13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1415

PROVIDING VALUE TO CLIENTS THROUGH NON-LINEAR DYNAMIC SOIL STRUCTURE INTERACTION

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SUMMARY

Traditional methods of soil-structure interaction, based on linear soil springs derived independently from structural response, can result in over-conservative, inefficient designs. It is the purpose of this paper to demonstrate by way of case histories, the value in terms of cost savings and increased confidence that can be gained through the use of a holistic dynamic soil-structure interaction performance-based approach. All the case histories presented have been analysed using the explicit time domain finite difference code Oasys LS-DYNA. Over the course of several years Arup has developed the program to provide complete seismic analyses that includes a non-linear soil model, a seismic beam element that models plastic hinging, and a shear wall element. Such analyses allow the performance of both the soil and the structure to be accurately modelled. The first example will present a commercial development in Taipei. By including the basement in the analysis, which had the effect of stiffening up the soil, the analyses showed a reduction in force levels of approximately 30%. The second example shows how the foundation system of a liquefied natural gas (LNG) tank was designed for soil that was potentially susceptible to liquefaction. Due to the nature of the founding soils standard ground improvement techniques were inappropriate. This highly non-linear problem was addressed by allowing for liquefaction and the stiffening effect of flexible steel tubular piles. The third example shows how by directly modelling the sliding behaviour of an offshore structure, the amount of ballast and the force levels in the topsides could be reduced to provide an efficient solution for the client. The final example discusses a LNG jetty in a region of moderate seismicity. Following the design of the jetty, some potential for the coastal slope to deform and load the jetty was recognised. The analyses that were performed indicated that the initial jetty design was adequate except for allowable relative displacements at the joints.

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INTRODUCTION

Dynamic soil-structure interaction (DSSI), sometimes referred to as seismic soil-structure interaction, occurs when a structure and the soil on which it is founded interact during a dynamic event such as an earthquake.

Simple elastic analysis methods (e.g. Velestos & Wei [1]; Roesset [2] etc) were developed more than thirty years ago to examine the dynamic behaviour of simple foundation systems subject to dynamic loads. Design engineers still use these methods today as they provide a relatively simple representation of the soil-structure system. Unfortunately, these methods can lead to either overly conservative designs or structures with poor seismic performance in some cases, particularly under high levels of excitation.

A number of computer programs were developed to examine earthquake DSSI problems during the 1970's (e.g. FLUSH, SASSI, CLASSI etc) to address the design nuclear power facilities in the US. Although these programs being available for some time, only recently have more advanced computer programs started to cross over into mainstream civil and structural engineering. This has occurred as computer power has increased and the costs associated with these types of analyses have been reduced.

Programs such as Oasys LS-DYNA, which can accurately model both the soil and structural non-linearity, now allow the design engineer to better understand how a soil-structure system behaves in an earthquake. Lubkowski *et al* [3] showed that for an ethylene tank under high seismic loading, an advanced analyses provided a better understanding of the performance of the soil-tank system than traditional methods. This lead to a potential reduction in the design loads for the tank of up to 30%. This paper presents four additional case histories in which either similar savings have been achieved or issues with the potential to significantly affect projects have been overcome.

DEEP BASEMENT CASE HISTORY

The primary objective of the study was to assess the effect of the extensive basement and foundation system on the seismic motions that would be experienced by the superstructure during the design level earthquake. A non-linear time history analysis was performed that included a representation of the basement and foundation system in addition to the soil. A 3-D model was used to permit interactions with the basement to be more accurately represented. The main analyses were performed on a "half model".

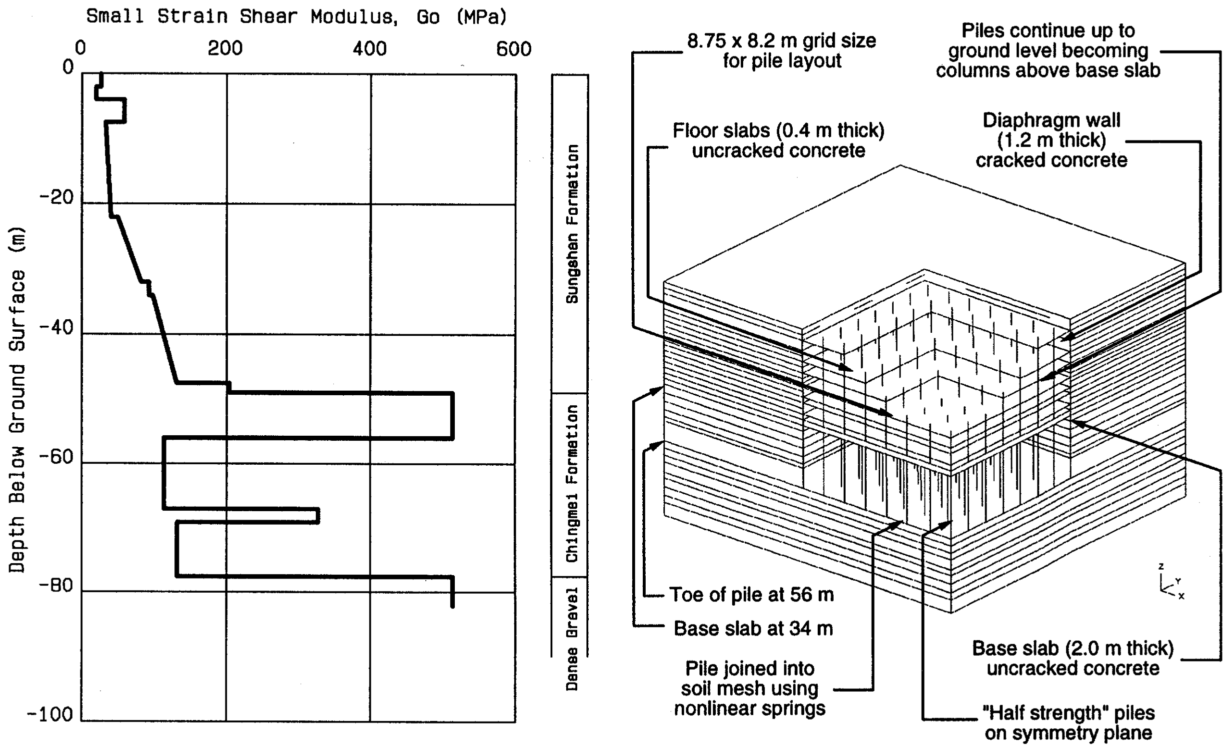
The Analysis Model

Figure 1(a) shows the small strain shear modulus profile for the site. The soil conditions consist of alluvial deposits, separated into three formations: the Hsingchung formation is immediately above bedrock and is overlain by the Chingmei Formation, which consists primarily of dense gravels, and the uppermost Sungshan Formation, which consists of interbedded loose to medium dense sands and soft to firm clays.

The 32m deep basement sits entirely within the Sungshan Formation as can be seen in Figure 1(b), which shows a representation of the finite element model. The key features of the model were as follows:

- The basement was modelled in 3-D using shell elements;
- The soil layers were modelled to a distance of three times the basement length from the centre of the model using a mesh of quadrilateral brick elements;
- The piles were modelled as vertical beam elements, with the ability to form plastic hinges at a proscribed moment. The piles were connected to the soil mesh by non-linear "local deformation" springs;

- The "bedrock" level and the vertical boundaries of the model were subjected to appropriate free-field seismic input motions, and non-reflecting boundaries were imposed.



(a) Small strain shear modulus profile

(b) Finite element model

Figure 1: Soil data and DSSI model

The Analyses

The input bedrock spectrum used in this study was based on a bedrock spectrum, anchored to a peak ground acceleration of 0.3g. This represented the 475 year return period. A series of analyses with different assumptions and modelling details were performed to explore the sensitivity of the key results to variations in basement design and to account for uncertainties in modelling assumptions. The principal variations examined the following elements:

- The effect of a diaphragm wall below the basement level (to -56.0m);
- The effect of the inplane stiffness of the diaphragm wall (i.e. whether there is any effective shear coupling between adjacent panels);
- The effect of modelling active and passive soil failure at the front and back faces of the basement;
- The differences in results for different input time histories.

Conclusions

The DSSI analyses performed demonstrated that the presence of the basement significantly affects the motion that will excite the superstructure. Figure 2 shows four response spectra. These are:

- Bedrock (site specific) which was used as input to the DSSI analyses
- Soil surface free-field based on a non-linear site response analysis
- Code based soft soil
- The top of basement output from the DSSI analyses

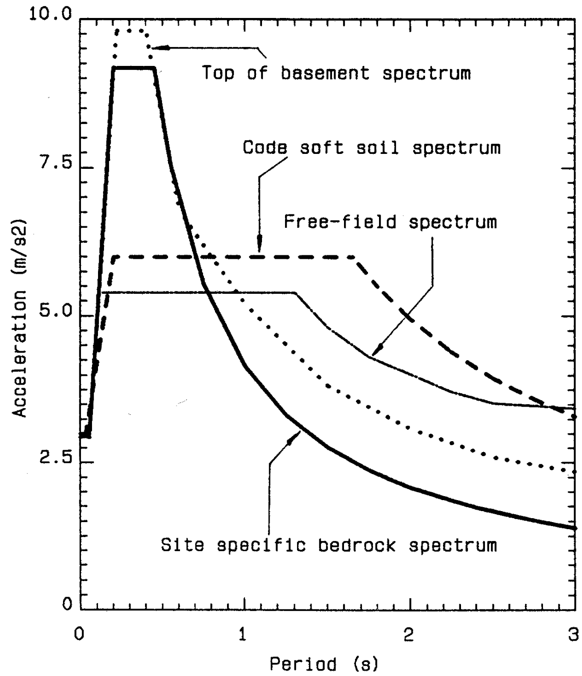


Figure 2: Comparison of enveloped response spectra

Relative to the free-field surface, it can be seen that the basement is subject to lower motions for periods greater than about one second, and amplified motions at shorter periods. The basement motions are similar or greater than the original bedrock for periods less than 0.6 seconds. These effects are of significance for the design of the proposed superstructure because it has a natural period in the region of 1.5 seconds. At this period the basement spectra are approximately midway between the bedrock and surface spectra.

The key conclusions were:

- The basement significantly affects the response of the structure.
- Directly taking account of the basement can lead to a reduction in design seismic load of the superstructure of about 40%.

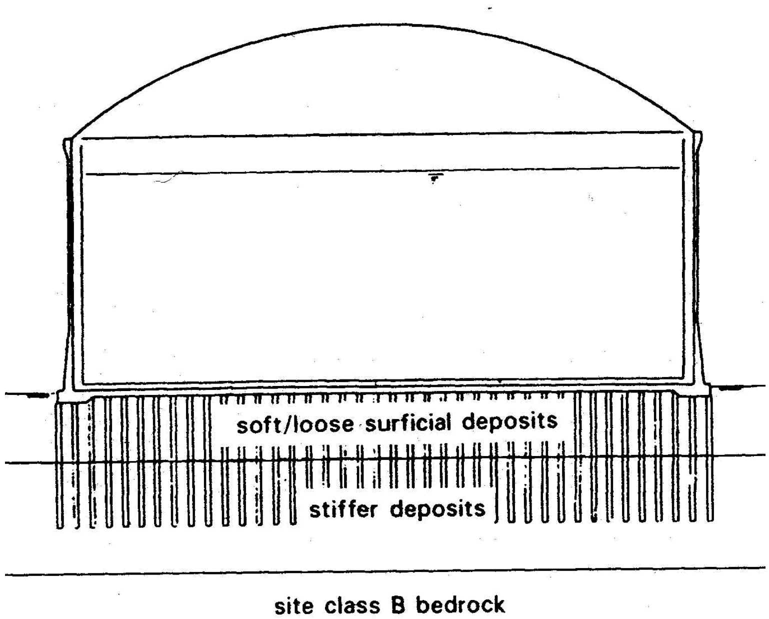
LNG TANK CASE HISTORY

This case history relates to the detailed design of two 105,000m^3 double containment LNG tanks. The site is located on the coast in a moderate to high seismic region. The tanks were designed to a safe-shutdown earthquake (SSE) return period of 10,000 years, which resulted in a peak bedrock acceleration of 0.55g.

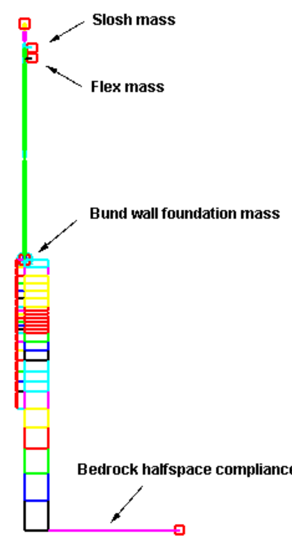
Design Issues

The site was composed of fill and marine deposits over more competent cohesive deposits. Figure 3(a) gives a representation of the tank and soil. A principal issue in the design of the tanks was the potential liquefaction of saturated cohesionless soils under the tank. The fill was a mixture of soft to firm clays and loose to medium dense silts and sands. The marine deposits generally consisted of loose to medium dense silty sands to sandy silts. On site trials to improve the ground using stone columns were found to be unsuccessful due to the high fines content and layered nature of the material. More details can be found in Cassidy [4].

Due to schedule constraints it was impractical to use an alternative method (e.g., soil-cement mixing) to reduce the potential for liquefaction. An alternative solution was proposed in which the foundation system was designed to carry the gravity loads with appropriate margins of safety, and the design was then checked for lateral performance in the design earthquakes, including performance in liquefied soil. The foundation solution consisted of about 1200, 0.6m diameter steel tubular piles. Clay bunds were also to be installed along the boundary of the tank platform to prevent lateral spreading.



(a) Sketch of LNG tank



(b) LS-DYNA Model

Figure 3: LNG Tank

The Analysis Model

Due to the geometry of the tank and small depth of the soft soils it was determined that a 1-D model would be sufficient to represent the soil-pile-tank interaction. The model was used to determine the force demands in the piles and provide peak accelerations for the tank design. The key aspects of the model were:

- Non-linear soil elements with stiffness properties formulated using the Iwan [5] multi linear type model. The model incorporates hysteretic damping in accordance with the Masing principles.
- The piles were lumped together as a single line of vertical non-linear beam elements. A fixed pile to slab connection was assumed.
- The interaction between the soil and piles was modelled by using transverse kinematic springs with appropriate stiffness and limiting strength properties. More details of can be found in Lubkowski [3].
- The inner tank and product was modelled as a two mass-spring-damper system using the standard mechanical analogues based on the New Zealand Guidelines [6]. The height of the masses was set such that the overall moment (pressure on the base plus shell moment) is simulated. The outer tank and roof were represented as a single mode mass/spring/damper system.

To account for variability in the dynamic non-linear soil properties both lower and upper bound conditions were analysed. This allowed for better understanding of uncertainty in the analyses. Liquefied soils properties were derived using a lower bound estimate of residual strength following the methodology presented by Seed and Harder [7]. Liquefaction was either assumed or not allowed during the entire analysis as indicated in Table 1.

Table 1 – Input Parameters for Stochastic Simulations

Soil Properties	OBE	SSE
Lower Bound	No Liquefaction	Liquefaction
Upper Bound	No Liquefaction	Liquefaction

Conclusions

Figure 4 shows a comparison of bedrock and surface response spectra in a free-field site response analysis in which liquefaction was assumed. As can be seen there is considerable attenuation of ground motion for periods less than about 2.5 seconds, which lead to a significant reduction of forces within the tank. Similar results were shown for all time histories used. These analyses indicated that lower bound liquefied conditions lead to larger loads in the foundations, whilst the non-liquefied upper bound conditions lead to larger loads in the tanks.

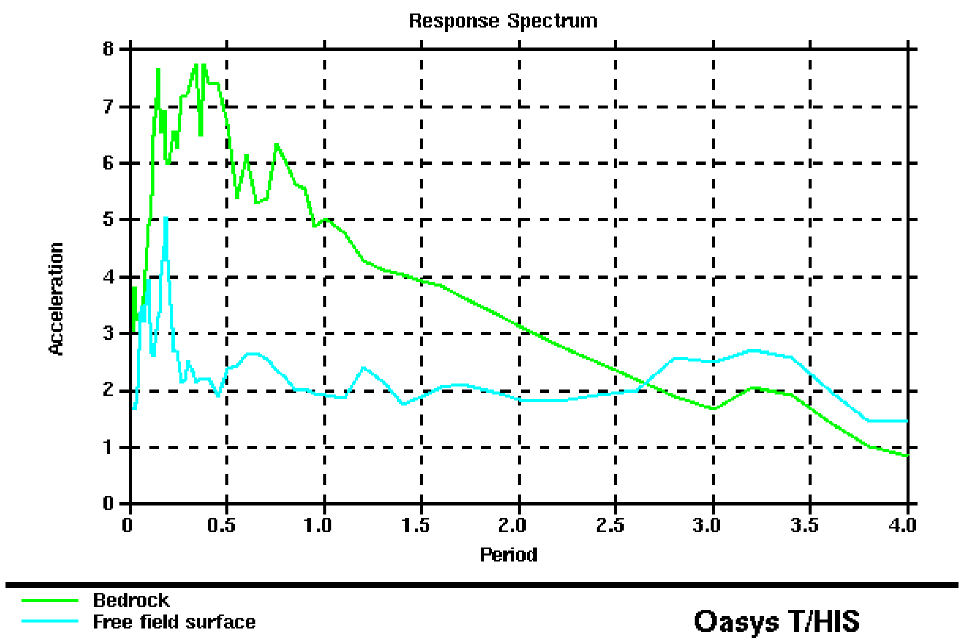


Figure 4: Comparison of bedrock and free-field response spectra

At the completion of the design process the performance of the tanks under both liquefied and non-liquefied conditions were captured and the proposed foundation design approved. The key conclusions were:

- The gravity designed pile arrangement performed satisfactorily in the design seismic events.
- Even with liquefaction of the loose silty sand deposits the bending moments in the piles did not significantly exceed their first yield moment.
- Expensive ground remediation was not required.
- Tank design forces were reduced by about 25% by incorporation of period elongation and high material damping in the soil.

OFFSHORE PLATFORM CASE HISTORY

A Concrete Gravity Substructure (CGS) was to be designed to support process facilities and provide temporary condensate storage in 43m of water. Figure 5 shows a 3-D model of a typical CGS. The structure was to be sited in a region of moderate seismicity, with a PGA of 0.38g for the ductility level earthquake (DLE). The soil conditions at the site comprised a relatively thin layer of medium dense to dense carbonate sand over limestone bedrock. In total the CGS contains some 66,000 tonnes of concrete, 7,100 tonnes of reinforcing steel and 600 tonnes of pre-stressing steel strands.

Key Issues

CGS structures maintain their stability under wave, ice and seismic loading by ensuring they have sufficient on-bottom weight to provide friction that will exceed the imposed lateral forces. Traditionally the key to their performance is a balance between the base area, the mass and the soil resistance. When considering seismic loading however, increasing the mass of the structure also increases the force that must be resisted. During the early phase of design these coupled effects led to a structure of an uneconomic size.

As a result, the design concept was changed to allow the structure to slide at the soil-caisson interface, within acceptable limits. Those limits were determined based on the connection details between the CGS and associated pipelines. A series of highly non-linear 3-D soil-structure interaction analyses were carried out using the LS-DYNA program to determine the performance of the structure.

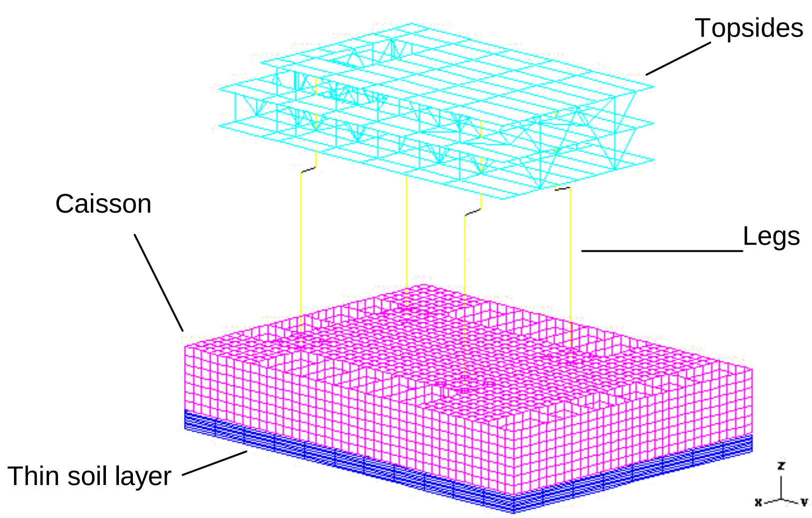


Figure 5: Typical finite element model of CGS

The models focused on the following issues:

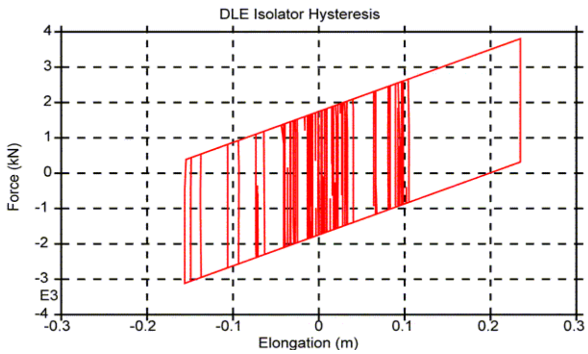
- Non-linearity in the thin carbonate sand layer.
- Non-linearity in the soil to caisson interface.
- Non-linearity in the leg to topsides connection, including possible seismic isolation bearings.

As with the previous case studies, the effect of variability and uncertainity in soil stiffness was examined by assuming upper bound and lower bound soil properties. Figure 6 presents typical hysteresis loops for the soil and a possible seismic isolator at the deck connection. As can be seen in Figure 6(b) there is failure in the soil, resulting in sliding along the soil-caisson interface.

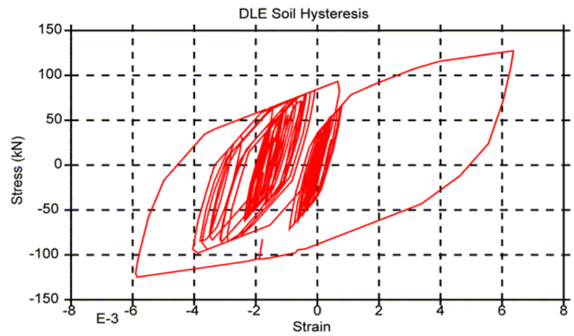
Conclusions

The analyses undertaken were able to provide confidence in the performance of the CGS under both SLE and DLE events. The key conclusions were:

- The loads were reduced by about 20% in the CGS and topsides, due to hysteretic damping in the soil and the deck connection.
- A reduced quantity of ballast was required.
- Under SLE and DLE events acceptable displacements were calculated.
- The checking authority accepted the design.



(a) Deck Connection Hysteresis



(b) Soil Hysteresis

Figure 6: Hysteretic Damping in the Soil and Deck Connections

JETTY CASE HISTORY

A jetty for an LNG plant had been designed without due consideration of the stability of the adjacent sea slope and its interaction with the jetty under seismic loading. In order to prevent delays to the project schedule associated with a revised design, a series of non-linear finite element analyses were commissioned to:

- Determine the extent of the deformation of the jetty slope.
- Determine the bending moments in the piles and check whether they are within the estimated design capacities.
- Review the design of the bearing gap at the connections.
- Determine the interaction of the slope movements following the earthquake and the deformation of the jetty structures.

The site is in a region of moderate seismicity with a design PGA of about 0.2g and the soil conditions consist of medium dense silty sand overlying soft marine clay.

Analysis Details

Because the study examined the effects that slope displacements caused by earthquake and gravity loading would have on the piled jetty structures, a 2D model was created representing a "slice" through the pipe trestle and the loading platform. This slice represents the mass contribution from the jetty over one row of piles perpendicular to the slope. A symmetric half model of this slice was created for the analysis, as shown in Figure 7.

The key features of this model were as follows:

- The soil and the jetty slope were modelled using quadrilateral brick elements with non-linear material properties. The variation of soft clay strength across the site due to dredging is accounted for by zoning the elements.
- The piles and the deck structures were modelled as “seismic” beam elements, which can form plastic hinges when the bending strength of the section is exceeded.
- The piles are connected to the soil mesh using non-linear horizontal and vertical springs.
- The connection between the beam elements representing the jetty deck and the piles allow moments to be transferred
- The pipe trestle beam element properties represent the composite action of the pre-stressed beams and the roadway slabs
- The mass of the piles exposed above the slope and below the water level accounted for the presence of water inside the hollow piles and the added mass due to hydrodynamic loading. The mass of the piles below the slope surface considered the presence of soil inside the hollow section
- The presence of water and the associated water pressures along the surface of the slope was considered in the model
- The “bedrock” level and the far-field vertical boundaries of the model were subjected to the free field seismic input motions and were attached to non-reflecting boundaries to model the semi-infinite space

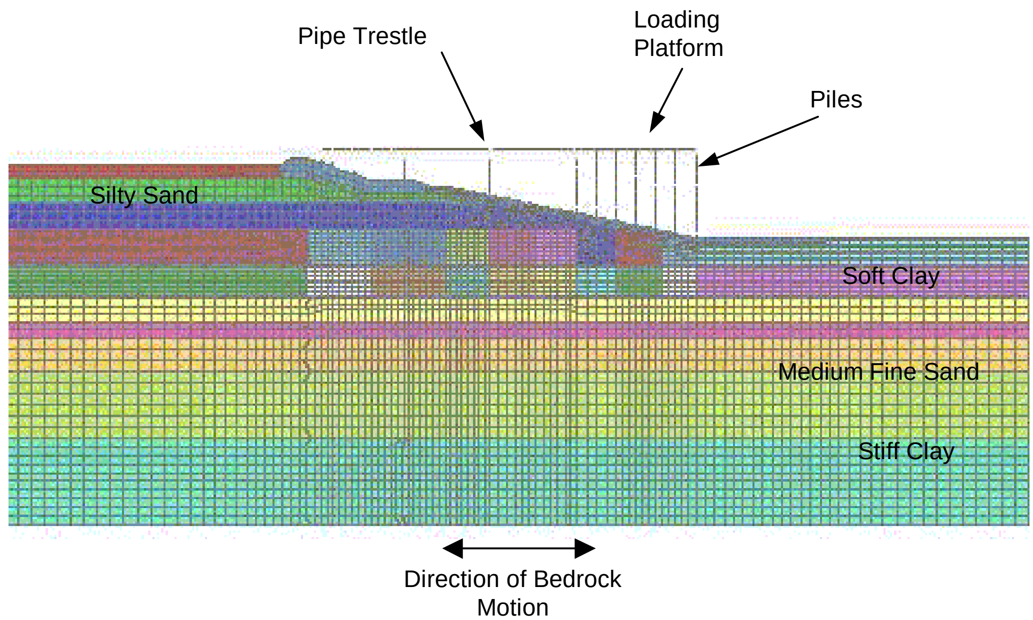


Figure 7: 2-D Model of Jetty Structure

Conclusions

The following conclusions were drawn from this study.

- The most onerous analysis showed a residual horizontal downstream slope movement of 100mm. In this model, the crest settled by approximately 50mm and the toe heaved by 30mm.
- The maximum bending moment in the piles was less than the capacities previously estimated.
- The maximum displacement measured in the jetty was approximately 230mm, which occurred when the lower bound soil properties were assumed.

- The analyses indicated that a maximum relative displacement between the loading platform and the pipe trestle of about 130mm, as is shown in Figure 8.

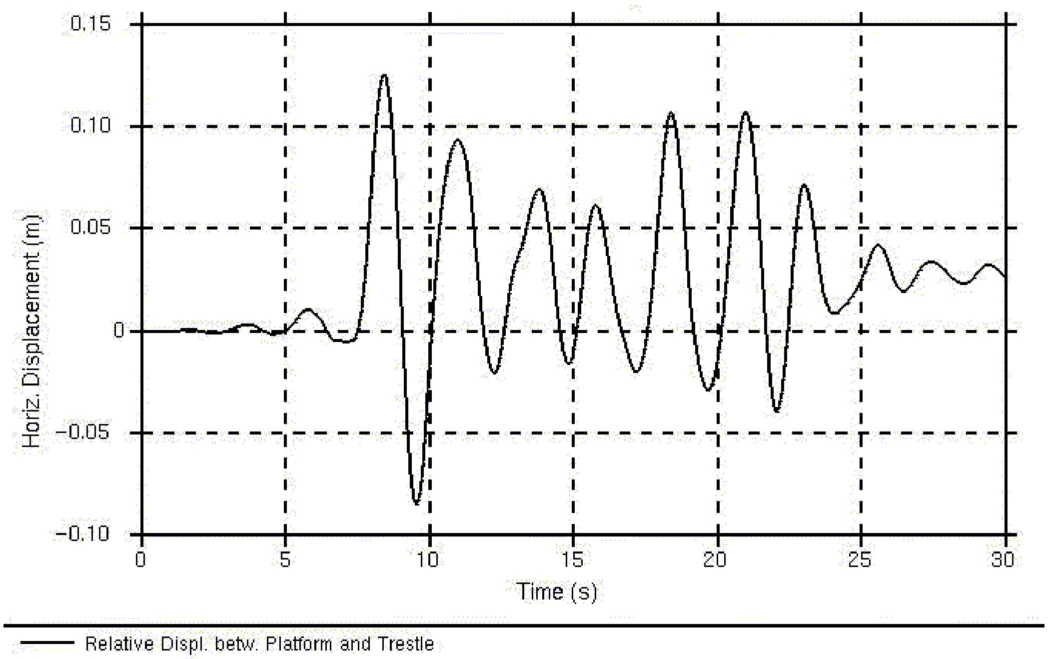


Figure 8: Typical Relative Displacement Time History

- The maximum relative displacement between the pipe trestle and the top of the slope was predicted to be about 160mm.
- It was estimated from the analyses that following the design earthquake, the jetty will have a small amount of residual displacement of the order of 115mm.

The analyses performed indicated that the existing design would perform in an acceptable manner if a small increase in the allowable movement of the connections within the structure was made.

CONCLUSIONS

The case histories presented show that non-linear DSSI analyses can provide considerable benefit to projects, compared to more traditional analysis methods, either by ensuring their viability at the conception of a project or by assessing their performance once they have been designed. Such analyses can ensure cost-effective design solutions and quantifiable levels of performance.

Furthermore, with the advances in computer analysis capabilities, such analyses are becoming more common place in the design office. Where once the analyses described in this paper may have taken several days, they can now take just a few hours. This makes such complicated non-linear analyses of an entire soil-foundation-superstructure system a viable design option.

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