Assessment of the effect of wave-in-deck loads on a typical jack-up

Prepared by **MSL Engineering Ltd**
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FOREWORD

This document represents a generic study undertaken by MSL Engineering Limited for the Health and Safety Executive to determine the role of air gap, and possible inundation of the hull, of a typical jack-up subjected to a 10,000 year wave.

The reserve strength of the jack-up was investigated for two different water depths with the hull at four different elevations for both depths. The lowest elevation corresponds to the minimum allowed (ie. 50 year wave + tide + surge + 1.5m) and the maximum elevation was such that the hull just cleared the crest of the 10,000 year wave. A new model to assess the wave forces, including buoyancy, acting on the hull was developed. Structural failure of the leg, pinion failure, leg lift-off and exceedance of foundation design capacity (sliding and preload) were the various criteria examined to define the reserve strength. Other parameters, such as wave theory and foundation fixity, were also varied for the pushover analyses.

CONTENTS

1. SUMMARY

A wave-in-deck load model has been developed ("The MSL method") to determine the important loads caused by wave inundation on the hull of a jack-up structure. The MSL method is a refinement of a method proposed by Shell for fixed structures to take into account the large buoyancy loads caused by a wave passing round a solid hull structure. Wave-in-deck loads have been calculated using the MSL method and the Statoil method, to allow a comparison of the wave-in-deck loads predicted.

Sixteen pushover analyses were performed on a detailed non-linear model of a typical jack-up structure. The pushover analyses were based on the 10,000-year wave, including wave-in-deck loads as appropriate. Different values of hull elevation, extreme wave height, foundation fixity and wave-in-deck model were used to assess their importance to system capacity. In addition, a site assessment was performed, based on SNAME 5-5A guidance, to determine what increase in still water level may be achieved by reducing the environmental load factor from 1.25 to 1.15. The effect of the revised still water level on the system capacity was also investigated.

Extreme Load Ratios (based on the 10,000 year wave) were found to be greater than unity where inundation did not occur. As the level of the hull was lowered to the 50 year crest height + 1.5m air gap, ELR's dropped to values less than unity. The critical failure mechanism for hull inundations of more than 1m was leg lift-off. The inclusion of wave-in-deck buoyancy loads was important in predicting this failure mechanism.

2. INTRODUCTION

The minimum elevation of a jack-up hull structure above the still water level is generally determined by adding 1.5m to the 50 year or 100 year extreme crest elevation [(1)]. The air gap of 1.5m is intended to give a margin of safety against a more extreme wave event hitting the hull structure, and in turn give the structure an acceptably low annual probability of failure. However, it has been suggested [(2)] that the return period against the air gap being exceeded is dependent on the location of the structure, for example, being much less than 10,000 years for the Northern North Sea. This variability in the return period may have a large effect on the reliability of a structure.

Structural robustness, i.e. the ability of the structure to withstand an extremely rare event such as a 10,000-year wave, is normally assessed by performing a pushover analysis. Usually, the 50 or 100-year wave load is applied to the structure, and the peak base shear or overturning moment is determined. This load is then applied to the structure and incrementally factored until structural failure occurs. The load factor at failure is between 2 and 4 for most structures, and is known as the Reserve Strength Ratio (RSR). The RSR may then be seen as an indication of a “factor of safety” on the design event. However, by factoring up the 50 / 100 year wave the wave inundation that may occur in the deck structure for extremely rare events will not be taken into account, and hence the RSR is likely to be unconservative.

The inclusion of wave-in-deck loads leads to a step change in environmental loading once wave inundation occurs. A better and more accurate understanding of the reliability of the structure is given if a pushover analysis is performed using the 10,000-year wave, including wave-in-deck loads (if any). Generally, to give a failure rate of less than 10^{-4} then the ELR (Extreme Load Ratio, ie. the load factor based on the 10,000-year wave) should be greater than unity. The importance of including wave-in-deck loads when considering structural reliability was demonstrated by Tromans [(3)] where the annual probability of failure of the Gulf of Mexico Jacket SP62-B increased from 1 in 1600 years (wave inundation not considered) to 1 in 370 years (wave-in-deck loads taken into account).

The calculation of wave-in-deck loads is generally not well defined nor well understood. In addition, research has tended to concentrate on loading on porous deck structures of jackets rather than the solid hulls of jack-ups. Wave-in-deck models may be divided into two categories:

i. Global / Silhouette Approach: A porosity or blockage factor is determined for the deck structure as a whole. A wave pressure is applied to the blockage area, generally based on a $\rho v^2/2$ term. A global wave loading coefficient is normally applied to increase the loading. Models in this category include the API, SHELL and Statoil methods. Their main advantage is ease of computation and application.

ii. Detailed Component Models: Loads on individual components of a deck structure are determined and summed to give a global load. The effects of component shielding and different drag and inertia coefficients for each component may be included. Examples of this type of method include the Kaplan, Chevron and Amoco methods. Although the component method is theoretically capable of producing the most accurate results, loads may be complex and time-consuming to determine.

Both of the above methods rely on empirical coefficients based on accepted values for individual members or on model test data. The treatment of vertical loading is inconsistent, with some models (e.g. Statoil and API methods) not considering them at all. All methods have primarily been developed for fixed structures, which usually have porous decks. For the special case of jack-up structures, where the hull may be considered as a single component, the global/silhouette method is likely to yield the most sensible results, although the potentially large buoyancy loads caused by wave inundation may have a large effect on structural integrity and should be taken into account. A wave-in-deck model, derived from the Shell method but modified to suit a jack-up Hull structure, has been developed and is presented (referred to as the MSL model) in this report

The degree of wave inundation, and hence the magnitude of the wave-in-deck load, is dependent on the hull position relative to the extreme crest elevation for the 10,000 year wave. The extreme crest elevation will be a function of the following random distributions:

- Maximum wave height, Hmax
- Ratio of crest height to wave height (often assumed to be 0.6)
- Tide
- Storm Surge.

In order to determine the true extreme crest elevation, data from a relatively short data acquisition period must be extrapolated to a very long return period. The various methods used to do this give significantly different results [4]. These differences are likely to have a large effect on deck inundation and hence a significant effect on structural reliability. Two different 10,000 year crest elevations have been used in the present study, and are referred to as the SNAME 5-5A method and the HSE Guidance method (the HSE Guidance Notes have been withdrawn):

Item	***SNAME 5-5A***	***HSE Guidance***
Hmax	31.2	29.3*
Crest elevation relative to maximum still water level	18.7	17.6
Combination tide and surge	2.2	2.5
Max. Crest elevation	20.9	20.1

*Hmax taken as 17.6/0.6

The work presented in this report assesses the relative importance of wave-in-deck loads on a typical jack-up structure. A detailed non-linear model of a jack-up structure has been created. Two site assessments have been performed, based on SNAME 5-5A recommendations, to determine the maximum still water levels the structure may be able to operate in with an environmental load factor of 1.15 and 1.25 respectively. Sixteen pushover analyses have been performed for the two water depths based on factoring the 10,000-year wave loads to determine the relative importance of the following on the ELR (Extreme Load Ratio):

- Deck inundation
- Wave-in-deck load calculation method
- Foundation fixity assumptions
- Extreme crest height calculation method
- Environmental load factor used in the Site Assessment.

A flow diagram of the analysis procedure is shown in Figure 1.

3. MODEL DESCRIPTION

3.1 Introduction

A detailed structural model of a typical harsh environment jack-up was created, see Figure 2, using the SACS analysis software marketed by Engineering Dynamics Inc. SACS contains advanced non-linear modelling features such as global and local buckling behaviour, plastic hinging and joint flexibility, that allow non-linear response to be determined.

The modelled structure is a 3 legged jack-up with a maximum elevated weight of 12500t. The legs are triangular, consisting of 3 tubular chords spaced at 12.2m and braced at bays of 5.96m with a K-bracing arrangement. The combined weight of the legs and spudcans (excluding buoyancy) is 1275t. At the bottom of each leg is a spudcan, with a plated tank section immediately above extending to a height of 23m. The overall length of each leg is 146m, and are spaced approximately 55m apart. The connection between the leg and hull consists of a set of pinions that essentially provide vertical support, and rigid horizontal guides at the bottom of the hull and at the top of a yoke frame.

In plan, the hull structure is approximately triangular, with a side length of approximately 72m (see Figure 3). The horizontal wave loading was assumed to extend over a length of 60m.

3.2 Legs

All major structural members in the legs were explicitly modelled. The leg model was sufficiently detailed to ensure that representative loads resulting from wave and current forces would be generated.

The main chord members of a jack-up generally have significantly different section properties about their major and minor axes, caused by the rack teeth and any additional strengthening to the tubular that may be required. For simplicity, the chord members were modelled as equivalent tubulars, whose diameter and wall thickness was chosen to give the average plastic modulus and the cross-sectional area of the combined section. It is these properties which have the most effect on the global structural stiffness and the axial and bending capacity of the legs. The tank plating at the base of each leg was represented by trussed members giving an equivalent stiffness. Brace offsets were not modelled.

The hydrodynamic loads for rack post members were generated assuming the original chord diameter of 850mm and modified values of Cd and Cm based on recommendations in SNAME 5-5A [(5)]. Averaged values of Cd for the three chords in each leg were used for simplicity. The following values of Cd and Cm were used:

Element	***Cd***	***Cm***
Rough rack post	1.66	1.8
Smooth rack post	1.45	2.0
Rough brace member	1.0	1.8
Smooth brace member	0.65	2.0
Tank region at base of leg	8.9*	89.1*

* coefficients applied to individual members of 850mm diameter.

Marine growth of 12.5mm was assumed on all members up to LAT +2m, as described in SNAME 5-5A.

3.3 Leg/Hull Connection

The guides and pinions forming the connection between the legs and hull were represented such that the connection stiffnesses were given by appropriate member properties and end releases. The guides and pinions were attached to the yoke frame, which was modelled to give a representative structural stiffnesses. The following guide and pinion stiffnesses were modelled:

Element		*Stiffness*
Upper Guide		86.0×10^3 t/m
Lower Guide		500×10^3 t/m
Pinions	Vertical	75.0×10^3 t/m
	Horizontal	35.1×10^3 t/m

The gaps that would exist between the guides and the chord members and in the pinion mechanism were not modelled in these analyses.

The pinion members were modelled as linear elements. Pinion slippage (yielding) was assumed to occur along a rack of 6 pinions when the combined vertical load in the pinions exceeded their combined capacity. The ultimate capacity of 6 pinions combined was taken as 73600kN.

3.4 Hull

The hull was modelled using plate elements. The plate thicknesses were chosen to reflect the true stiffness of the bulkhead stiffnesses including stiffeners. Plate thicknesses and stiffener geometries were taken directly from available drawings of the jack-up structure. The yield stress of the plate elements was raised above the true yield stress to ensure the hull behaviour was elastic.

3.5 Foundations

Foundation fixity was represented using linear springs at the bases of the legs, except in the "pinned" cases, where translationally rigid supports in three orthogonal directions were applied. The following three foundation stiffnesses were modelled in the analyses:

Foundation Type	$K_{Rotational}$ *(kNm/rad)*	$K_{Vertical}$ *(kN/m)*	$K_{Horizontal}$ *(kN/m)*	*Comments*
SNAME '97	4.96×10^6	0.13×10^6	75.8×10^3	Used as an 'Extreme' stiffness
Measured	66.0×10^6	1.39×10^6	386×10^3	Stiffness inferred from measured data of a jack-up [(6)]
Pinned	0	∞	∞	

The "measured" fixities were values inferred from measured natural periods of the Maersk Endurer [6]. It can be seen from the above table that the measured fixity is an order of magnitude greater than the stiffnesses determined using the SNAME 5-5A recipe, albeit the former was not for an extreme event.

3.6 Code Checking

Code checking parameters were required for the initial site assessments. The present work is a comparative study, and the objective of the site assessment was to give the same maximum member utilisations for environmental load factors of 1.15 and 1.25. For this reason, it was considered acceptable to use the code checking parameters of API LRFD [7], instead of the (albeit similar) parameters used in SNAME 5-5A, as these were already embodied in the SACS software.

The following code check parameters were used:

Element	***K factor***	
Main Chords	1.0	
Main Diagonals	0.718	Original factor of 0.8 reduced to reflect offsets at member ends
Main horizontal members	0.744	Original factor of 0.8 reduced to reflect offsets at member ends

Member lengths were set to the node to node distances.

4. LOAD GENERATION

4.1 Wave-in-Deck Loads

4.1.1 Summary of existing wave-in-deck models

A survey[7] of open literature revealed a number of wave-in-deck load assessment models. Essentially the approaches fall into two categories, namely:

- A global or silhouette approach, which provides an overall assessment of platform risks from wave-in-deck loading; and
- A detailed component approach, which provides detailed evaluation of component damage or more specific force calculations.

The available global approaches include the method in API RP 2A Section 17 [1], the Shell method[8] and the Statoil method[9]. The available detailed component approaches are the Kaplan model[10], the Chevron model[11] and the Amoco model[12].

API RP 2A Section 17 presents a simple method for predicting the global wave/current forces on platform decks. In the API model the load is evaluated based on calculating the drag force from a wave incident on the projected area of the deck. It only produces front horizontal loads without explicit consideration of inertial, impact or pressure gradient effects. Wave phasing is also neglected.

The Shell model is based on conservation of momentum principles. It assumes that the top of the wave that hits the deck is “shaved off” during its passage. The forward horizontal momentum of water particles in the portion of the wave that hits the front of the deck is completely and instantly nullified, which produces a horizontal impact force on the front of the deck. For a transparent deck, additional horizontal and vertical forces are also generated when the wave passes up through the bottom of the deck. It should be recognized that initial slamming effects from the upward motion are followed a second or two later by downward inertia effects which can be significant due to the large vertical accelerations in the crest. These cannot be captured by the Shell model.

Of all the available models for jackets, the Kaplan model may be the one with the soundest theoretical background. In this model both horizontal and vertical forces are included for individual structural members with different cross-sectional shapes. The horizontal or vertical force in this component-based approach consists of the impact force, the inertia force, the force component resulting from pressure gradient effects, the drag force and the buoyancy force etc.

The Statoil model only considers the wave impact pressure force (or slamming) on the front of the deck. This model may be considered as a simplification of the Kaplan model by omitting secondary terms, or a variant of the Shell model with the slamming coefficient determining the degree of momentum change assumed.

The basis of the Amoco Model is to adopt Morison’s equation and modify the coefficients to give an effective representation of wave-in-deck loading phenomena in terms of drag, inertia and slamming. In common with all other models, buoyancy is neglected in the Amoco work.

The drag and inertia coefficients are modified to capture the high initial impacts through calibration to the more complex Kaplan equations derived from the technical data.

Chevron Petroleum Technology Company has been at the forefront of experimental investigations to evaluate the consequences of a wave encroaching the platform deck. The results formed the basis for the silhouette API model. Chevron recommended that the API model should be used as a general procedure for wave-in-deck load calculations. However for more detailed accurate evaluations, an alternative component based approach is proposed utilizing a numerical model of the deck to calculate the local loads as the wave passes through the structure. The wave forces are computed using Morison's formulation, with specific drag coefficients depending on wave approach direction and equipment density, and an inertia coefficient of 1.5.

Although all the above methods exhibit the same general trend (i.e. increasing wave-in-deck inundation leads to increased loading), there is a large difference in the magnitude and direction of the resultant loads [(7)].

It can be seen from above description that a global model may be more appropriate to be used for the wave-in-deck force evaluation, since the present jack-up has a hull deck structure. Among the three global models, the Shell model may be the most appropriate as it considers the wave phase. However, the Shell model only provides the impact force on the front of the deck hull, and the vertical force, e.g. buoyancy etc., has not been covered. A new model, referred to as MSL model has therefore been developed based on the Shell model.

4.1.2 MSL wave-in-deck force model

The MSL wave-in-deck force model was developed from the Shell Model to suit better the special case of wave loading on a solid hull-type structure. Both horizontal and vertical forces have been considered. The horizontal force includes the initial impact force on the deck hull front. The vertical force includes the buoyancy and the hydrodynamic force induced by the wave particles when it passes under the deck hull.

Following the Shell Model, it is assumed here that the top of the wave that hits the deck hull is "shaved off" during its passage, and the forward horizontal momentum of the water particles in that portion of wave which hits the front of the deck is completely and instantly nullified. The impact force on the front of the deck hull is evaluated using the principle of momentum balance. The predicted impact force is therefore conservative. With this conservative "shaved off" assumption, it is considered that the drag force of the wave after it passes the front of the deck is small and can be neglected.

<u>Impact Force</u>

As shown in Figure 4, using an appropriate wave theory, the following times can be predicted by referring t=0 for wave crest at X=0:

t_1 = the time when the crest starts to hit the front of the deck,
t_3 = the time when the crest starts to leave the front of the deck,
t_4 = the time when the crest starts to hit the back of the deck,
t_6 = the time when the crest starts to leave the back of the deck.

At each instant in time between t_1 and t_6, the wave crest hitting the deck can be considered as a quasi-stationary process. The expression of the momentum balance can be written as:

$$F_h = \frac{dm}{dt} \cdot (u + u_c) \qquad (1)$$

where, F_h is the impact force, $\frac{dm}{dt}$ the rate of mass flow, u the initial horizontal wave velocity of the water particle, and u_c the current velocity. The impact force on the front of the deck hull occurs between t_1 and t_3.

The mass flow rate in equation 1 can be written as:

$$\frac{dm}{dt} = \rho \, . \, b\,(x) \, . \, (\eta - h_d) \, . \, (u + u_c) \qquad (2)$$

where, ρ is the water density, b(x) is the width of the hull at X=x, η is the (undisturbed) wave surface elevation, and h_d is the air gap.

<u>Vertical Hydrodynamic Force</u>

The vertical force acting upwards at the bottom of the deck hull occurs between t_1 and t_6. This force can be written as follows by using the same momentum balance argument as that for the impact force:

$$dF_V = \frac{dm}{dt} \, . \, v, \quad \frac{dm}{dt} = \rho \, . \, b\,(x) \, . \, v \, . \, dx \qquad (3)$$

which gives

$$dF_v = \rho \, . \, b(x) \, . \, v^2 \, . \, dx \qquad (4)$$

In the above equations, v is the vertical velocity of the water particle. Integrating Equation 4 from X=a to X=b gives the vertical hydrodynamic force F_v. The integral bounds are, by ignoring the downward components:

X = a, location of the wave crest if the crest has passed the front of the deck, as shown in Figure 5a, or the location of the front of the deck shown in Figure 5b if the crest has not passed the front of the deck.

X = b, location of the front of the wave if the front of the wave has not passed the back of the deck, as shown in Figure 5c, or the location of the back of the deck shown in Figure 5d if the front of the wave has passed the back of the deck.

<u>Buoyancy</u>

For the present problem, the buoyancy force, which occurs between t_1 and t_6, can be written as:

$$dF_b = \rho \, . \, g \, . \, b(x) \, . \, (\eta - h_d) \, . \, dx \qquad (5)$$

where, g is the gravity . The other parameters have been defined above. Integrating Equation 5 between X= c and X=b will give the buoyancy. Here, X=b is defined in Figure 5. X=c denotes the front of the deck as shown in Figure 6a, or the location along the bottom of the deck of the back face of the wave crest, as shown in Figure 6b, when the crest has passed the front of the deck.

Using the derivation above, a spreadsheet has been created to implement the wave-in-deck force calculations. Careful attention has been paid to phasing of the horizontal and vertical wave-in-deck loads as well as the phasing of the wave loads on the legs. As in the Shell model, Airy waves were used for ease of application. The still water level was adjusted to ensure the crest elevation of the Airy wave coincided with that of the true crest elevation using a higher order wave theory. The wave-in-deck loads were applied to the structural model as a series of nodal forces.

4.1.3 Statoil wave-in-deck force model

The Statoil wave-in-deck model was developed into a spreadsheet to allow a comparison with the MSL model. The model is much simpler than the MSL model, as vertical wave and buoyancy loads are not taken into account. The horizontal load on the front face of the hull was taken as:

$$F = \frac{1}{2} \rho C_s b_p \int_{h_d}^{\eta} u^2 dz \qquad (6)$$

where C_s was assumed to be 3.0. Stokes 5th order wave theory was used.

The distribution of particle velocity, u, with height was taken from the SACS output. The overall impact force was then determined using numerical integration in a spreadsheet, and applied to the structural model as a series of nodal loads.

4.2 Wave Loading on Legs

The legs were modelled in sufficient detail for representative drag and inertia loads to be generated, using the drag and inertia coefficients outlined in Section 3.2.

SNAME 5-5A recommends that a deterministic wave height of 1.6Hs should be used to generate representative extreme loads. This factor implicitly includes a wave kinematics factor equal to 1. As the crest elevation is particularly important in the present study, the true maximum wave height (equal to 1.86Hs) was used to accurately represent the crest elevation. The kinematics factor was reduced to 0.84 to match the wave loads with the SNAME recommendations using a deterministic wave height of 1.86Hs.

4.3 Wind Loading

Wind loads were applied to the leg members above the water level using the same drag coefficient as was used in the wave loading calculations. The parts of the legs inside the hull structure attracted no wind load.

Wind loads on the hull structure and its equipment were modelled by creating a "Wind Area" of $3000m^2$ within SACS. 50-year wind loads were applied with the 10,000-year wave conditions, ie. no attempt was made to estimate a joint probability value for wind speed.

4.4 Dead loads

The structural selfweight of the legs and hull structure were automatically generated within SACS. Operating and equipment loads were applied as point loads to nodes raised above the hull structure by a suitable distance to give a realistic topsides centre of gravity.

4.5 Inertia Loads

Inertial loads on the structure are created due to the dynamic interaction between the period of the wave and the natural period of the structure. The dynamic amplification factor (DAF) is at a maximum when the natural period of the structure and the period of the wave are the same.

SNAME 5-5A recommends a number of calculation methods of varying complexity to determine the DAF due to wave loading on the legs. The method used in this analysis was to take a previously determined DAF for the Maersk Endurer (13) and to modify it to suit the natural period of the structure in its present configuration using a Single Degree of Freedom model. Although this method may be more simple than some of those outlined in SNAME, it provided a consistent set of inertial loadsets across the range of analyses performed. Changes in DAF for wave loading on the legs did not greatly affect the total base shear and overturning moment once wave-in-deck loads are considered. Once the appropriate DAF was calculated, the inertial loadset was determined using the method outlined in SNAME 5-5A.

4.6 Phasing of Loads

The maximum horizontal loads and vertical loads due to wave dynamics and buoyancy caused by the wave inundating the deck do not occur simultaneously. In addition, the maximum base shear and overturning moment from the wave on the legs is likely to be out of phase with the wave-in-deck loads. When creating the loadset to be used in the pushover analyses, the phase angle was chosen to give the maximum total overturning moment and base shear (both occurring at the same time).

5. SITE ASSESSMENT

5.1 Introduction

A simple site assessment of the Jack-up was performed in order to

(i) Determine the maximum member utilisation using an environmental load factor of 1.25 with a LAT of 90m.

(ii) Determine the increased water level that would give the same maximum member utilisation when a load factor of 1.15 is used.

It should be noted that the site assessment performed was limited to the extreme in-place condition only.

5.2 Input Parameters

The following input parameters, representative of conditions in the Central North Sea, were used in the site assessments:

LAT	=	90m
Hs	=	12.4m (50 year value)
Peak period	=	15.3sec
Hmax	=	23.1m (=1.86Hs)
Current	=	0.67m/s
1 minute mean wind speed	=	40.1m/s.

The above wave data gave the following extreme wave crest:

Crest height = 105.4m (Trough depth = 82.3m)

This led to a minimum hull elevation of 106.9m, using the SNAME recommended practice of adding 1.5m to the 50 year crest height.

5.3 Utilisations and Increased Water Level for Load factor of 1.15

Using the loading parameters outlined in Section 5.2, a maximum member utilisation of 1.0 was determined on a diagonal chord near the leg/hull connection on the compression leg with an environmental load factor of 1.25. The yield stress of the primary members required to give a utilisation of 1.0 was 607 N/mm^2.

The water level was then increased to such a level that the maximum member utilisation remained at unity, whilst using an environmental load factor of 1.15. The hull elevation was increased to 1.5m above the revised 50 year wave crest height. The change in inertial loading caused by the increased natural period at the raised hull elevation was taken into account in the analysis. The raised value of LAT which gave a maximum utilisation of unity was 96.6m – i.e. 6.6m above the original water depth.

5.4 Hull Elevations for Pushover Analysis

For the pushover analyses, four hull elevations for each of the two water levels were chosen to demonstrate the effect of increasing wave in hull inundation on environmental loading and consequently on the ELR. The maximum and minimum hull elevations were defined as:

Minimum elevation
(Position a): 50 year wave crest elevation + 1.5m

Maximum elevation
(Position d): 10,000 year wave crest + 0m (i.e. 50 year crest + 5.6m)

Two intermediate hull elevations were chosen between the maximum and minimum elevations, referred to as positions ‘b’ and ‘c’:

Position b: 50 year crest + 2.7m (roughly equivalent to 1,000 yr wave elevation)

Position c: 50 year crest + 4.3m (roughly equivalent to 5,000 year wave elevation)

Positions a to d give a range of deck inundations ranging from zero (position d) to over 4m (position a).

The deck elevation was changed relative to the mudline by extending or shortening the plated sections at the bottom of the legs. This meant that the connection between the hull and the leg remained constant - i.e. the location of the top and bottom guides relative to the leg was unchanged. This ensured that new failure mechanisms were not introduced as a result of a change in connection geometry, and hence the pushover results could be directly compared. The lower guide was positioned at mid bay level.

Changes in deck elevation have the following consequences, which have been modelled:

(i) Change in the wave-in-deck loading

(ii) A change in the natural period of the structure, resulting in a change in inertial loading.

(iii) Change in the length of leg susceptible to wave loading, as the portion of the leg inside the hull is assumed to attract no load

6. RESULTS

6.1 Loadcases Analysed

A summary of the Pushover loadcases analysed is shown in Table 1. The cases were chosen to demonstrate the effect of changes in environmental load factor, hull position, wave-in-deck load model, wave height prediction method and foundation fixity on the pushover response of the structure.

6.2 Loads Generated

6.2.1 Introduction

A summary of the loads due to waves on the legs, wave-in-deck loads, wind and inertia is shown in Table 2. The loads presented for each individual loadcase occurred at the time of maximum combined overturning moment. These loadsets were subsequently used for the pushover analyses.

6.2.2 Effect of inundation on global loads

Figure 7 shows the loads generated for analysis runs 1, 8 10 and 12. These runs represent the structure with SNAME fixity, 90m LAT water depth, wave height prediction based on SNAME recommendations, and the MSL wave-in-deck model. The following may be observed:

(i) The overall base shear increases from 38.2MN (no inundation) to 51.8MN (4.1m inundation), an increase of almost 36%.
(ii) The overall overturning moment increases from 3246MNm to 5971MNm, an increase of 84%.
(iii) For an inundation level of 4.1m, the wave-in-deck loads represent 33% of the total base shear and 52% of the overturning moment.
(iv) Base shear and overturning moment (wave only) for the 50-year wave is approximately half that of the 10,000-year wave with no inundation. These values are indicated on Figure 7.

The wave loading on the legs gradually reduces with increasing inundation, as it has been assumed that no wave loading occurs on the part of the leg inside the hull. The inertia loading decreases very slightly with increasing inundation, as the hull is lower and hence the natural period is lower, moving the natural period of the structure away from the period of the wave.

Figure 8 shows the loads generated for analysis runs 2, 9, 11 and 14. These runs are similar to those presented in Figure 7, except the initial environmental load factor was 1.15. The values of base shear generated are very similar to those derived for a 90m LAT water depth, with the exception of the inertial loading, which is slightly greater. This is due to the increased natural periods (as a result of the increased hull height and hence longer leg length) increasing the dynamic amplification factor.

Table 1
Pushover Analyses using 10,000-year wave

Loadcase	*Water Depth based on Load Factor of*	*Hull position*	*Hull elevation*	*Inundation (m)*	*Fixity*	*Wave-in-deck Model*	*Wave height prediction method*
1	1.25	1a	50 year crest + 1.5m	4.1	SNAME 5-5A	MSL	SNAME 5-5A
2	1.15	2a	50 year crest + 1.5m	4.1	SNAME 5-5A	MSL	SNAME 5-5A
3	1.15	2a	50 year crest + 1.5m	4.1	"Measured"	MSL	SNAME 5-5A
4	1.15	2a	50 year crest + 1.5m	3.3	SNAME 5-5A	MSL	HSE Guidance
5	1.15	2a	50 year crest + 1.5m	4.1	SNAME 5-5A	Statoil	SNAME 5-5A
6	1.15	2a	50 year crest + 1.5m	3.3	SNAME 5-5A	Statoil	HSE Guidance
7	1.15	2a	50 year crest + 1.5m	4.1	Pinned	MSL	SNAME 5-5A
8	1.25	1b	≈ 1,000 year crest + 0m	2.8	SNAME 5-5A	MSL	SNAME 5-5A
9	1.15	2b	≈ 1,000 year crest + 0m	2.8	SNAME 5-5A	MSL	SNAME 5-5A
10	1.25	1c	≈ 5,000 year crest + 0m	1.2	SNAME 5-5A	MSL	SNAME 5-5A
11	1.15	2c	≈ 5,000 year crest + 0m	1.2	SNAME 5-5A	MSL	SNAME 5-5A
12	1.25	1d	10,000 year crest + 0m	0	SNAME 5-5A	-	SNAME 5-5A
13	1.25	1d	10,000 year crest + 0m	0	"Measured"	-	SNAME 5-5A
14	1.15	2d	10,000 year crest + 0m	0	SNAME 5-5A	-	SNAME 5-5A
15	1.15	2d	10,000 year crest + 0m	0	"Measured"	-	SNAME 5-5A
16	1.15	2d	10,000 year crest + 0m	0	Pinned	-	SNAME 5-5A

Table 2
Environmental Loading Summary

Loadcase	*Wave-in-deck*			*Wave on Legs*		*Wind*		*Inertia*	
	Hor, kN	*Ver, kN*	*Moment, MNm*	*Hor, kN*	*Moment, MNm*	*Hor, kN*	*Moment, MNm*	*Hor, kN*	*Moment, MNm*
1	17100	77370	3085	26722	1477	3075	382	4916	1027
2	16540	78090	3123	28004	1590	3072	403	5583	1228
3	16540	78090	3123	28004	1590	3072	403	974	213
4	11700	59120	2346	26023	1461	3081	404	5075	1078
5	20240	-	2334	28004	1590	3072	403	5583	1228
6	14960	-	1722	26023	1461	3081	404	5075	1078
7	16540	78090	3123	28004	1590	3072	403	4781	1050
8	11870	46770	2168	27323	1497	3094	388	5081	1052
9	11470	47250	2198	28577	1613	3092	409	5734	1240
10	5186	14560	919	28233	1543	3108	395	5335	1100
11	5013	14730	934	29426	1662	3106	415	5935	1272
12	-	-	-	29533	1655	3106	398	5641	1193
13	-	-	-	29533	1655	3106	398	1004	212
14	-	-	-	30228	1768	3204	431	6188	1363
15	-	-	-	30228	1768	3204	431	1092	239
16	-	-	-	30228	1768	3204	431	5157	1139

6.2.3 Effect of wave-in-deck model on generated loads

The wave-in-deck loads predicted using the MSL and Statoil methods are shown in Table 3. The most obvious difference in the two methods is the large upward load predicted by the MSL method, as compared to no vertical loads generated using the Statoil method. The vertical load, mainly caused by buoyancy effects, is 78090kN (7960t) for an inundation of 4.1m. This load represents approximately 45% of the total dead and operating vertical loading on the structure. The buoyancy load also acts to increase the total overturning moment. The Statoil method predicts a higher base shear than the MSL method, although this value is directly dependent on the slamming coefficient used (equation 6). As a result of the large base shear, the overturning moment in the Statoil method is only approximately one third less than that of the MSL method.

Table 3
Comparison of Wave-in-deck forces, SHELL and Statoil methods

Loadcase 2		***Loadcase 5***	
Wave-in-deck model: MSL		Wave-in-deck model: Statoil	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: SNAME 5-5A	
Inundation: 4.1m		Inundation: 4.1m	
Wave-in-deck Loads:		Wave-in-deck Loads:	
Base shear:	16540kN	Base shear:	20240kN
Vertical (upwards):	78090kN	Vertical (upwards):	0
Overturning Moment:	3123MNm	Overturning Moment:	2334MNm

Loadcase 4		***Loadcase 6***	
Wave-in-deck model: MSL		Wave-in-deck model: Statoil	
10,000 yr wave height: HSE Guidance		10,000 yr wave height: HSE Guidance	
Inundation: 3.3m*		Inundation: 3.3m*	
Wave-in-deck Loads:		Wave-in-deck Loads:	
Base shear:	11170kN	Base shear:	14960kN
Vertical (upwards):	59120kN	Vertical (upwards):	0
Overturning Moment:	2346MNm	Overturning Moment:	1722MNm

*ie. Crest height for HSE Guidance method 0.8m lower than the SNAME method.

6.2.4 Effect of extreme wave height prediction on generated loads

Two methods for determining the 10,000-year crest have been investigated, one based on the SNAME 5-5A recommendations and one based on the now withdrawn HSE Guidance (modified using site specific data). The difference in the crest elevations was 0.8m. The wave-in-deck loads resulting from the two methods are shown in Table 3. Loadcases 2 and 5 use the SNAME wave height while loadcases 4 and 6 use the HSE Guidance. It can be seen that although the difference in inundation is relatively small, the change in loading is significant, with overturning moment reducing by approximately 25%.

6.3 Typical Pushover Response

The 16 pushover analyses were performed using an enhanced yield stress, ie. 14% greater than that used for the site assessments. An increase in the nominal yield stress is often used in pushover analyses to model a more realistic value of yield stress.

Figure 9 shows the pushover response for Case 8, which had an inundation of 2.8m, with wave-in-deck loads determined using the MSL method.

The global displacement of the structure is approximately linear with applied load. A significant non-linear response would not be expected as linear foundations have been assumed, and the structure has very little redundancy. In Case 8, where there is significant upwards loading, the structure gets slightly stiffer due to the reducing average compression load in the legs as the environmental loading is increased.

From the individual graphs of structural response shown in Figure 9, the following behaviour can be observed:

(i) The moments at the foundations increase relatively linearly with applied load.

(ii) The horizontal foundation loads on the compression leg (leg 3) takes a smaller percentage of the horizontal load as the applied load is increased, caused by the increasing compression in this leg reducing its lateral stiffness. The two tension legs take progressively more horizontal loading as a result.

(iii) The initial vertical foundation loads are extremely similar, with each leg taking approximately one third of the weight of the structure. The vertical foundation reactions respond linearly with the applied loading. It can be seen that the two "tension" legs go into tension (and hence indicate leg lift off) approximately simultaneously (initially at an Extreme Load Ratio (ELR) of 0.697).

(iv) The combined pinion loads on the compression leg increase linearly with applied load. The loads in the compressive racks a and b become more compressive until they exceed the combined pinion capacity of 73557kN at an ELR of 0.879.

(v) Ultimate structural capacity is reached at an ELR of 1.06, where the stiffness matrix was no longer solvable. The global horizontal displacement at this load factor is greater than 8m.

6.4 Failures / Extreme Load Ratios

6.4.1 Introduction

A summary table of the load factors at which failure occurs (i.e. the ELR for the given failure mechanism) is shown in Table 4. The values of ELR refer to the 10,000 year wave, rather than the 50 year or 100 year wave which is traditionally used to define the Reserve Strength Ratio (RSR). For the special case of no inundation, the RSR is approximately twice the value of the ELR.

Foundation “failures” have been determined by comparing foundation loads with capacities determined based on a preload of 10,200t (100MN). This preload was determined as being equal to the largest vertical load likely to occur under the 50-year wave conditions[13]. The resulting horizontal and rotational capacities therefore only refer to the assumed preload and should not be seen as an ultimate capacity.

6.4.2 Effect of increasing inundation

The effect of increasing inundation on the 10,000 year ELR is shown in Figure 10, for environmental load factors of 1.15 and 1.25. The MSL wave-in-deck model and SNAME 5-5A foundation fixity were used.

It may be seen that for an environmental load factor of 1.25 (upper diagram), structural failure, leg lift off and pinion failure are not predicted to occur for the zero inundation case. As inundation levels increase, however, the ELR reduces rapidly. At an inundation of 1m, pinion failure and leg lift off are predicted at an ELR of less than unity. The ELR for leg lift off reduces extremely rapidly with increasing inundation, due to the large buoyancy loads generated, and becomes the dominant failure criteria (excluding foundation sliding, moment and vertical failures) at inundations greater than 1m.

The ELR values for an initial load factor of 1.15 are shown in the lower diagram of Figure 10. It may be seen that the general pattern of results are very similar to those for the load factor of 1.25 discussed above.

6.4.3 Effect of Initial Environmental Load Factor on ELR

The effect of the original environmental load factor used in the site assessment on the 10,000 year ELR is shown in Figure 11.

The gap between the ELR values for pinion failure remains fairly constant for increasing inundation levels. The difference is approximately 10%, similar to the difference in original load factor.

The gap between the ELR values for leg lift off gets smaller with increasing inundation levels, with a difference of only 7% at an inundation of 4.1m. Leg lift off is largely a function of the buoyancy component of the wave-in-deck load, and is not present in the original Site Assessment. The buoyancy load is primarily a function of the level of hull inundation, and is not significantly affected by still water depth.

Table 4
Summary of Failure ELR's

Loadcase	*Failure Criterion*						
	Pinion	*First plasticity*	*Foundation capacities **				*Ultimate capacity*
			Leg lift off	*Horizontal*	*Moment*	*Preload exceeded*	
1	0.842	0.7	0.574	0.63	0.386	0.478	0.98
2	0.718	0.7	0.533	0.606	0.372	0.443	0.9
3	1.909	1.6	0.906	0.783	0.182	1.537	1.92
4	0.837	0.9	0.647	0.705	0.423	0.508	1.06
5	0.57	0.7	0.661	0.56	0.327	0.339	0.76
6	0.679	0.8	0.788	0.654	0.388	0.404	0.90
7	0.598	0.7	0.485	0.638	-	0.379	0.71
8	0.879	0.8	0.697	0.685	0.439	0.482	1.08
9	0.753	0.8	0.641	0.655	0.404	0.446	0.94
10	0.97	0.9	0.934	0.775	0.505	0.52	1.18
11	0.831	0.9	0.845	0.736	0.461	0.481	1.10
12	1.07	0.9	1.152	0.85	0.558	0.567	1.34
13	-	1.8	-	1.122	0.316	1.825	2.84
14	0.969	0.9	1.04	0.809	0.494	0.507	1.18
15	-	1.8	-	1.105	0.292	1.703	2.26
16	0.878	0.8	0.96	0.846	-	0.459	1.00

6.4.4 Comparison of wave-in-deck models

The influence of the wave-in-deck model on the 10,000 year ELR is illustrated in Table 5. The Statoil method generally predicts failure at lower factors than the MSL method. However, because the Statoil method does not include buoyancy or vertical wave loading, leg lift off is predicted at a higher ELR than the MSL model (ELR = 0.661 for the Statoil method as compared to 0.533 for the MSL model).

The results below may be compared to Loadcase 14, where no inundation occurs (see Table 4). Pinion failure occurs at an ELR of 0.969, leg lift off at 1.04 and ultimate structural collapse at 1.18. These values are much greater than those found in either of Loadcases 2 or 5 and indicates that just by considering wave-in-deck loads, using either wave-in-deck model, then the estimate of system capacity is vastly reduced.

Table 5
Effect of Wave-in-deck Model on ELR

Loadcase 2		***Loadcase 5***	
Wave-in-deck model: MSL		Wave-in-deck model: Statoil	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: SNAME 5-5A	
Inundation: 4.1m		Inundation: 4.1m	
Foundation: SNAME 5-5A		Foundation: SNAME 5-5A	
Results:		Results:	
Leg lift off:	0.533	Leg lift off:	0.661
First plasticity:	0.7	First plasticity:	0.7
Pinion slippage:	0.718	Pinion slippage:	0.570
Structural Collapse:	0.9	Structural Collapse:	0.76

6.4.5 Comparison of extreme wave height prediction method

The effect of changes in the wave-in-deck model is shown in Table 6. The method of extreme wave height prediction has a significant effect on the ELR of the structure. This is primarily caused by the change in inundation level rather than the change in wave loading on the legs (see Table 3).

Table 6
Effect of 10,000 year Wave Height on ELR

Loadcase 2		***Loadcase 4***	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: HSE Guidance	
Inundation: 4.1m		Inundation: 3.3m	
Wave-in-deck model: MSL		Wave-in-deck model: MSL	
Foundation: SNAME 5-5A		Foundation: SNAME 5-5A	
Results:		Results:	
Leg lift off:	0.533	Leg lift off:	0.647
First plasticity:	0.7	First plasticity:	0.9
Pinion slippage:	0.718	Pinion slippage:	0.837
Structural Collapse:	0.9	Structural Collapse:	1.06

Table 6 suggests that the ELR values are extremely sensitive to changes in extreme wave height prediction, and uncertainties in the extreme crest elevation may have the greatest effect on ELR and hence reliability.

6.4.6 Effect of Foundation Fixity Conditions

The effect of three types of foundation fixity has been assessed in this study. Increased rotational fixity at the base of the legs reduces the leg moment at the leg/hull connection, hence reducing chord axial loads and pinion loading, as well as reducing inertial loading by moving the natural period of the structure away from the period of the extreme wave. ELR values for two sets of cases, firstly with 4.1m inundation (column 1) and no inundation (column 2) are shown in Table 7. The MSL wave-in-deck model was used where inundation occurred.

The ELR values for the SNAME “extreme” soil stiffness and the pinned case are similar, although the SNAME stiffness gives slightly higher ELR values for pinion slippage in particular. This demonstrates that the SNAME fixity, although relatively flexible, still generates a significant moment at the base of the legs, hence relieving leg/hull connection loads.

The ELR values for the "Measured" case are much greater than for either the SNAME or pinned case. The Measured fixity is relatively close to the fully fixed condition, with similar natural periods being determined [(6)]. This measured fixity does not represent the fixity of the foundations at failure and hence is probably unsuitable for use in an extreme event pushover analysis.

Table 7
Effect of Foundation Fixity on ELR Values

Loadcase 2		***Loadcase 14***	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: SNAME 5-5A	
Inundation: 4.1m		Inundation: 0m	
Wave-in-deck model: MSL		Wave-in-deck model: -	
Foundation: SNAME 5-5A		Foundation: SNAME 5-5A	
Results:		Results:	
Leg lift off:	0.533	Leg lift off:	1.04
First plasticity:	0.7	First plasticity:	0.9
Pinion slippage:	0.718	Pinion slippage:	0.969
Structural Collapse:	0.9	Structural Collapse:	1.18

Loadcase 3		***Loadcase 15***	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: SNAME 5-5A	
Inundation: 4.1m		Inundation: 0m	
Wave-in-deck model: MSL		Wave-in-deck model: -	
Foundation: Measured Data		Foundation: Measured Data	
Results:		Results:	
Leg lift off:	0.906	Leg lift off:	-
First plasticity:	1.6	First plasticity:	1.8
Pinion slippage:	1.909	Pinion slippage:	-
Structural Collapse:	1.92	Structural Collapse:	2.26

Loadcase 7		***Loadcase 16***	
10,000 yr wave height: SNAME 5-5A		10,000 yr wave height: SNAME 5-5A	
Inundation: 4.1m		Inundation: 0m	
Wave-in-deck model: MSL		Wave-in-deck model: -	
Foundation: Pinned		Foundation: Pinned	
Results:		Results:	
Leg lift off:	0.485	Leg lift off:	0.960
First plasticity:	0.7	First plasticity:	0.8
Pinion slippage:	0.598	Pinion slippage:	0.878
Structural Collapse:	0.71	Structural Collapse:	1.0

6.5 Discussion of Airy Wave Approximation

Wave-in-deck loads in the MSL model have been determined using an Airy wave approximation to facilitate spreadsheet implementation. Airy waves are sinusoidal, whereas higher order waves have a pronounced peak at their maximum surface elevation. Example surface profiles of Airy and Stokes 5^{th} order waves are shown in Figure 12. A refinement to the MSL wave-in-deck model would be to include higher order waves which would then have the following loading effects:

(i) The buoyancy on the hull may be reduced by approximately 30%. This will in turn reduce the susceptibility of the jack-up to leg lift-off.
(ii) The duration of the wave-in-deck loading would be reduced by approximately 35%.
(iii) Horizontal crest velocity may be slightly increased, hence increasing horizontal wave-in-deck loading.

The dynamic response of the jack-up to wave-in-deck loads has not been considered during the present study, ie. the DAF for wave-in-deck loads has been assumed to be unity. The reduced loading duration described in item (ii) for higher order waves is likely to reduce the DAF. A simple initial assessment has shown that the DAF for wave-in-deck loading may lie between 0.5 and 0.7 for inundations greater than 3m, based on a structural natural period of 9.5 seconds. However, it should be noted that if a stiffer foundation is assumed that the DAF may become greater than unity, and hence using a relatively soft foundation may give lower total global loads.

7. CONCLUSIONS

The following conclusions have been made as a result of the analyses presented in this report:

- The reduction of the environmental load factor from 1.25 to 1.15 allowed the water depth in which the unit could operate to be increased from 90m LAT to 96.6m, an increase of 6.6m.

- The Extreme Load Ratio (ELR, based on the 10,000 year wave) values for the no hull inundation case was 1.07 for an environmental load factor of 1.25 (pinion slippage predicted). This decreased to 0.97 when a load factor of 1.15 was used.

- Wave inundation loads are extremely significant when compared to the environmental loads on the legs. For example, at a hull inundation of 4.1m, the wave-in-deck loads represented over 50% of the total overturning moment (using the MSL wave-in-deck model).

- The vertical wave and buoyancy loads generated during inundation were considerable (MSL model). For an inundation of 4.1m, the vertical wave-in-deck load represented approximately 50% of the total weight of the jack-up.

- A more refined wave model, using higher order wave theory, would be expected to lead to a reduction of wave-in-deck loads.

- The largest decrease in ELR was associated with the consideration of wave-in-deck loads, regardless of the wave-in-deck model used.

- The large vertical loads (primarily caused by hull buoyancy) have the effect of increasing the likelihood of leg lift off. For inundations of greater than about 1m, leg lift off became the dominant failure mechanism. The ELR against leg lift off reduced by approximately 0.15 for each metre of inundation.

- Foundations calculated using the SNAME 5-5A recipe and pinned foundations give broadly similar pushover responses. The ELR's determined using a measured rotational fixity were over double the SNAME 5-5A values.

- Wave-in-deck loads are most sensitive to changes in extreme crest height (and hence inundation). Uncertainty in the 10,000 year crest height is likely to have a large effect on structural reliability.

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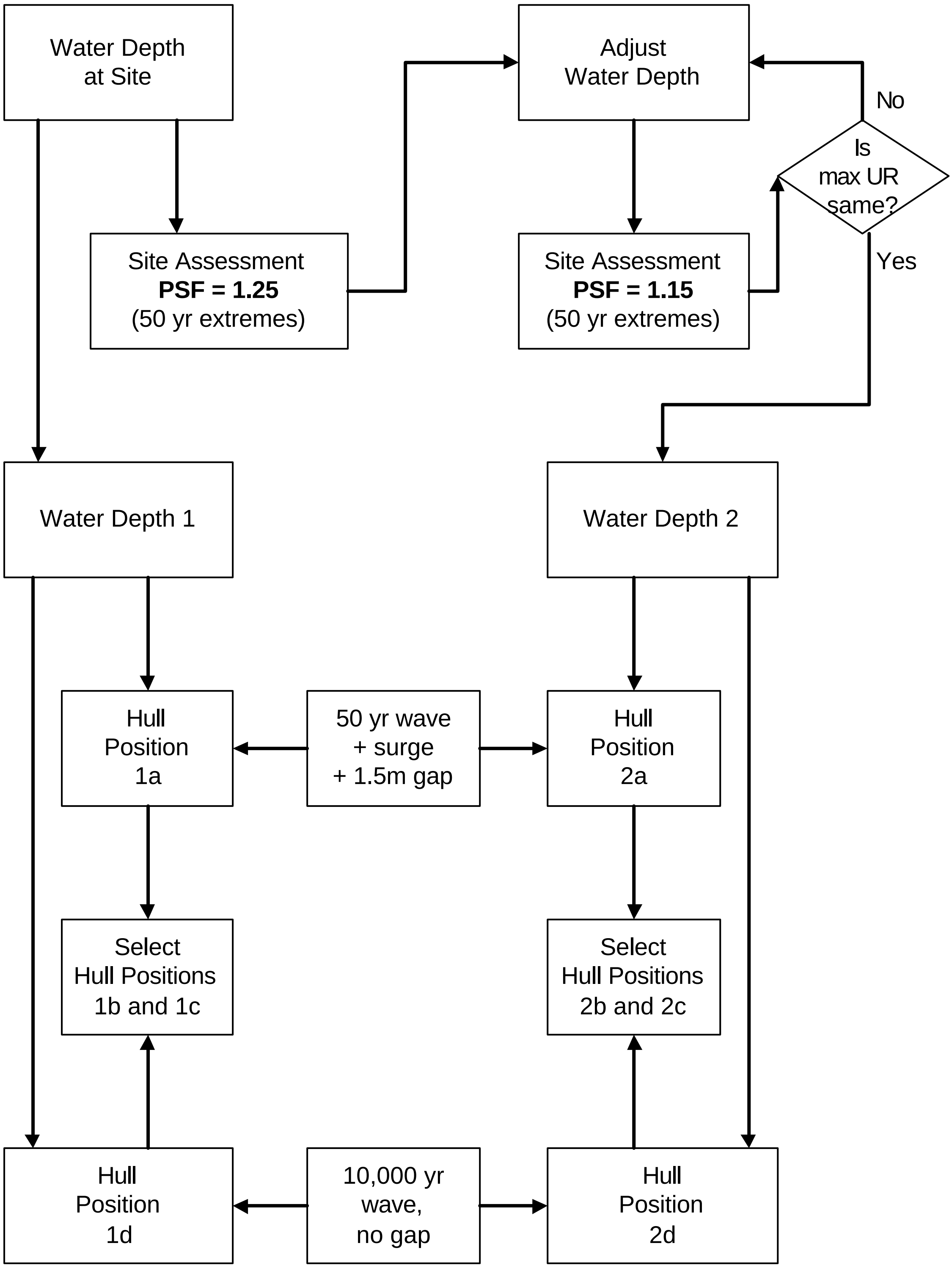


Figure 1
Analysis Flowchart

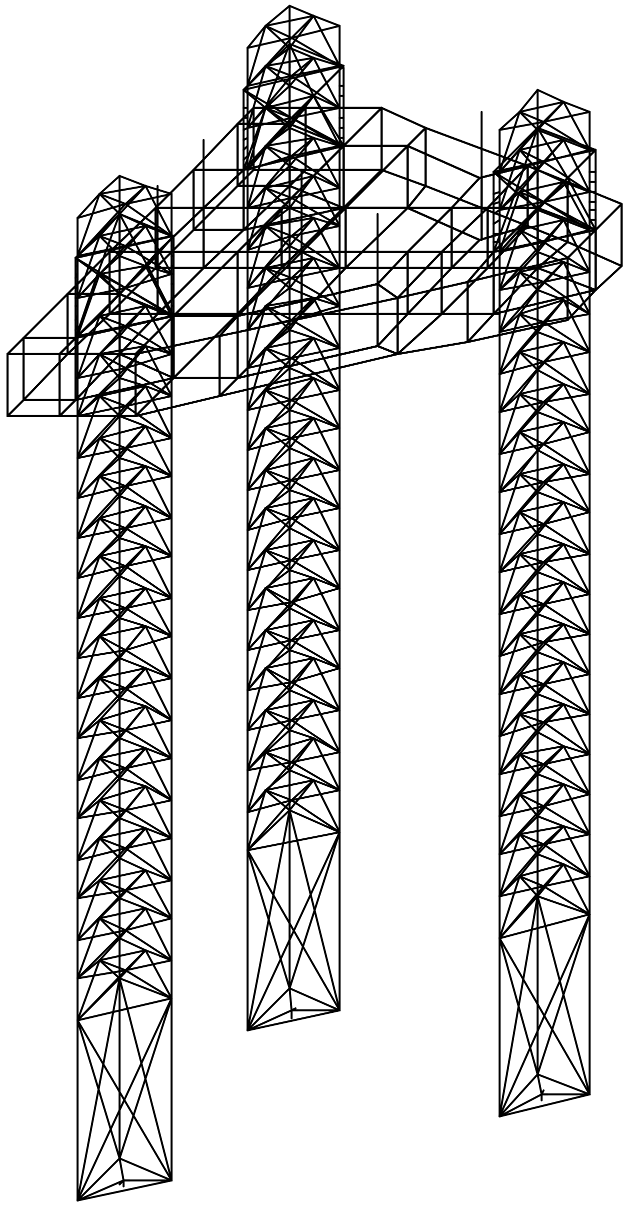


Figure 2
Structural Model

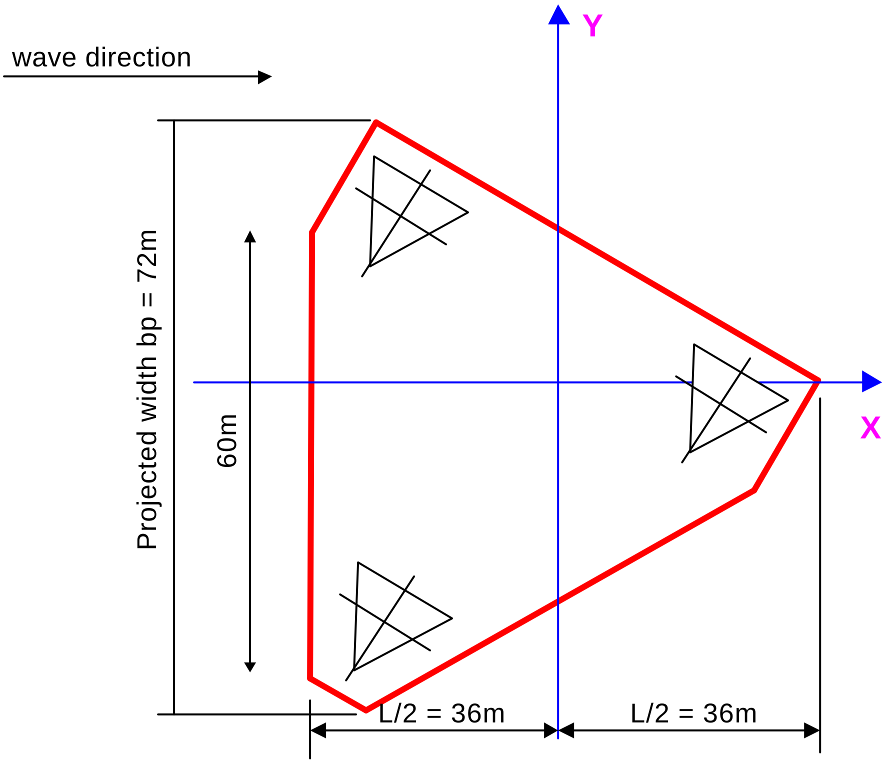


Figure 3
Plan dimensions of Hull

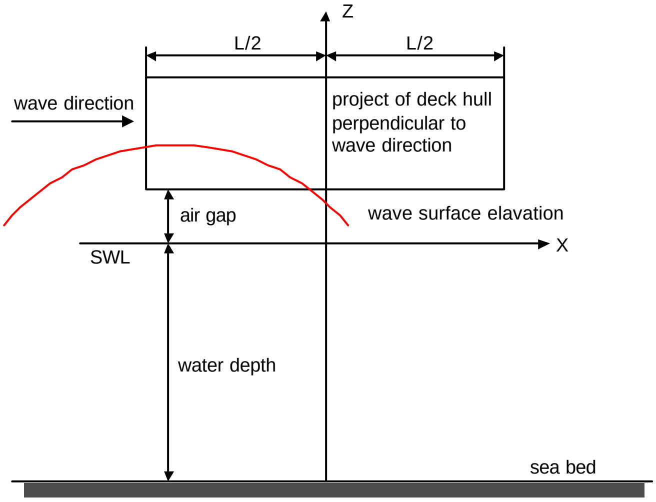


Figure 4
MSL Wave-in-deck Force Model

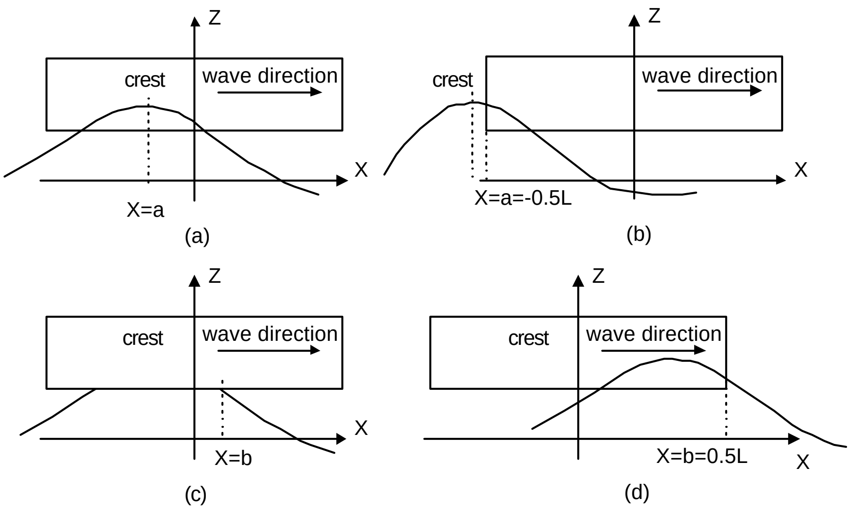


Figure 5
Integral bounds for Vertical Hydrodynamic Force, MSL Model

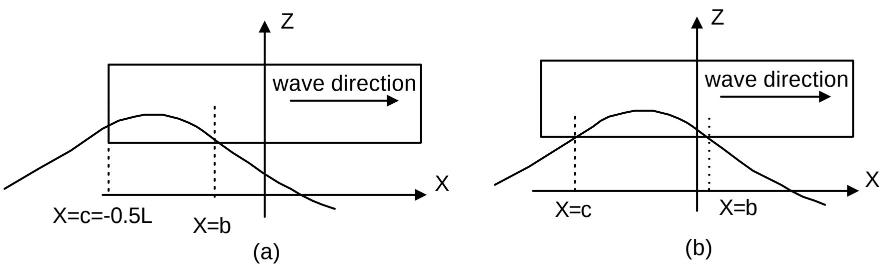
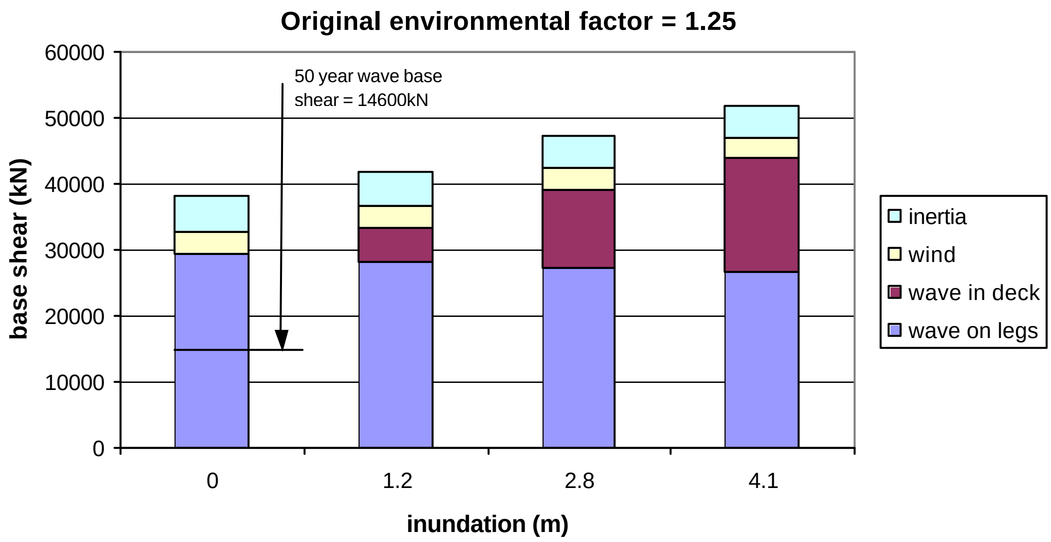


Figure 6
Integral Bounds for Buoyancy, MSL Model



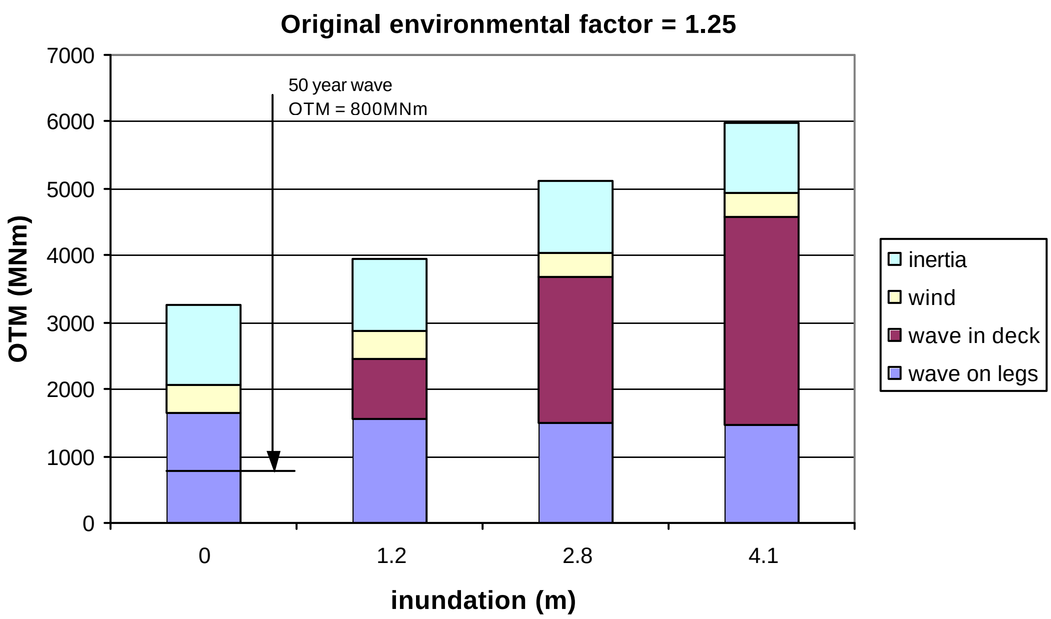
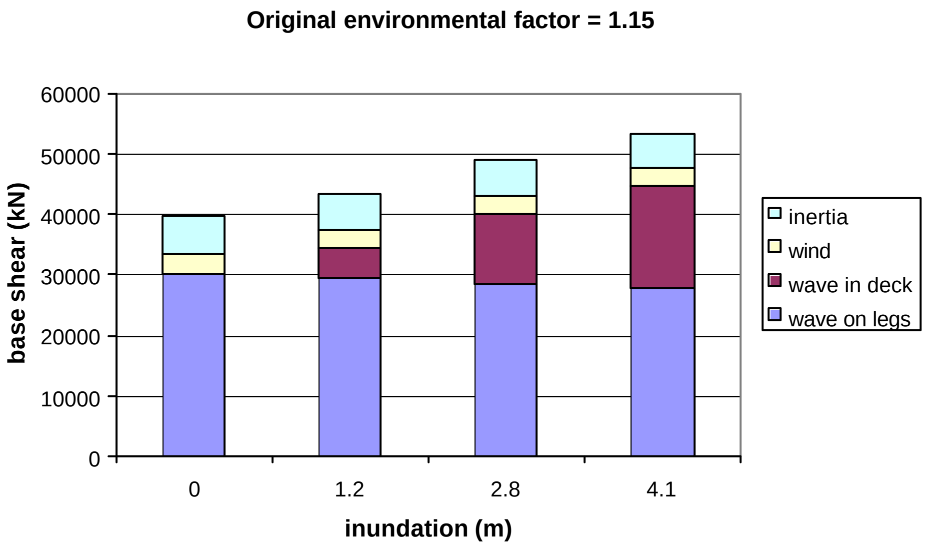


Figure 7
Loads generated, MSL method, 90m LAT



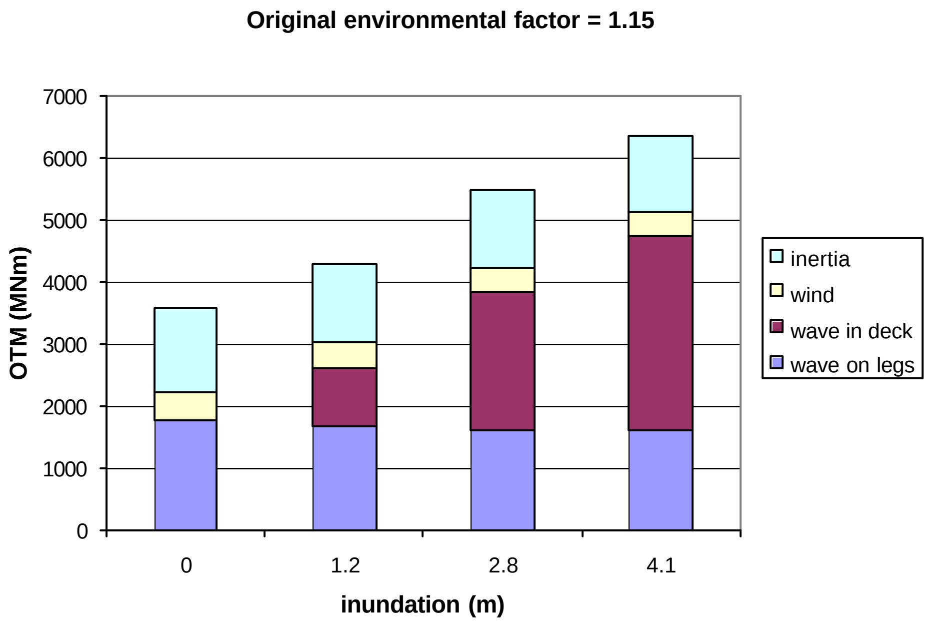


Figure 8
Loads generated, MSL method, 96.6m LAT

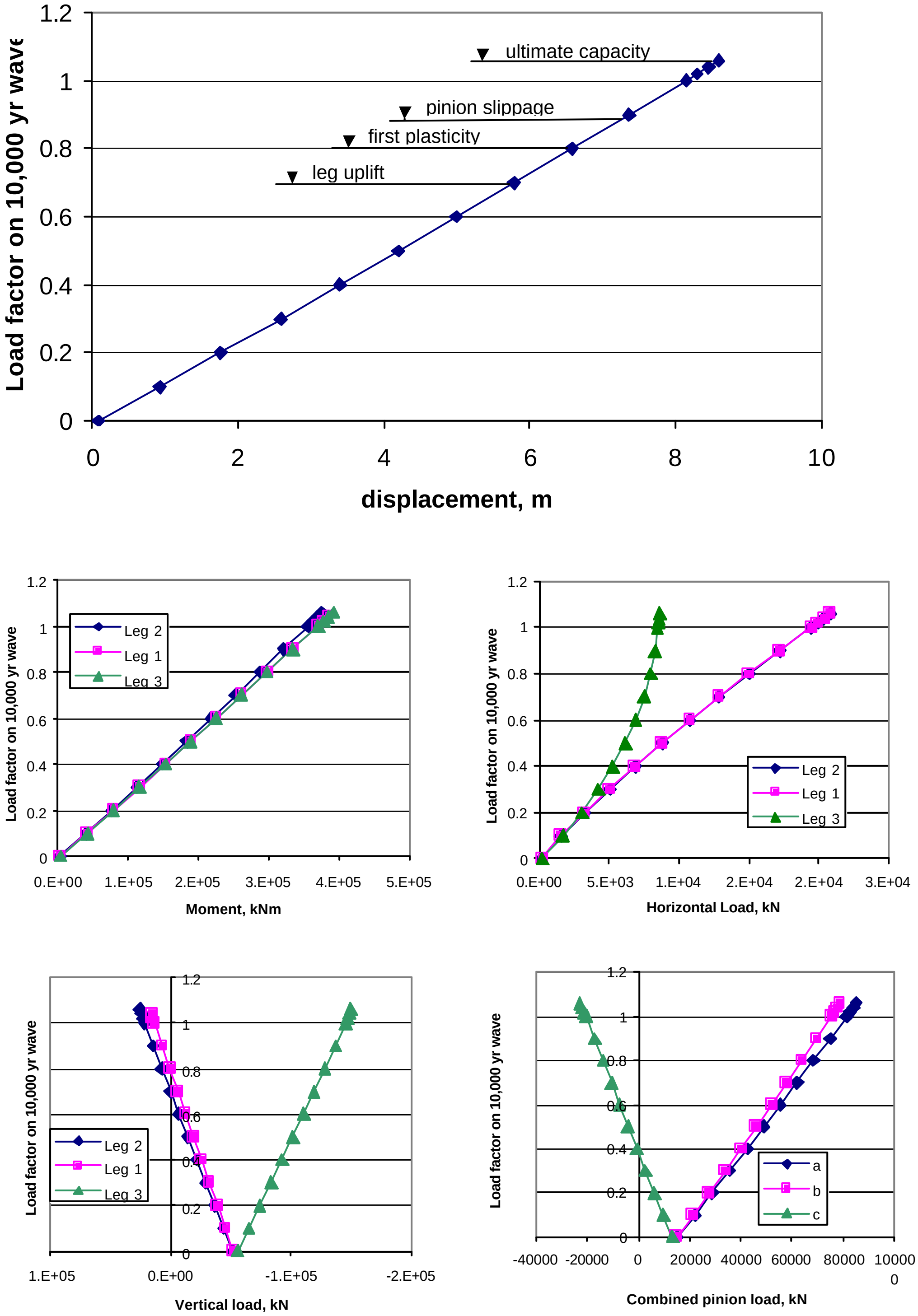
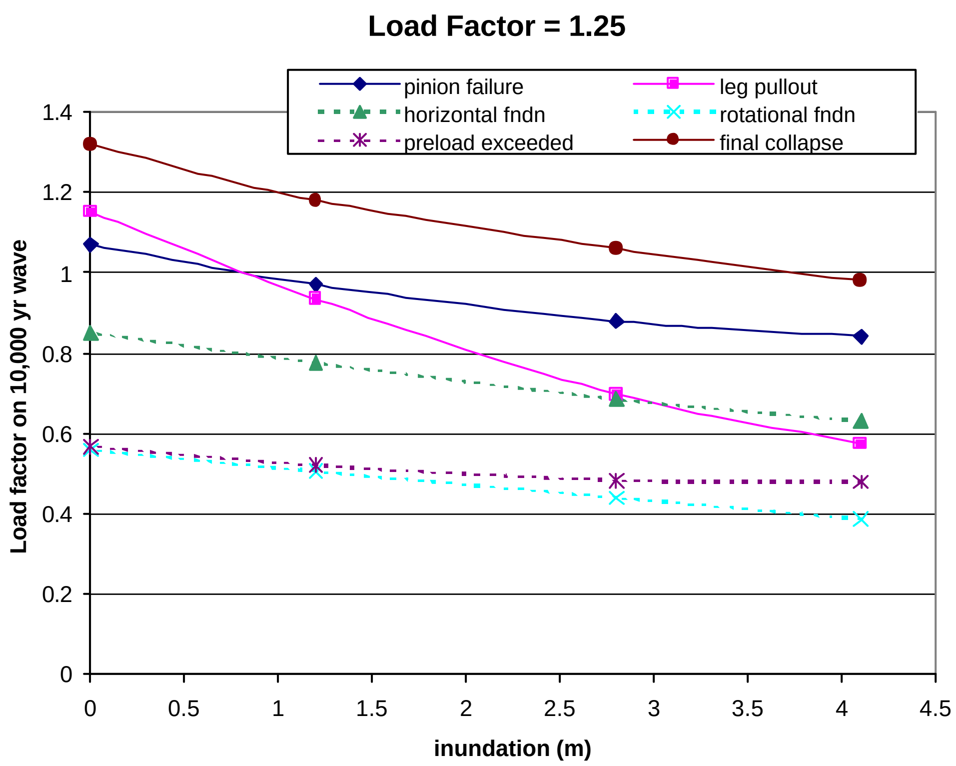


Figure 9
Results for Case 8



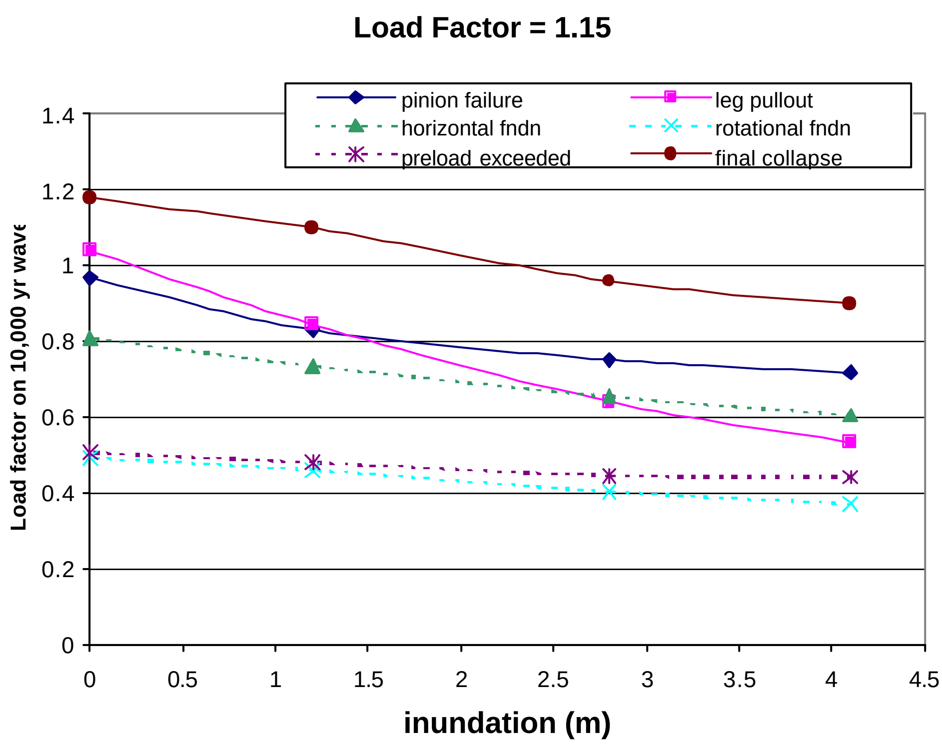
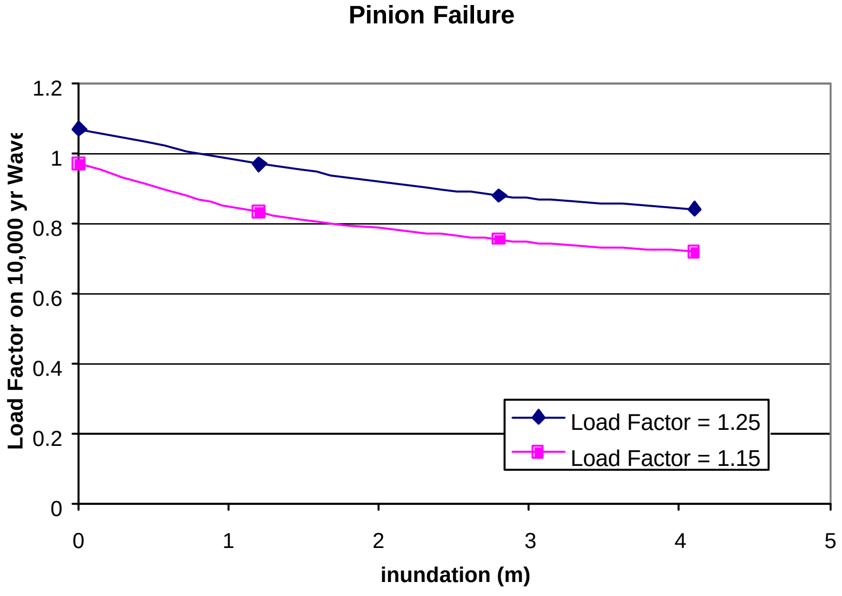


Figure 10
Effect of Inundation on ELR Values



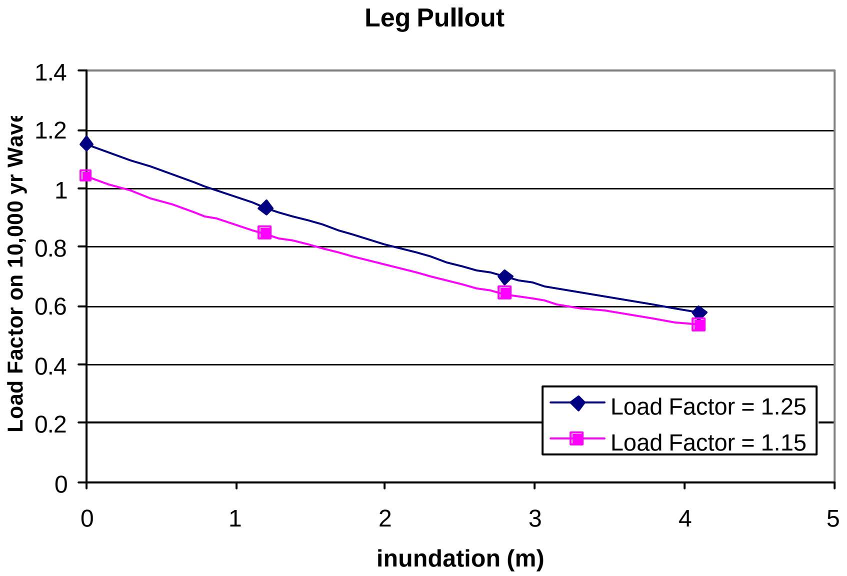


Figure 11
Effect of Environmental Load Factor on ELR Values

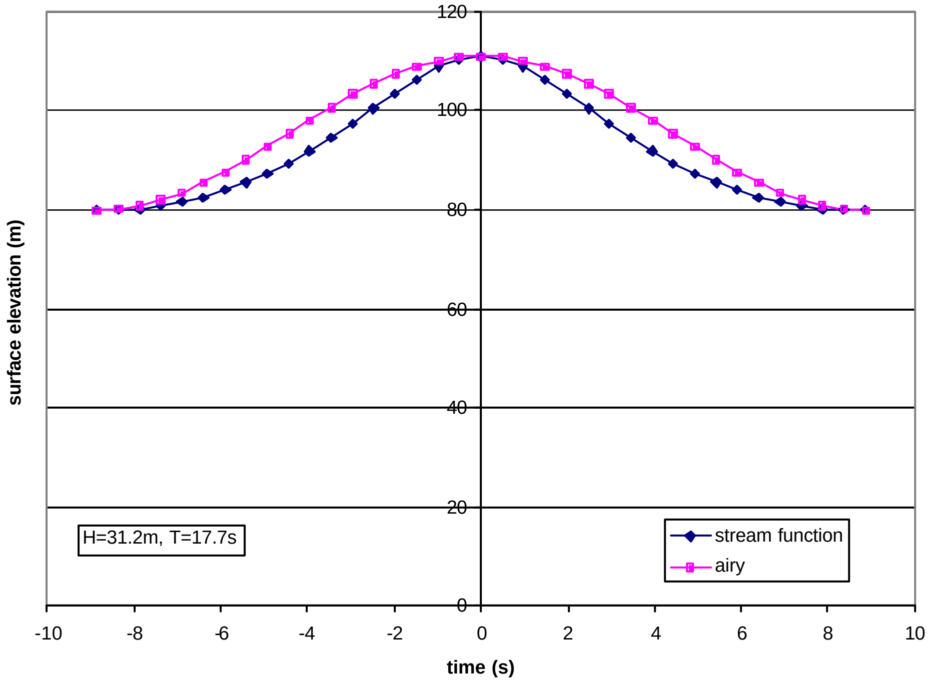


Figure 12
Surface Profile of Airy and Stream Function Waves

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