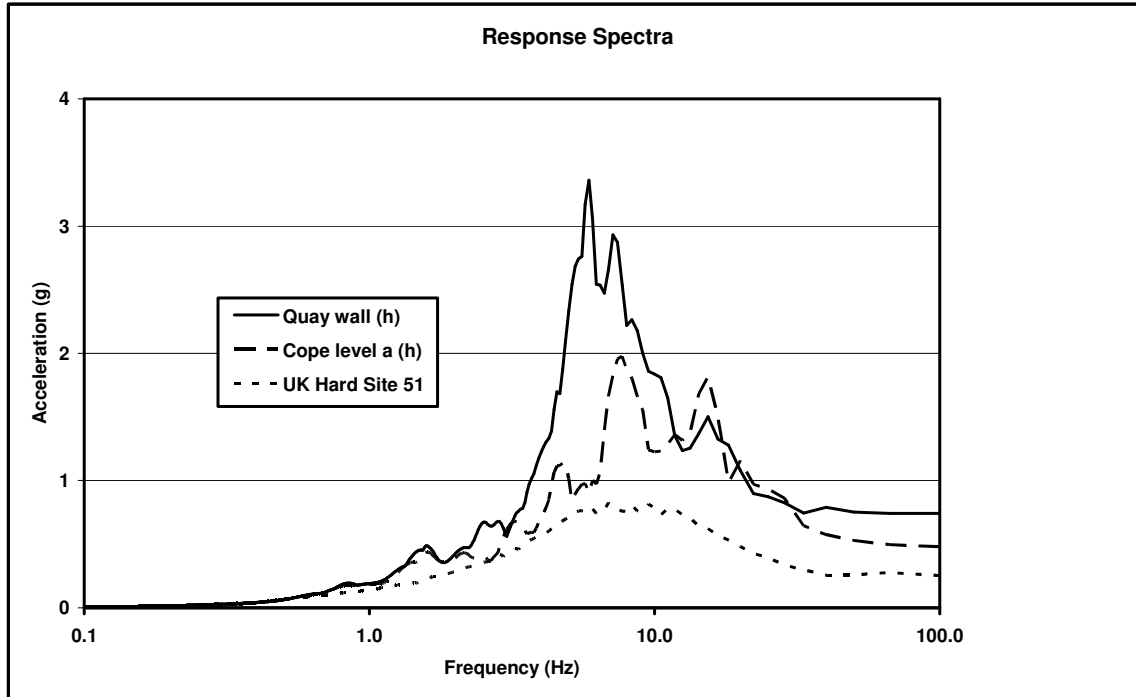
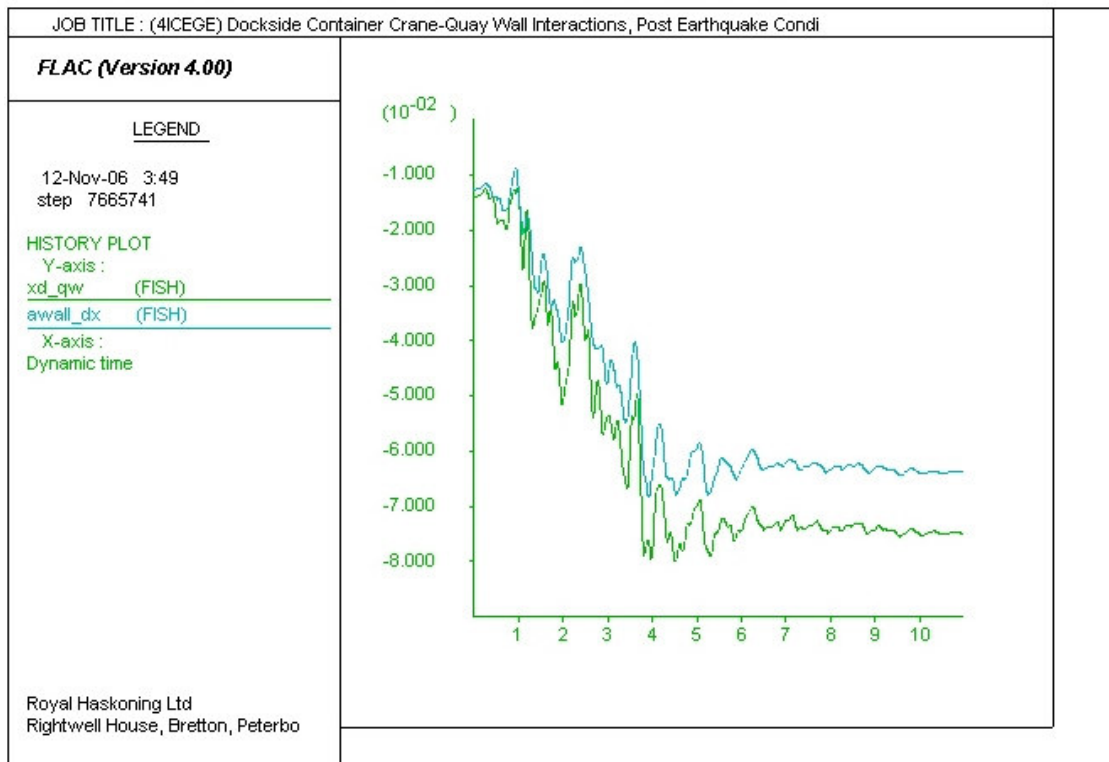


Figure 5: Response Spectra of Input Motion and Cope Level Locations

### Seismic-induced wall displacements

Horizontal movements of tops of the quay wall and the anchor wall are given in Figure 6. It is interesting to notice that their dynamic responses are approximately in phase.

These horizontal displacements are significantly influenced by the seismic input motions, in which strong motion occurring between approximately one and five seconds. The general tendency is that, during the design earthquake, the quay wall and anchor wall are moving toward the sea. However, under post-earthquake conditions these structures reach stable conditions.



**Figure 6: Horizontal displacements of quay wall and anchor wall. horizontal axis – time (second), vertical axis - displacement (m)**

### Crane Response

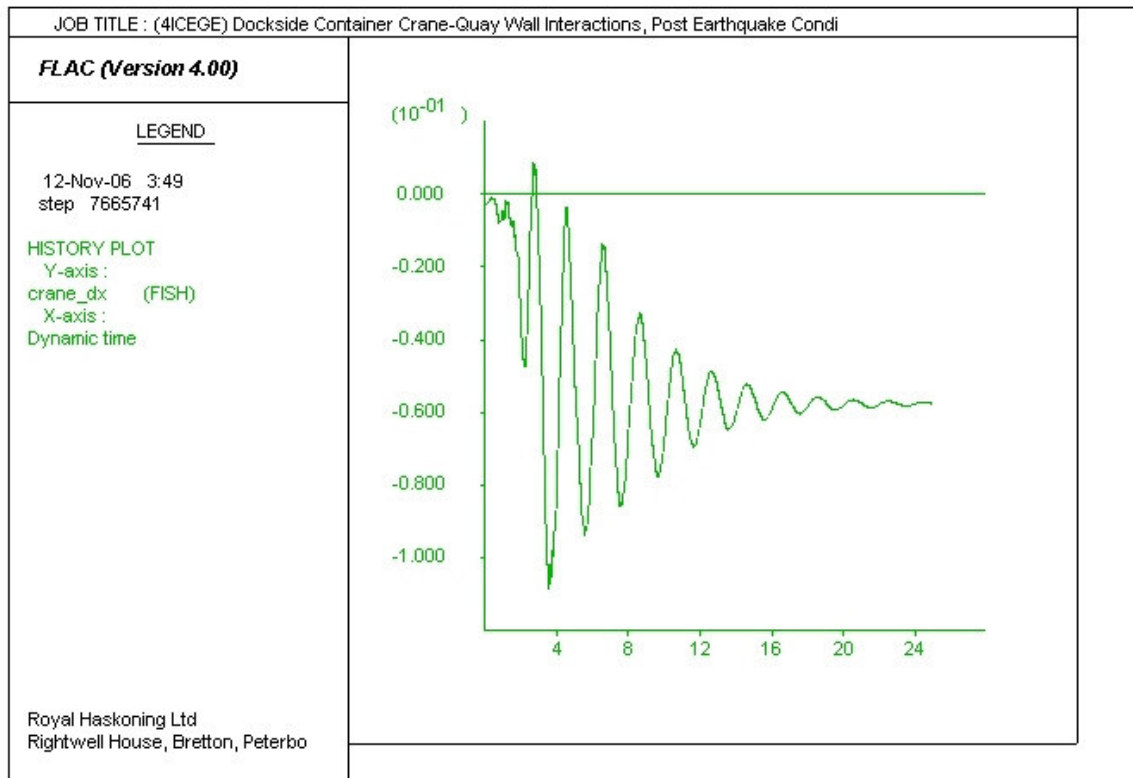
The response of a structure to a dynamic loading generally contains two distinct vibration components: the forced vibration giving an oscillation at the exciting frequency and the transient component giving an oscillation at the natural frequency of the structure.

For the given ground motions, the predicted horizontal and vertical responses of the crane tip are given in Figures 7 and 8, respectively. Noticeable crane responses have been triggered in the early stage of the seismic event. It is interesting to notice that the crane seems to oscillate freely during the seismic event and, because the damping present in the structure, the responses decay with the decrease of seismic loadings and time. The times required for the crane to complete a cycle of vibration are very close to the natural frequencies of the system. This interesting result, valid for typical ground motions containing a wide range of frequencies, has been proven using random vibration theory (Chopra, 1995).

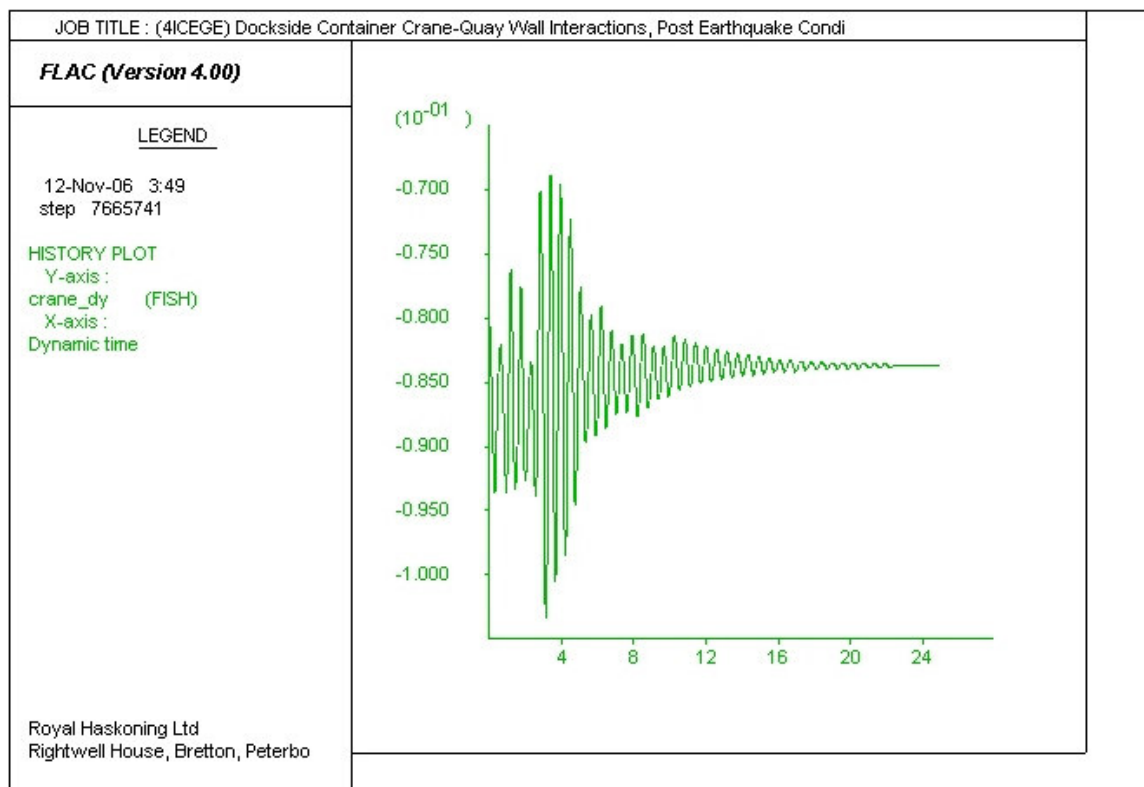
Shear reactions at crane supports with respect to time are given in Figure 9. As expected, the shear reactions at two different crane supports are out of phase. Figure 10 illustrates the relative movements between crane rails. The relative horizontal displacement remained stable approximately after 6 second while the relative vertical displacement was relatively insignificant. It should be noted that the stiffness of the crane supported at two points may restrict movement of the quay wall system. This possibility should be reviewed in practice, and decoupling of the primary structure (the quay wall system) and the secondary structure (the crane) should be avoided when it will result in significant errors.

Once the crane response has been evaluated by seismic analysis of the quay wall system, the detail analysis and design of the crane can be carried out by the crane manufacturer by using, for example, methods based on the concept of the equivalent static force.

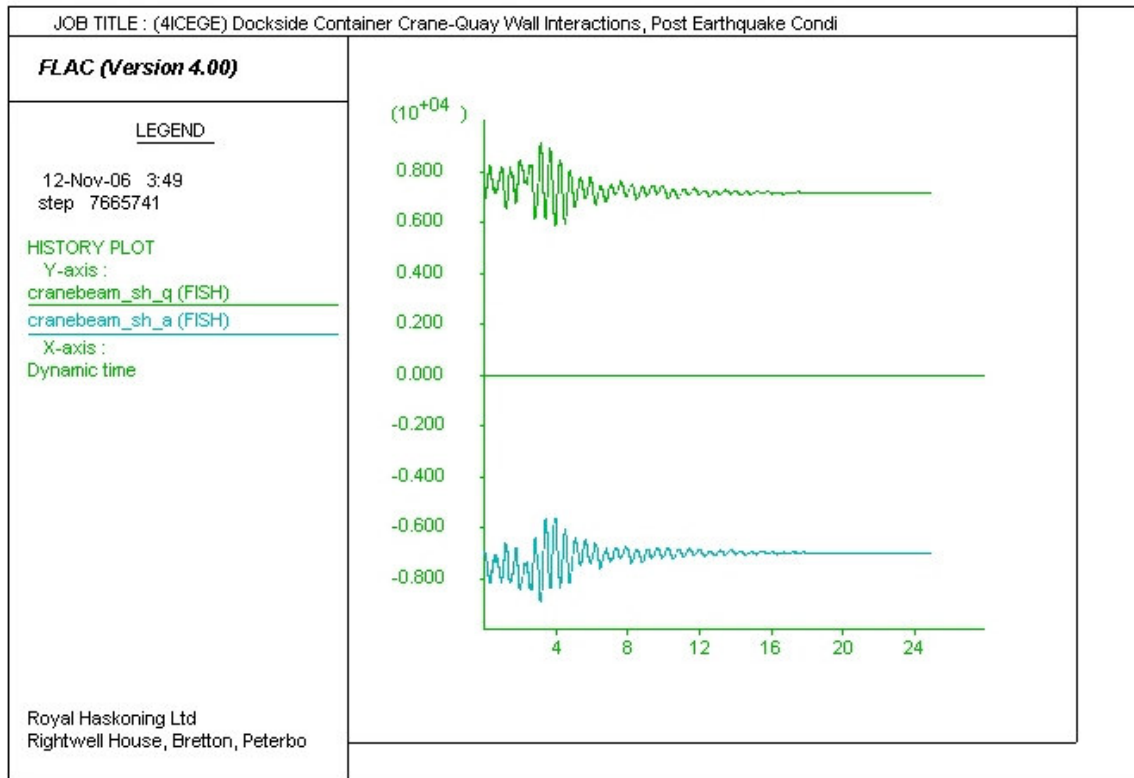




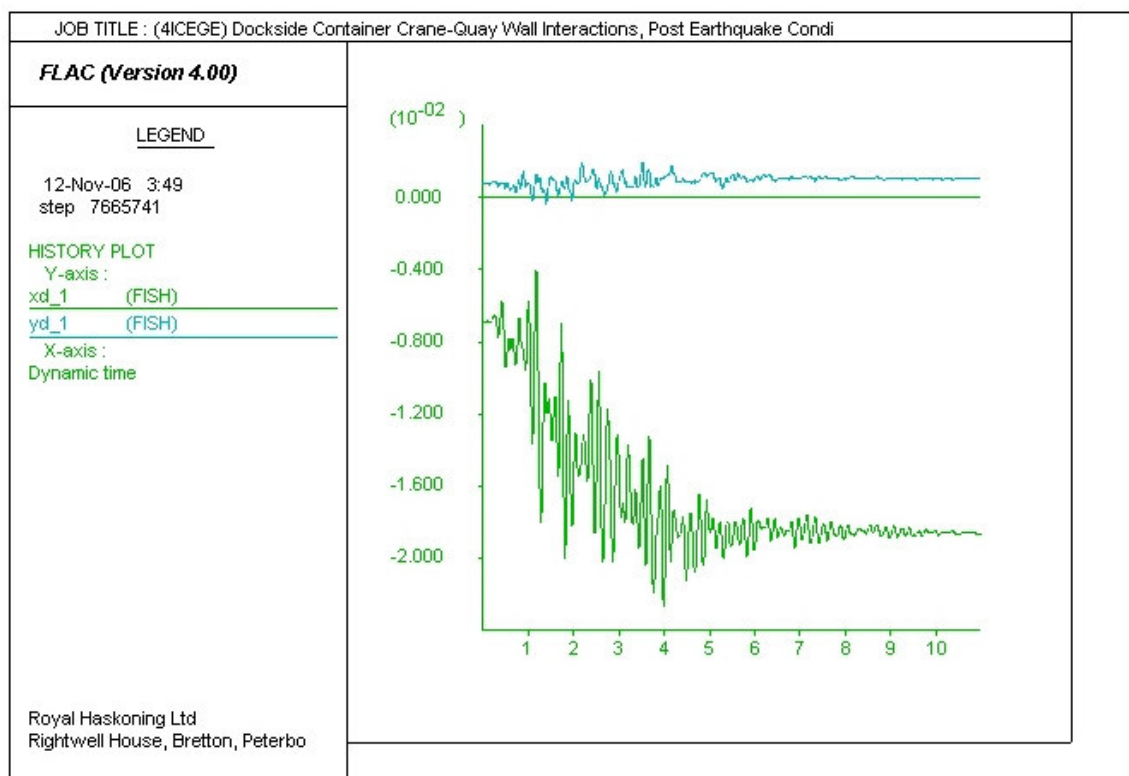
**Figure 7: Horizontal displacement crane response. horizontal axis – time (second), vertical axis - displacement (m)**



**Figure 8: Vertical displacement crane response. horizontal axis – time (second), vertical axis - displacement (m)**



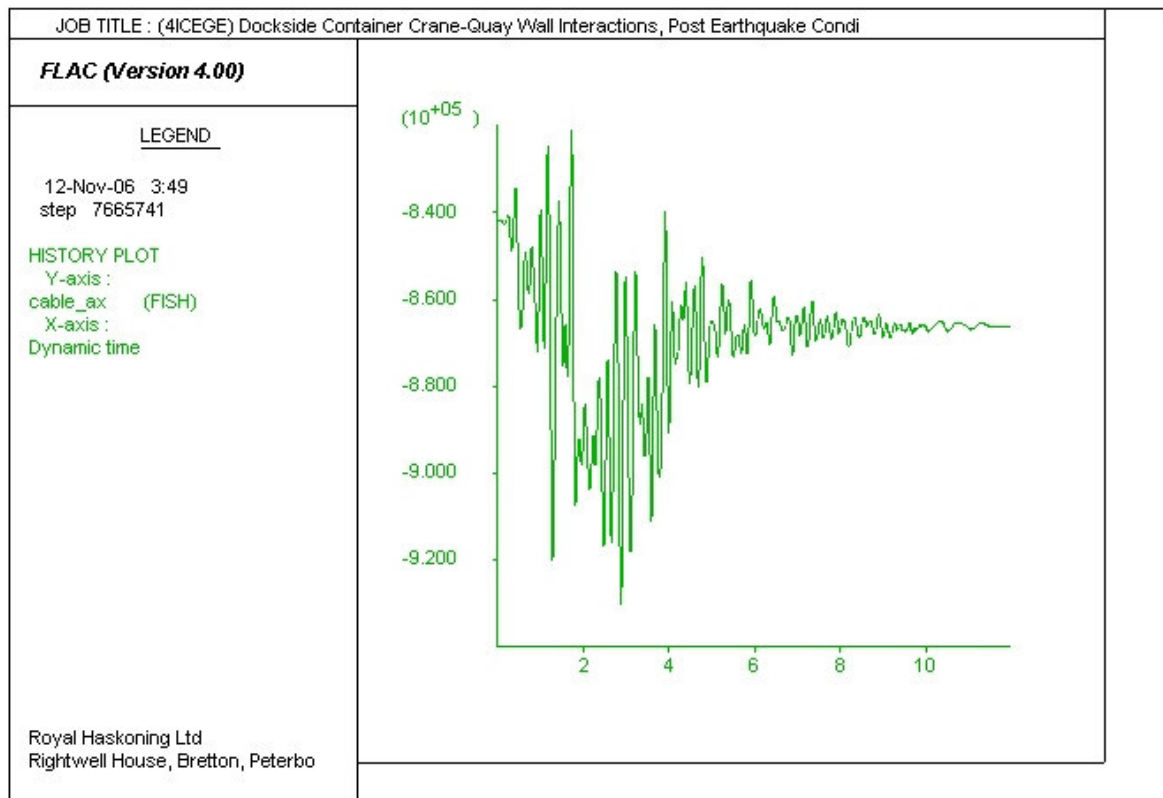
**Figure 9: Crane shear reactions horizontal axis – time (second), vertical axis - displacement (m)**



**Figure 10: Relative displacement between crane rails, Horizontal axis – time (second), vertical axis – displacement (m)**

### Tie rod Response

Because the tie rod is an elastic component, the variation in the distance between quay wall and anchor wall will affect the seismic-induced tie rod force. As shown in figure 6, considerable horizontal differential movements between the quay wall and anchor wall are developed in the first four seconds. The corresponding increase of tie rod force reveals a reasonable interaction between the quay wall, tie rod, and anchor wall. However, the increase of tie rod force will encourage the anchor wall and quay wall to move together and, as a result, reduce the tie rod force. The tie rod force is approximately stable after 6 seconds and, because the damping present in the system, variation in the tie rod force decays with time.



**Figure 11: Tie rod axial force. horizontal axis – time (second), vertical axis - force (N/m run)**

### CONCLUSIONS

Container traffic continues to grow worldwide and ports and shipping lines are ordering larger than post-Panamax ships. Large feeder ports will load and unload the entire ship's cargo at one berthing and the operational sophistication of the terminals that serve the largest ships will become particularly critical. Container operations are undertaken 24 hours per day while vessels are on the berth. This requires a design that involves low maintenance for the quay structure and particularly the crane rails on which the quayside cranes operate. In the event of an earthquake the crane rails have limited flexibility to be easily re-aligned provided the permanent movement in the quay structure is limited. Further more, the front line of a terminal is its container handling quay cranes, which require rigorous structural design and stringent serviceability.

The paper present important issues of time domain seismic analyses of port structures. The dockside crane is simplified as a 2DOF simple stick model, which possesses approximately the same dominant frequencies and participating masses specified by the crane manufacturer. It is shown that noticeable

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