THE INFLUENCE OF DYNAMIC SOIL-STRUCTURE INTERACTION ON THE SEISMIC DESIGN AND PERFORMANCE OF AN ETHYLENE TANK

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SUMMARY

This paper presents the analyses and results for the dynamic soil-structure interaction assessment of an ethylene tank, in the Philippines. The initial design had been carried out by two independent organisations, one designing the pre-stressed concrete piles the other the steel tank structure. The purpose of the study was to determine the effects of kinematic interaction on the piled foundations and/or potential cost savings for future projects, by considering the entire soil-structure system in a single analysis.

The non-linear finite element code *Oasys* LS-DYNA was used to model the interaction between the soil, piles and tank structure. The program models the soil as non-linear elements with hysteretic properties. Structural members are modelled with elements that can form plastic hinges. Two levels of earthquake were considered, the Operating Basis Earthquake with a return period of 500 years and the Safe Shutdown Earthquake with a return period of 5000 years.

The assessment showed that the piles would behave in a plastic manner under the service level earthquake. The analyses also showed that the tank would remain elastic in the ultimate level earthquake, which suggests that significant cost savings could have been made in the original design of the steel tank.

INTRODUCTION

Figure 1 shows a cross section of the steel double wall ethylene tank. The outer tank diameter is 45.8m and the inner tank is 42.8m diameter. The aluminium inner deck is suspended by hangers from the outer tank roof. The tank is supported by an elevated pile cap, 1.3m above ground level, resting on 525, 0.5m square prestressed concrete piles. The soil comprises of about 20m of fill and loose to medium dense clayey silt with a significant sand and gravel content, overlying a very dense boulder layer.

The purpose of this study was to carry out a combined tank and foundation analysis to check the suitability of the tank and foundation design loadings.

METHODOLOGY

The method adopted for the dynamic soil-structure interaction (DSSI) analyses was to perform a non-linear time history analysis using the finite element program *Oasys* LS-DYNA. A 3-D model, simplified to a quarter model with the two axes of symmetry was developed. The model represents a uniform horizontally layered soil profile, the pile group, the pile cap and the tanks (modelled as mass-spring analogous system).

The analysis model is illustrated in Figure 2. The DSSI analyses have been carried out using our *Oasys* LS-DYNA non-linear dynamic finite element program. Previous uses of *Oasys* LS-DYNA for dynamic soil-structure interaction analyses are described in Willford *et al.*,1996 and Lubkowski and Pappin (1993).

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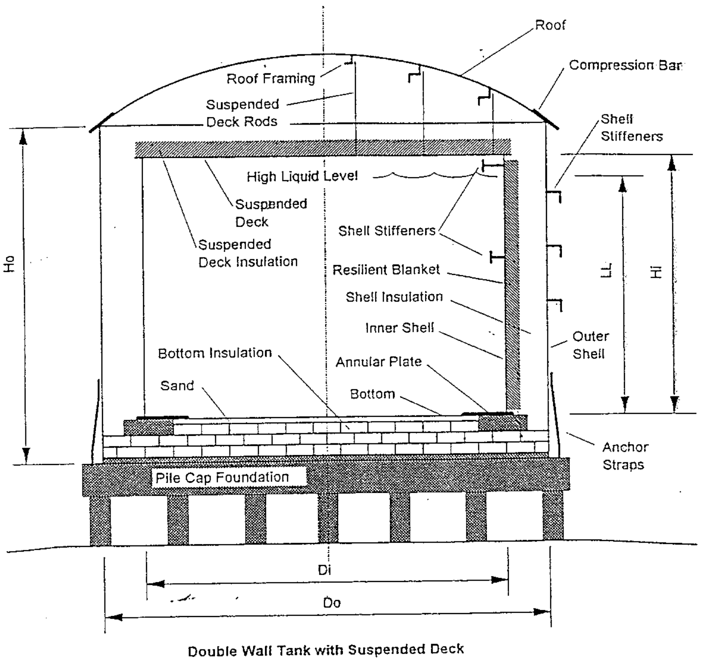


Figure 1: Typical Cross-Section of Ethylene Tank

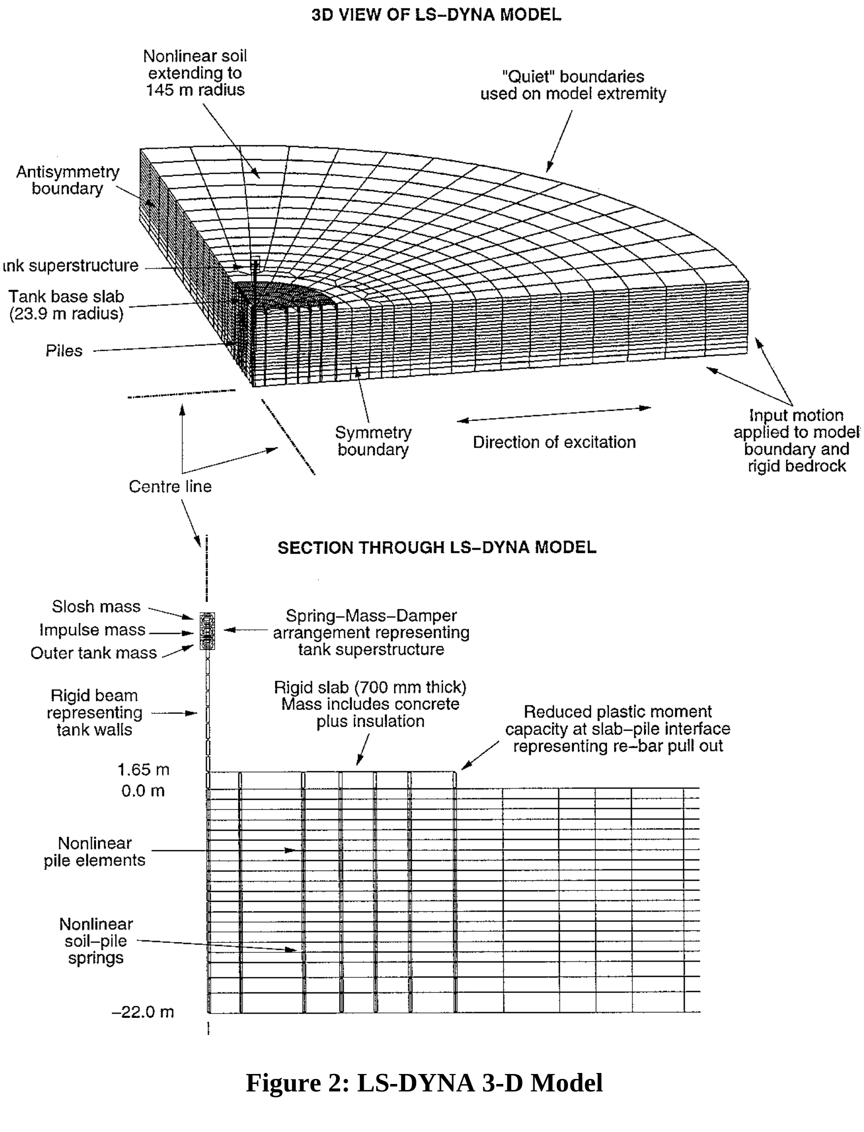


Figure 2: LS-DYNA 3-D Model

The analysis is designed to simulate the transmission of seismic motions from the assumed bedrock level at -7.0m RL (22m below ground level) to the pile cap as a vertically propagating shear wave. The model is subjected to time histories at assumed bedrock level, and the (non-linear) responses of the soil and structural elements are computed. Maximum values of the key load effects during the time history are extracted for comparison with the values adopted in the design.

By modelling the soil down to bedrock and applying the ground motion at bedrock level the analysis effectively combines a site response analysis (including the effect of piles on site response) and a DSSI analysis with the tank. Because both DSSI and site response are non-linear it is more realistic to perform the two parts together than seperately.

SOIL PARAMETERS

All the available information was reviewed in order to develop a suitable soil stratigraphy specific to the tank area.The natural ground formation comprises weathered pyroclastic deposits, consisting predominantly of clayey silt with a significant sand and gravel content, overlying an Andesitic boulder layer. The natural topography of the site has been levelled by 'cut and fill', where the fill material comprises the same pyroclastic unit as the natural ground. The fill has generally been well compacted.

The best estimate soil parameters used for the analyses are presented in Table 1. Upper and lower bound soil parameters were actually used for the analyses, assuming ±50% on the best estimate low strain shear modulus values which were derived using standard empirical relationships (e.g. Imai and Tonouchi, 1982).

Table 1: Assumed Soil Properties

Stratum	Depth (m)	Bulk Unit Weight (kN/m^3)	SPT 'N' value (blows per 300mm)	Total Stress	Effective Stress		PI (%)	OCR
				S_U (kPa)	ϕ' (deg)	c' (kPa)		
Fill	0.0 -7.0	17.5	8	65	25	10	13	1.6
Pyroclastics (Clayey Silt)	7.0 - 17.0	17.5	6	50	25	10	13	1.0
	17.0 - 22.0	17.5	16	130	25	10	13	1.4
Boulders	22.0 - depth	22.0	>50	>200	30	50	-	-

INPUT TIME HISTORIES

Analyses were performed for OBE and SSE ground motions of which the bedrock uniform hazard response spectra (UHRS). Three input earthquake motions were selected such that the shape of their response spectra match the design UHRS. The selected earthquake motions are described in Table 2 below.

Table 2: Description of Unscaled Time Histories

Earthquake	Date (D-M-Y)	Time Step (sec)	Length of Record (sec)	PGA (m/s^2)
El Centro, California	18/05/40	0.02	53.73	3.42
Taft, California	21/07/52	0.02	54.38	1.76
San Fernando Valley, California	9/02/71	0.02	42.98	1.77

The time histories were scaled to the OBE and SSE levels, which had peak ground accelerations (PGA) of 0.45 and 0.75g respectively. Horizontal motions only were applied to the model.

A one-dimensional site response analysis was carried out to provide input motions for the DSSI analyses. This was done using the non-linear time domain program, *Oasys* SIREN (Henderson et al, 1989). The SIREN program allows for reflected energy to be partially absorbed by the bedrock, hence the motion at the top of the bedrock is modified from the input motion. This modified motion was used in the DSSI analyses.

Non-linear site response analyses were performed for each of the three scaled records, and the most onerous was selected for input to the DSSI analysis model. This was Time History 2, generated from the Taft (1952) record.

FINITE ELEMENT MODEL

The sections below identify the data and assumptions used in the analyses.

Geotechnical Modelling

The soil was represented explicitly using the LS-DYNA non-linear seismic soil model, which is described by Lubkowski (1996). The non-linear stiffness is formulated using the multi linear type model (Iwan, 1967) and the model exhibits hysteretic damping in accordance with the Masing principles (Pyke, 1979). No viscous damping is included in this model. The best estimate dynamic stiffness parameters for the soil model were specified as a value of G_0 (small strain shear modulus) and the variation of G/G_0 with strain amplitude, based on the data in Table 1.

Piles

The piles were represented explicitly in the model since they will stiffen the soft soil layers beneath the structure. The piles (0.5m square prestressed concrete piles) were modelled as vertical beams. In the model, the nodes of the soil element mesh were further apart than the pile spacing to enhance computational efficiency. Therefore several piles were ‘grouped’ together at each nodal position. For the purpose of these analysis the piles were initially modelled assuming elastic stiffness parameters and uncracked section properties. Checks could then be made to see whether the moment capacities had been exceeded. The pile to slab connection proposed by the designers would result in ‘pinned’ behaviour, and this was modelled.

Soil to Pile Springs

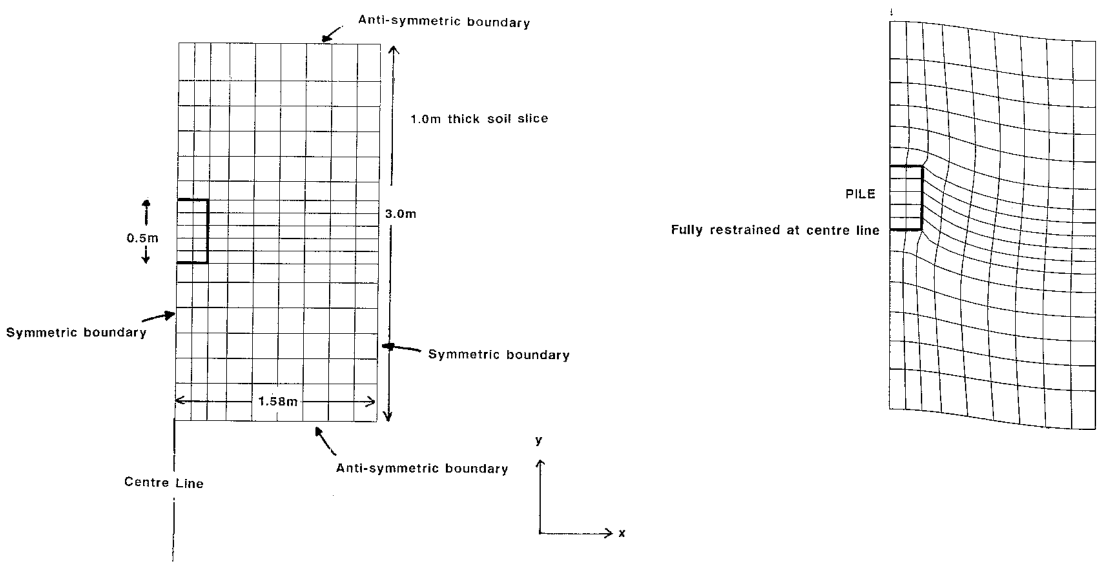
The interaction between the soil and piles was modelled in LS-DYNA by using transverse and vertical kinematic springs. The stiffness of the horizontal springs was determined by a simple elastic plane stress finite element analysis. A typical repeating horizontal slice of soil surrounding a single pile was analysed to determine the elastic deformation of the soil when resisting a lateral load from the pile as shown in Figure 3. The spring stiffness at each soil node was then determined from the ratio of the actual dynamic soil stiffness (computed from the free-field site response analysis) to the soil stiffness used in the linear finite element model.

The passive limiting strength (Fp) of the transverse springs was based on the passive resistance coefficients established by Brinch Hansen (1961) where:

$$F_p = [K_q \sigma_v' + c K_c] h D \tag{1}$$

where K_q and K_c are the passive resistance coefficients for frictional and cohesive soils, σ_v' is the effective vertical stress, c is the cohesion, h is the length of pile corresponding to one spring and D is the pile diameter.

The vertical stiffness at each node is a function of the ultimate shaft friction of the pile and the deflection required before this shaft friction is fully mobilised. The deflection was calculated according to API RP2A (1993), and the skin friction was calculated assuming an undrained cohesive material with the strength properties presented in Table 1.



Finite Element Model Deflected Shape

Figure 3: Calculation of Soil-Pile Spring Stiffness

Structural Modelling

The base slab is modelled for the purposes of this analysis as a rigid shell having density to simulate the mass of the 700mm concrete slab plus insulation and half the tank shell masses.

The inner tank and product is modelled as a two mass-spring-damper system using the standard mechanical analogues based on the New Zealand Guidelines (1986). The height of the masses is set such that the overall moment (pressure on the base plus shell moment) is simulated. The outer tank and roof have been represented as a single mode mass/spring/damper system. These mass/spring/damper systems are located at the centre line of the tank (at the corner of the quadrant modelled). The damping levels are given in Table 3 below.

Table 3: Damping Levels

Mode	Damping (%)	
	OBE	SSE
Welded Steel Structures	2	4
Prestressed Concrete Structures	2	5
Impulsive Liquid	3	7
Sloshing Liquid	0.5	0.5

DISCUSSION OF RESULTS

Eight analyses have been undertaken, which examine the effect of different assumptions for soil stiffness and design motion criteria. Analyses were carried out for both upper and lower bound soil conditions for OBE and SSE motions. Figure 4 shows a typical bending moment envelope and the instantaneous bending moment diagram at 5 seconds for an actual pile. The maximum values from these analyses are presented in Table 4. below.

Table 4: DSSI Analysis Results

System Component	Original Design Values. *The design values marked thus *, include an R_W factor of 2.*	DSSI Predictions			
		Lower Bound Soil		Upper Bound Soil	
		OBE	SSE	OBE	SSE
Inner Tank	0.56g*	0.55g	0.65g	0.63g	0.86g
Outer Tank	0.41g*	0.57g	0.75g	0.74g	0.83g
Slab	0.56g*	0.42g	0.64g	0.60g	0.76g
Foundation Loads	Pile Capacities**	OBE	SSE	OBE	SSE
Shear at Top of Pile	306kN	80kN	105kN	110kN	130kN
Max Bending Moment (*elastic*/ultimate)	*305*/480kNm	*375kNm*	600kNm	*270kNm*	400kNm
*(N.B. Shaded values have exceeded either the tank or pile design parameters) (** Pile capacities assumed static axial load from tank applied. Values will be lowered under reduced axial load arising from overturning moments)*					

The results indicate that under SSE loading and assuming lower bound soil conditions the ultimate bending moment capacity of the pile (modelled as elastic uncracked) is exceeded. The maximum bending moment in this case is found about 17m below ground level, at the change in soil from clayey silt to sandy silt. Furthermore, for lower bound soil conditions and OBE loading the elastic capacity is exceeded. For upper bound soil neither the elastic or ultimate capacities are exceeded for OBE and SSE loading respectively. Table 4 also indicates that the original tank design loads are exceeded in almost every case.

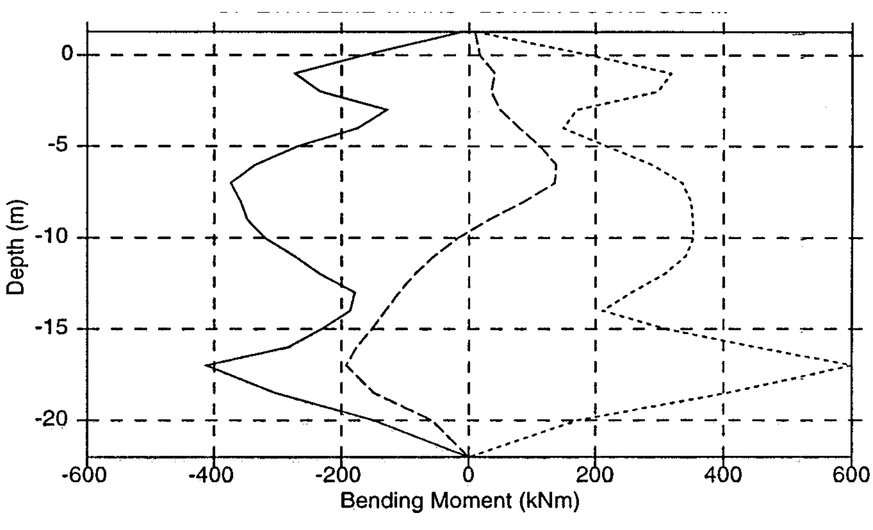


Figure 4: Pile Bending Moments, Lower Bound Soil, SSE Input

To further examine the behaviour of the pile, for lower bound soil conditions under SSE loading, a series of revised analyses were carried out to quantify the effect of different modelling assumptions for the pile elements. The results are presented in Table 5 below.

Table 5: Effect of Modelling Assumptions

Modelling Assumption	Max Bending Moment (kNm)	Plastic Rotation (radians)
Ultimate Capacity of 480kNm with uncracked EI	>480	0.0025
Ultimate Capacity of 480kNm with cracked EI	390	0
Elastic Capacity of 305kNm with uncracked EI Ultimate Capacity of 480kNm with cracked EI	400	0
N.B. The piles were modelled so they could not exceed their ultimate capacity quoted.		

These analyses indicate that there may be some plastic rotation of the pile at 17m below ground level. Since the pile shear reinforcement is the minimum non-seismic required, it is therefore possible that some damage to the piles (crushing and loss of confinement) may occur under the SSE event, assuming lower bound soil conditions.

The tank design loads are, with the exception of the OBE load cases for the inner tank and slab, exceeded by the estimates from the analyses. However, application of appropriate force reduction factors would bring the results below the design values.

The results shown in Table 4 indicate that the tank loads calculated from the DSSI analyses are less than the original design values if appropriate reduction factors (2 for OBE and 3 for SSE after ASCE, 1997) are applied. Table 6 gives a comparison of design loads for both the OBE and SSE criteria assuming the appropriate reduction factors. As can be seen the most onerous loads calculated for the inner tank and slab are about 60% of the original design values.

Table 6: Effect of Reduction Factors on DSSI Results

System Component	Original Design Values	DSSI Prediction			
		Lower Bound Soil		Upper Bound Soil	
		OBE	SSE	OBE	SSE
Inner Tank	0.56g	0.28g	0.22g	0.32g	0.29g
Outer Tank	0.41g	0.29g	0.25g	0.37g	0.28g
Slab	0.56g	0.32g	0.21g	0.30g	0.25g

CONCLUSIONS

The static design technique originally used for the design of the tank and the foundation ignored the potential benefit from including dynamic soil-structure interaction effects as presented in this paper.

A series of dynamic soil-structure interaction analyses have been carried out by modelling the entire soil-pile-tank system using the non-linear, time domain, finite element code *Oasys* LS-DYNA. A range of soil properties has been assumed in the analyses to take account of the uncertainty in soil parameters. The lower bound properties impose the largest loads on the foundations, whilst the upper bound soil properties impose the largest loads on the tanks.

The piles do not exceed the moment capacities under the loads induced by the OBE criteria. However, the piles may just exceed their moment capacities at about 17m below ground level assuming lower bound soil conditions at the SSE criteria.

The shear force at the top of the piles is well below the capacity provided, for all soil conditions and loading criteria.

The tank design parameters originally assumed are generally all exceeded for the OBE and SSE criteria design. However, application of force reduction factors would reduce them to well below the design values.

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