NOBLE DENTON

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REPORT

IADC

THE CALIBRATION OF SNAME SPUDCAN FIXITY EQUATIONS WITH FIELD DATA

Report No: L19073/NDE/MJRH

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1 SUMMARY

1.1 BACKGROUND

It has been long recognised that one means of increasing the operating capabilities of jack-up units has been the exploitation of greater foundation fixity. Members of the IADC, amongst others, have expressed concern over the ways in which spudcan foundations are dealt with by the present SNAME T&RB 5-5A document (Refs. [1]a, [1]b). The methods used currently employ very simple soils models and recent field measurements suggest that they are overly conservative.

In the short term the major concern is that SNAME T&RB 5-5A (SNAME) requires the use of rather low soil stiffness, that is believed to be unrealistic.

The justification of higher stiffness can only be on the basis of measured field data.

1.2 PROJECT OBJECTIVES

The principal objectives of the project were as follows :

- Preliminary investigation of the sensitivity of derived fixity values to FE modelling assumptions and determination of potential benefits of increased foundation stiffness.
- Back-analysis of instrumented cases to determine calibration factors for the initial stiffness equations in SNAME.
- Recommendations for further developments of fixity methodology.

1.3 APPROACH

In the first instance Noble Denton Europe (NDE) undertook initial studies to respond to queries raised by key industry players and also to determine the benefits that might be obtained from the study.

The calibration of the SNAME equations was achieved by back-analysing case records of jack-ups in the North Sea relating to three different jack-ups at a total of eight locations which include a variety of different soil conditions. At each of the locations instrumentation records were available of the response of the unit in terms of horizontal deck movement under wave loading. The records were obtained from monitoring programmes commissioned by Santa Fe, who have kindly made them available to this project.

The back-analysis, was carried out by Prof. Guy Houlsby and Dr Mark Cassidy of Oxford University (Oxford) under sub-contract to Noble Denton. Oxford's work was submitted to NDE in Ref. [1]. The jack-up data and measurement results used by Oxford were provided by NDE. The models used in Oxford's JAKUP program were verified against the equivalent NDE models.

1.4 CONCLUSIONS

The preliminary investigation showed that, for the Santa Fe jack-up units studied here, the Finite Element model responses are not sensitive to variation in the assumed hull or leg-hull connection stiffness when these are varied by factors of 2 and 5 respectively. Consequently the fixity inferred when comparing analytical and measured responses will not be significantly influenced by these parameters. For deeper water cases, the benefits of increased foundation stiffness were expected to improve utilisation checks by 4-7%, or 6-12% if the preload check is ignored.

The results of the back-analyses show that SNAME formulations, as applied in the original site assessments, under-predict the foundation rotational stiffnesses by a factor of at least 2. The SNAME document provides little guidance as to the selection of the geotechnical input parameters and adjustment factors. The revised formulations proposed here address this issue. Appendix E contains the relevant sections of SNAME showing how the recommendations may be implemented.

The second draft of this report was made available by IADC for Industry comment. Appendix F summarises the comments received, and how they have been handled, together with the full text of the comments. A number of the comments are outwith the scope of this report and should be addressed by the SNAME OC7 panel.

The effect of the increased stiffness resulting from the revised formulations will vary from case to case. In the deeper water examples used here, the utilisation checks reduce by between 2-6%, or 3-9% if the preload check is ignored.

2 INSTRUCTIONS AND SCOPE

2.1 INSTRUCTIONS RECEIVED

Instructions to proceed with the study were received on November 23rd 1999 by fax from Dave Lewis, in his capacity as Technical Secretary to the IADC Jack-Up Rig Committee.

2.2 SCOPE OF WORK

The scope of work for the study is given in the NDE report, "Phase I Proposal" (ref. EQN/206/99/18) dated 25th October 1999. The scope of work is summarised below :

1) a) Undertake studies to investigate the sensitivity of the back-calculated foundation stiffness (required to match the measured natural frequency) to variations in hull stiffness and leg-to-hull connection stiffness.

   b) Investigate the potential benefits to site assessments from foundation stiffness increases of the order anticipated.

2) Review the 8 available rig-locations and available spudcan penetration and soil reports. Determine spudcan foundation stiffness using current SNAME methodology.

3) Liase with Oxford University (Oxford) in the preparation of basic 3-leg FE model. Run calibration tests to ensure that NDE and Oxford models are equivalent for the bounding cases (pinned and fully fixed).

4) Run Oxford models for given soils and determine calibrated linear foundation stiffnesses.

5) Based on above analyses, determine calibration factors to apply to the SNAME spudcan foundation stiffness equations and propose revised text.

6) Make recommendations for the further development of fixity methodology.

3 INTRODUCTION

Increased stiffness of jack-up foundations has two complementary advantages in a jack-up site assessment:

- ***Static Analysis*** – Rotational stiffness at the footing causes the leg bending moments to be distributed more favourably thus reducing loads in the region of the lower guide and consequently the loads in the leg and jacking system. The increase in overall system stiffness also reduces hull sway and 'p-delta' loads.
- ***Dynamic Analysis*** – Making a jack-up stiffer reduces its natural period. The rig natural period is thus generally moved further away from the peak period of the incident waves making the dynamic amplification factor smaller, and causing a consequent reduction in the hull sway and 'p-delta' loads.

At present the site-specific assessment of jack-ups generally uses the guidelines set out in the SNAME T&RB 5-5A document (Ref. [1]). SNAME assumes initial uncoupled elastic soil stiffnesses. The SNAME interaction formulation degrades the rotational stiffness under increasing load towards the yield surface. If the load combination exceeds the yield surface, the footing is said to be in plastic deformation, at which point the rotational stiffness is zero. Recent research indicates that this is a gross simplification of the actual response.

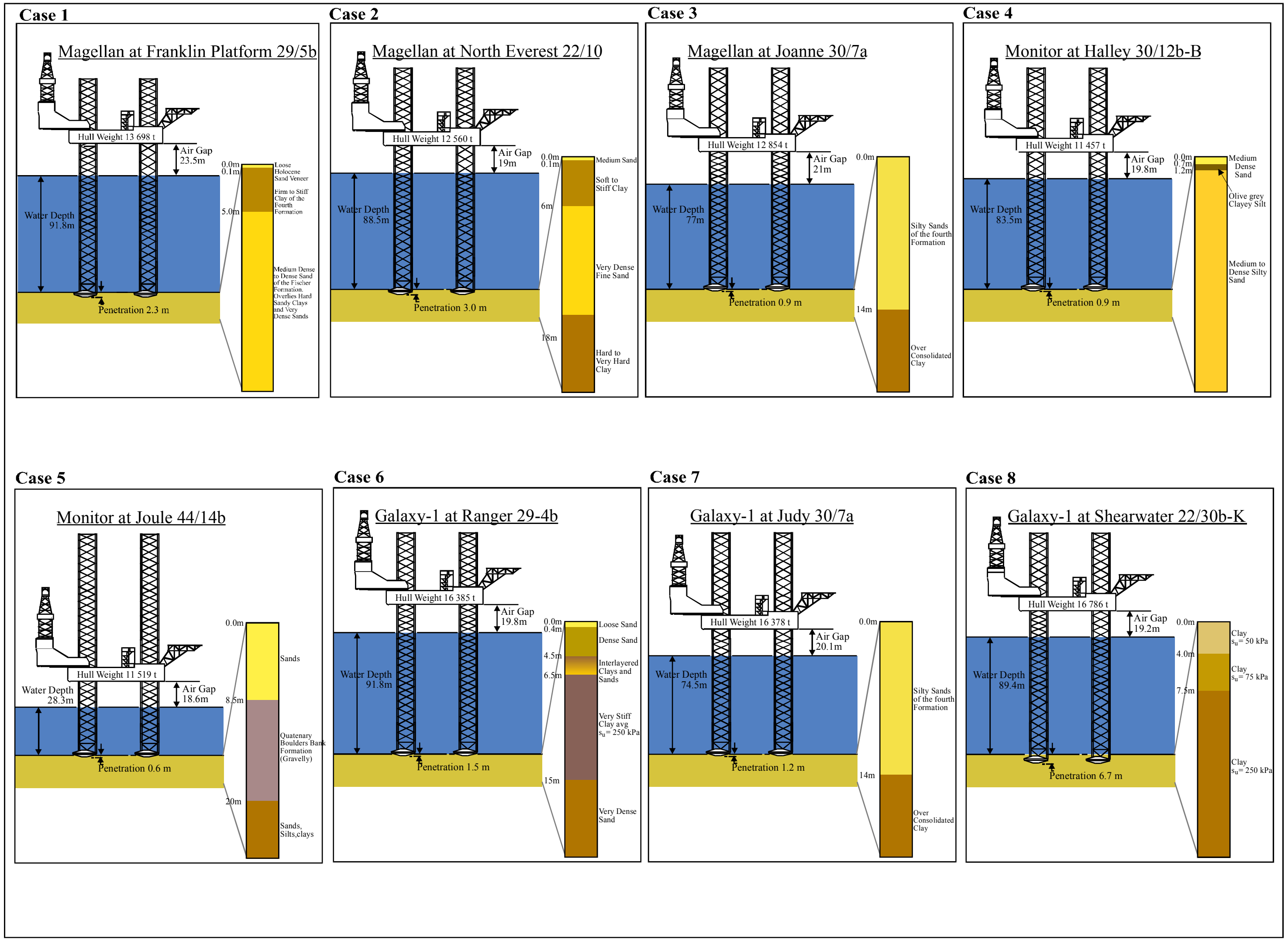
It is also recognised from analysis of measurements made on Santa Fe rigs (Refs. [3] & [4]) that the SNAME approach is conservative in its prediction of the initial small strain soil stiffnesses.

The first stage of the project was intended to allay concerns that had been raised previously and then to confirm that the more detailed work of the second stage would be expected to result in improvements to the results of SNAME assessments.

The purpose of the second stage of the project was to back-analyse case records of jack-ups in the North Sea. The records relate to three different rigs at a total of eight locations which include a variety of different soil conditions. The monitoring data were provided to Oxford University by NDE, and derive from monitoring programmes commissioned by Santa Fe. At each of the locations records were available of the response of the unit in terms of horizontal deck movement under wave loading. Records of the sea state were also available. These data almost certainly represent the best currently available database of monitoring information of full-scale jack-ups under wave loading.

The project has been carried out by NDE with Oxford University working under a sub-contract. The input from Oxford University (Ref. [1]) was provided by Prof. Guy Houlsby and Dr Mark Cassidy (who is now at the University of Western Australia).

4 SITES STUDIED



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5 STAGE 1 ANALYSIS AND RESULTS

5.1 STAGE 1A - HULL / LEG STIFFNESS SENSITIVITY

In this first phase, the NDE 3-leg finite element model was used to investigate the sensitivity of variations in hull stiffness and leg-to-hull connection stiffness on the unit's response by comparing natural periods. This was carried out for the F&G Mod V design in water depths of approximately 30, 60 and 90m (representing the range of water depths in the measurements). Three sets of boundary conditions have been considered – pinned footings, 'SNAME fixity' and 'inferred fixity'. ('Inferred fixity' is the level of fixity which gives the same natural period as measured in the field).

The results for the surge periods (seconds) are tabulated below for the base case and for the base case with either the hull stiffness or the leg-to-hull connection stiffness increased by factors of 2 and 5. Note that the natural periods given below are linear elastic (rather than non-linear) and that added mass has been included.

Table 5-1 : Sensitivity of Surge Period to Hull and Leg-Hull Stiffness

Boundary Condition	28m WD	60m WD	92m WD
Pinned:			
Base Case	4.14	6.95	9.98
Hull * 2	3.92	6.68	9.67
Hull * 5	3.79	6.52	9.48
Connection * 2	4.11	6.92	9.95
Connection * 5	4.10	6.90	9.92
SNAME '97 fixity:			
Base Case	4.47	6.76	9.09
Hull * 2	4.35	6.61	8.92
Hull * 5	4.27	6.52	8.81
Connection * 2	4.46	6.74	9.07
Connection * 5	4.45	6.73	9.06
Inferred fixity:			
Base Case	3.13	4.64	6.21
Hull * 2	3.03	4.54	6.10
Hull * 5	2.97	4.47	6.02
Connection * 2	3.11	4.63	6.21
Connection * 5	3.11	4.62	6.20

5.2 STAGE 1B – BENEFITS FROM FOUNDATION STIFFNESS INCREASES

In order to determine the potential benefits of increased foundation stiffness the Noble Denton JUSTAS program was utilised to determine key analysis results for SNAME '97 fixity parameters and also for the fixity parameters 'inferred' in previous NDE studies, refs. [3], [4]. The JUSTAS program was run for the same cases as were considered in Stage 1A with the inertial loads determined from the time-domain approach, using linearised fixity. The final global response used non-linear fixity.

The key results were compared and are presented in the table below which shows the benefits of the increased foundation fixity on key parameters and on the resulting utilisation checks. The information is presented in terms of the ratio of the 'inferred' case to the SNAME '97 case i.e. 'inferred/SNAME '97.

Table 5-2 : Potential Benefits from Foundation Stiffness Increases

	Monitor at Joule	Magellan at North Everest	Magellan at Franklin (1)	Magellan at Franklin (2)
Site Details:				
Water depth (m)	28.0	88.5	91.2	
Airgap (m)	18.6	19.0	23.5	
Soil type	Sand	Clay	**Clay** on Sand	
Ratios of Key Parameters:				
Foundation Stiffness	3.50	4.15	9.60	4.01
Natural period	0.85	0.81	0.66	0.80
Hull side-sway	0.87	0.94	0.73	0.86
Ratios of Unity Checks:				
Preload UC	0.98	0.96	0.87	0.93
OTM UC	0.96	0.92	0.76	0.88
Leg Strength UC	0.95	0.94	0.77	0.90
Percentage Gains In UC:				
Including preload check	2%	4%	13%	7%
Ignoring preload check	4-5%	6-8%	23-24%	10-12%

(1) Lower bound stiffness, (2) Stiffness used in site assessment

5.3 CONCLUSIONS

5.3.1 Stage 1a - Hull / Leg Stiffness Sensitivity

From the results of stage 1a, it was concluded that the results are not sensitive to the leg-to-hull connection stiffness and that they show some sensitivity to the hull stiffness. This sensitivity is however low and not sufficient to have a significant impact on the levels of inferred fixity, at least for the more likely range of 'error' i.e. a factor of 2, rather than 5.

Note that the example unit is fitted with rack chocks. For a unit without rack chocks, the uncertainty in the leg-to-hull connection stiffness may be greater, however all cases we are considering are for units with rack chocks installed.

5.3.2 Stage 1b – Benefits from Foundation Stiffness Increases

The results of stage 1b show that in deeper water a benefit of 4-7% would accrue when all utilisation checks are considered. When, as is often the case, the additional displacements arising from preload exceedance are negligible the preload check can be ignored, and a benefit around 10% in UC may be expected. It was therefore concluded that the study was worth pursuing.

6 JACK UP MODELS

6.1 NOBLE DENTON 3-LEG MODEL

The global response of the unit was investigated using a Finite Element (FE) 3-leg model of the unit, as illustrated in Figure 6-1, on page 14.

The truss type legs were modelled as equivalent beam elements, whose properties accounted for 10% of the rack tooth. The leg-to-hull connection was modelled by springs at the level of the chock. Stiffness properties of these springs were derived by comparing results of unit loads applied to the clamped leg to those of the same loads applied to a detailed model of one leg, guides and holding system.

The hull was modelled by a set of equivalent beams representing the main longitudinal and transverse frames, leg wells and side shell. Half of the hull mass was added uniformly to the hull grillage beams and the remainder was divided into three lump masses at the leg centres. This has the effect of implicitly reducing the locked in hull-sag leg moments due to dead load to 50% of the theoretical maximum.

The footings were earthed via springs with stiffness properties derived for the level of spudcan fixity determined.

6.2 OXFORD UNIVERSITY "JAKUP" MODEL

Analytical models for the eight cases were constructed using the "JAKUP" program. The program has been developed at Oxford University, and is described in detail in the theses by Thompson (Ref. [5] and Cassidy (Ref [6]). The key features of this program are:

- The program uses the finite element method with dynamics modelled by time-stepping.
- The structure is modelled as a 2-dimensional "bar stool" model, with the legs and hull represented by elastic finite elements.
- Wave loading is modelled using either regular waves or (as in this project) pseudo-random waves using the "NewWave" method. It is also possible to model specific events using "Constrained New Wave", but this was not employed in this project. The wave kinematics is explicitly calculated, and the forces on the rig determined by Morison's method.
- Wind loading is specified by prescribed loads.
- Most importantly for this project, the foundation can be modelled as pinned, fixed, elastic springs, or using one of two advanced work-hardening plasticity models in terms of force resultants on the foundation.
- Hydrodynamic damping is introduced through Morison's method, and structural damping is specified using Rayleigh's method.

The most important limitations of the program are:

- Two-dimensional modelling is used.
- Detailed modelling of leg and hull is not possible.
- No account is taken of non-linearity at the leg-hull connection.

In spite of the above, the program is able to provide realistic modelling of jack-up movements and loads. It has been extensively used for research. In terms of capabilities on foundation modelling it probably represents the most sophisticated option available at present.

6.2.1 Structural Models

The structural models for each of the units were as specified by NDE. The details for each case are given in Appendix A.

6.2.2 Wave Loading

For each site one sea state was chosen as the most severe during the monitoring for each site. The direction of the sea was assumed to be coincident with the measured wind direction. An equivalent 2-D model was chosen (2 legs to windward or 2 legs to leeward) to be as close as possible to this direction. The Sea State was represented by a spectrum, with specified significant wave height, mean crossing period and peakedness parameter.

The assumption that the wind and wave are co-linear is not of significance here because, although the assumption may result in an overestimate of the mean response, this is not used as the mean has been filtered from the measured data. The consequence of modelling the jack-up with either 2 legs to windward or 2 legs to leeward is considered to be low given that the principal outputs required from the model are natural frequency and displacements.

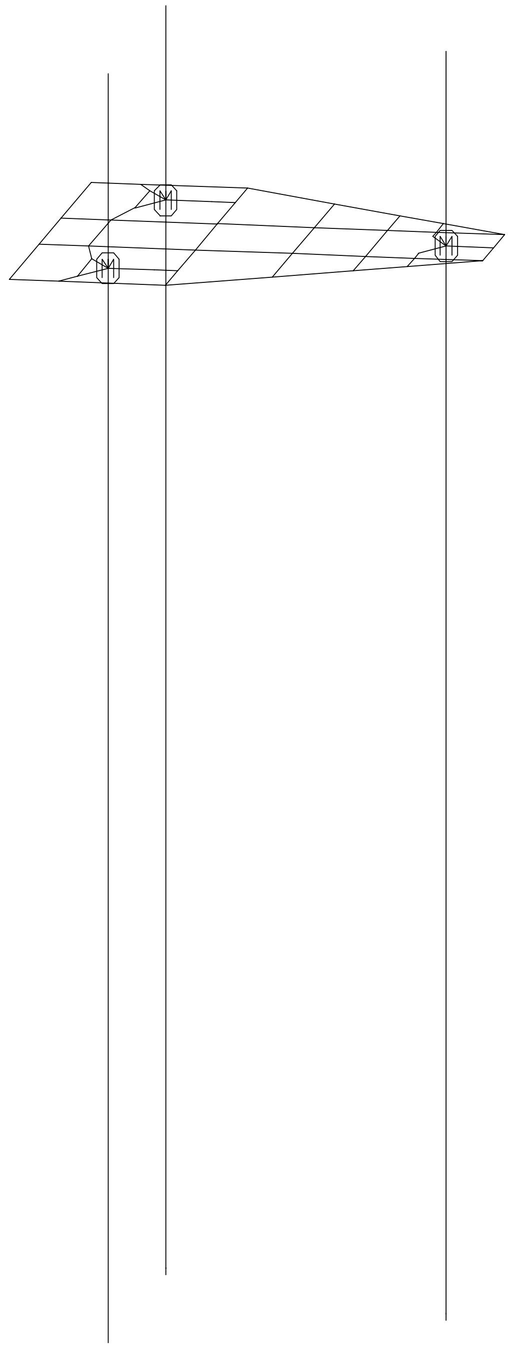
6.2.3 Foundation Models

The two work-hardening plasticity models used in JAKUP are:

- "Model B" for clay foundations. This is fully described in the thesis by Martin (Ref. [7]). The details are in chapter 6, which is reproduced here as Appendix B.
- "Model C" for sand foundations. This is fully described in the thesis by Cassidy (Ref. [6]. The details are in section 3.3, which is reproduced here as Appendix C.

The two models are closely related, and differ principally in the way the work-hardening is described and in the details of the flow rule. (Note that "Model A" was a preliminary model used for comparison purposes by Martin (Ref [7])).

Figure 6-1 : NDE 3-Leg Model



7 CALIBRATION

Each structural model was first subjected to a 10MN impulse load at deck level, to allow a comparison between the NDE program and the JAKUP analyses. Four cases were examined:

- pinned footings
- elastic springs (based on the inferred foundation stiffness from previous NDE studies)

In each case the jack-up was firstly considered in air (thus excluding hydrodynamic damping) and secondly in water (with damping and added mass effects). The results are presented as Figures 1 to 8 (in the Oxford Figures section, following the main body of the report).

For each case the mean (steady state) deck displacement is obtained. This is a direct measure of the overall stiffness of the system. The deck displacements in the two sets of analyses are compared in Table 1(a) (note that the steady state displacements are of course the same for analyses in air and water. In most cases the agreement is extremely good. Only in one case do the displacements differ by about 5%.

The natural period is deduced from the time interval between the first and seventh peaks. The periods for the analyses in air and water are very similar, and (as expected) those for pinned footings are longer than those for linear springs. The natural period is a function of stiffness and mass distribution. So (having confirmed the stiffness above) this represents a check on the mass distribution. The pinned natural periods from Figures 1 to 8 (in the Oxford Figures section, following the main body of the report) together with the fixed natural periods estimated from the results in Appendix D are presented in Table 1(b).

The damping factor is determined from the logarithmic decrement for the first six cycles. Since the damping factor depends on mass, stiffness and damping, this represents (having checked the mass and stiffness above) a check on the implementation of the damping. In each case it can be seen that the contribution of hydrodynamic damping is small. The overall damping factor (at about 5%) is higher than the specified structural damping of 4% because of the way Rayleigh damping implies damping at frequencies other than those at which the damping factor is specified.

8 PRELOADING

The unit was analysed by applying vertical loads up to and beyond the specified preload values to calibrate the chosen strength values. At the beginning of the main analysis the preload was applied, then the load reduced to the working load. The calibration procedure was as follows for the two models, and the resulting properties are given in Appendix A.

8.1 MODEL B

Initial estimates of the profile of undrained strength with depth were made on the basis of the site investigation information that had been provided to Noble Denton for the original site assessment. In each case the strength profile was fitted by a linear variation of strength with depth. Most emphasis was placed on the data for depths between the observed penetration and the observed penetration plus one footing radius. None of the sites represented a significant risk of "punch through" conditions.

The load-penetration curve was computed using JAKUP, and the profile of strength then fine-tuned to give a match of the observed penetration at preload. Only minor adjustments were necessary at this stage. The jack-up penetration process has effectively been used as a very large scale investigation of the strength of clay. The final penetration curves for the relevant three cases are shown in Figure 9.

8.2 MODEL C

For Model C the penetration depends almost entirely on the chosen angle of friction, with a minor influence of the effective unit weight (which was fixed on the basis of site investigation). The angle of friction was adjusted (downward) until the observed penetration was matched. The resulting values are quite low (in the range of 27^o to 30.5^o)[(1)], but are quite credible for silty material. Note that, since the penetration appears only to have been resolved to the nearest 0.3m, this process is rather approximate, and too much credibility should not be given to the friction values. The final penetration curves are shown for the relevant five cases in Figure 10. The subsequent behaviour of Model C depends principally on the ratio of the vertical load to preload and not the angle of friction itself.

Note:

(1) This appears to be consistent with SNAME [1]a, [1]b, which in Section 6.2.3, recommends that the friction angle determined in the laboratory be reduced 5 degrees in order to account for the observed load-penetration behaviour of large diameter spudcans. The site investigations for cases 3, 6 and 7 indicated angles of friction of 35 to 38 degrees. For case 4 the data indicates that the friction angle increases with depth to between 30 and 35 degrees at 1.2m below mudline. For case 5 there is no laboratory data to hand.

9 COMPARISONS WITH CASE HISTORIES

For each case analyses were carried out for one hour of the measured sea state, with a variety of different assumed stiffness properties at the foundation. The values of stiffness factors are recorded in Table 2.

The history of deck displacement was recorded, and processed in the following way.

Firstly the frequency spectrum for the response was obtained for comparison with the measured spectra. The spectrum was obtained by two means: a Fast Fourier transform, and by use of Autoregression Coefficients (Ref. [8]). The latter is a more recently developed method which allows a representation of the spectrum using fewer variables. By comparison with the FFT it gives a much smoother spectrum, but otherwise the spectra derived from the two methods are (except for some special cases) very similar. The spectra are usually characterised by two peaks – one at the dominant wave period, and another at the natural period of the structure (which is usually much shorter than the wave period). The main purpose was to match the natural period of the structure.

The figures giving the observed spectra for the eight cases, followed by all the analysis cases are given in Appendix D. The measured spectra are given on the first two pages in FFT form. The calculated spectra are then given (2 pages per case) showing the analyses of the pinned case, fixed case and all the stiffness values given in Table 2. There then follows two pages in which the observed spectra are compared with the analyses that give the closest results. In these figures "storm" refers to the measurements in the direction of the storm and "bow" refers to the bow direction. The "storm" observations are the most relevant.

While the spectra give (usually) good data on the frequency response, they are poor indicators of the magnitude of response. For this reason a measure of the magnitude of response was also necessary. The details are given in Table 3, where the range of observed displacements (in the storm direction) can be compared with the calculated values. In this table the values closest to the measured values are highlighted.

Note:

In Model B, the shear modulus is determined as $G = I_r.S_u.$, where I_r is a dimensionless stiffness factor and S_u is the shear strength of the clay.

In Model C, the shear modulus is determined from $G/p_a = g(V/Ap_a)^{0.5}$, where g is a dimensionless stiffness factor V is the spudcan vertical load in still water conditions, A the spudcan area and p_a atmospheric pressure.

9.1 CASE 1

This was modelled using Model B. The OCR of the clay at this site is in the region of 15 to 60. On the basis of the measured spectra (position of peak response) it is clear that $I_r \geq 250$ is required and values of 300 or 350 fit well, but since the fixed case gives only a fractionally different frequency (about 0.16 Hz), no upper bound can be placed on the stiffness factor. Examining the range of the displacement magnitudes, and also the Standard Deviation of the displacements suggests that in fact the higher values of stiffness cannot be justified, with 250 to300 fitting quite well. An appropriate value is $I_r = 300$.

9.2 CASE 2

This was modelled using Model B. The OCR at this site is in the region 5 to 15. On the basis of the measured spectra (position of peak response) it appears that $I_r \geq 600$ is required, with both 600 and 700 fitting quite well, but somewhat lower values would give a peak not far from the observed values of about 0.175 Hz. Since the fixed case again gives only a marginally higher frequency it is again difficult to place an upper bound on the stiffness factor. Examining the range of displacement magnitudes, and also the Standard Deviation of the displacements suggests a best fit with an I_r value of about 200 to 300. Taken together it is therefore considered that the best overall fit is achieved with $I_r = 350$.

9.3 CASE 3

This was modelled using Model C. The measured data show no clear peak in response other than near the dominant wave frequency. The calculated response only shows this type of behaviour for very low stiffness, say g = 100. What slight peak there is at about 0.18Hz can be fitted quite well with *g* of 500 or 1000. On the other hand, the magnitude of the displacements for the low stiffness values are significantly too large: these are best fitted by stiffness in the range g = 500 to 4000. No firm conclusion can therefore be drawn from this case about the relevant stiffness, but a value of 500 could be justified.

9.4 CASE 4

This was modelled using Model C. On the basis of the spectrum a value $g = 300$ clearly appears to be the best fit. On the basis of measured displacements the values $g = 400$ or 500 fit equally well. On balance a value of $g = 400$ would seem to be appropriate.

9.5 CASE 5

This was modelled using Model C. The measured data show a peak at about 0.32Hz (note the significantly higher frequency associated with the much shallower water at this site). The calculated spectra do not fit this feature very well, but *g* values in the range 500 to 2000 do provide some indication of a small peak in broadly the same frequency range. The displacement data are inconclusive - the extremes of the range are best fitted with a high stiffness, whilst the standard deviation is fitted well with a rather low stiffness. Sadly it has to be concluded that the match between analysis and data in this case is not good enough to be able to draw any conclusions about the stiffness.

9.6 CASE 6

This was modelled using Model C. The measured data show only a weak peak at about 0.18 Hz, with this frequency being quite well matched by the calculations with g = 400. The data on displacements are best fitted by the pinned case. Again for this case it has to be concluded that the analysis does not fit the pattern of the data sufficiently well to draw any firm conclusion.

Note: It is believed that the displacement results obtained from the measured data for this case are affected by noise in the measured acceleration data which, when double integrated to produce displacements, can result in significantly increased spectral area, particularly at low frequencies.

9.7 CASE 7

This was modelled using Model C. There is a sharply defined measured peak in the spectrum at about 0.18 Hz, with this being well matched by the calculations with g = 300. The displacement data suggests that g = 300 is appropriate.

9.8 CASE 8

This was modelled using Model B. The OCR at this site is in the region 10 to 20. The sharp peak at about 0.21Hz is quite well matched by the analysis with $I_r = 600$, taking the value of s_u as 80.0kPa, which was the value used in the analysis. The displacement data also suggest that high values of stiffness factor can be justified, so that the value of 600 is felt appropriate.

Close examination of the strength profile at this site reveals that there is a large jump in strength almost exactly at the level where the strength is evaluated for the stiffness calculation (0.15*D* below the reference point on the foundation). JAKUP is unable to deal with this sudden change of strength, and so in the calculation of strength for the penetration analysis emphasis was placed on the material above the transition. However, where soils are strongly layered it is thought appropriate that, rather than using the strength evaluated at 0.15*D* below reference level for the stiffness calculation, it is more appropriate to take the average of values from 0.0*D* to 0.3*D*. This recommendation is made as a rational extension of the single value, not on the basis of any rigorous set of calculations. When the strength is calculated in this way a value of 163kPa is obtained. Thus the stiffness used in the analysis corresponds to a value of $I_r = 294$, (calculated from 600 x 80 = 294 x 163). The final fitted value for Case 8 is therefore taken as 294.

9.9 CONCLUSIONS ON STIFFNESS FACTORS

9.9.1 Model B for clay

On the basis of laboratory testing a starting value of I_r = 80 has been assumed. Martin (ref. [7]) had found that I_r = 100 was satisfactory, although his tests were not particularly directed towards establishing stiffness at small displacements. Earlier laboratory data by de Santa Maria [11] had been best fitted by I_r = 31, although it should be noted that a different model was used in that case. The two case histories (1 and 2) where a clear indication of an appropriate stiffness value can be obtained both give significantly higher stiffness: giving I_r = 300 to 350. In a third case (Case 8) a value as high as 600 is suggested. The third case (Case 8) is rather harder to interpret, but a value of 294 is suggested. A “best” compromise figure for I_r in Model B, based on the field monitoring, would appear to be about 300. This seems quite a reasonable figure, and indeed the figures from the laboratory tests had appeared rather low.

The reasons for the fourfold difference between the laboratory and field values are thought to be related to the fact that:

a) The laboratory tests were all on very highly overconsolidated material, and the value of I_r reduces with overconsolidation ratio. It may be necessary in future to recommend a variation of the appropriate stiffness factor with OCR.

b) In the laboratory tests it was difficult to resolve the very small displacements which would be expected as a consequence of high stiffness, hence the initial stiffnesses may have been inherently inaccurate.

c) The laboratory tests are at approximately 1/100th of full scale, and some scaling effects are inevitably to be expected.

There is no reason (on the basis of the work done here, which was all on moderately overconsolidated clays) to alter the recommendations on relative stiffness currently in SNAME T&RB 5-5A (Refs. [1]a, [1]b) for different OCR's, although there would be some merit in altering the present step changes (by a factor of 2 in each case) to a smooth variation. A comment should, however, be included to the effect that the variation with OCR should not be regarded as having any degree of precision. Adopting a slightly conservative approach to the above data we consider that a factor of approximately 3.0 higher on the recommended stiffness can be justified. Figure 11 shows a plot of the previous recommended stiffness factors against OCR, the data from the case histories and the new recommendation (see below). Note, however, the lack of data at both very low and very high OCR values.

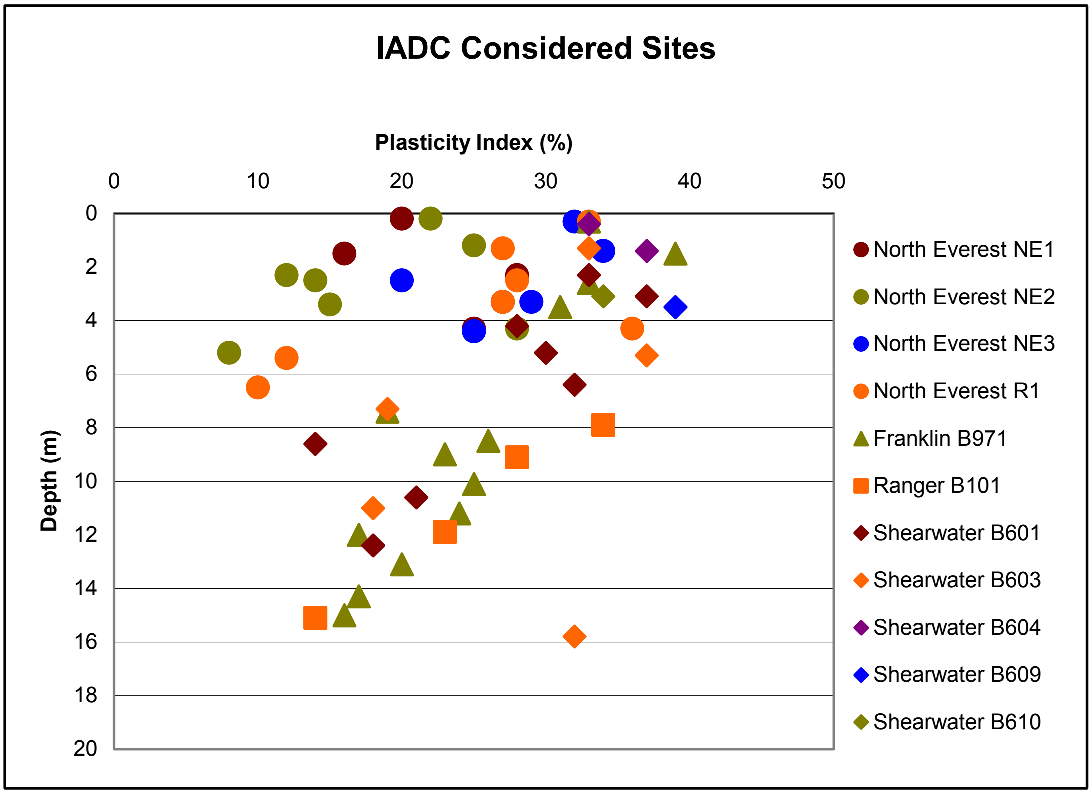
Recommendation: The value of the shear modulus for clay should be based on the value of the shear strength measured at the depth $z = D + 0.15B$, where B is the diameter of the spudcan and D is the depth below mudline of the lowest point on the spudcan at which this diameter is attained. Where the clay is significantly layered the average strength within the range $z = D$ to $z = D + 0.3B$ should be used. The shear modulus should be calculated as $G / s_u = 600 / OCR^{0.25}$.

Following Industry comment, and noting the lack of data at low OCR, it is proposed that this relationship be subject the limitation that G / s_u should generally not exceed 400. Figure 9-1 shows a plot of the plasticity indices, (Ip), against depth at the measurement sites. Noting that the shear modulus may be influenced by the Plasticity Index caution is recommended should the site-specific Ip significantly exceed that of the measurement sites, 40%.

However:

- For Normally Consolidated clays Andersen's data supports the use of G / s_u of 400 up to about Ip = 60% if, as suggested by Andersen in correspondence, the low points at Ip of around 50% are given less weight as they fall outside the main trend;
- Field data reported by Templeton [12] supports G / s_u = 600 for jack-up response in the Gulf of Mexico clays with low OCR and Ip less than about 40%;
- Data presented by Andersen in Figure 10.2 of [13], reproduced in Figure 9-2, shows that G / s_u appears to be fundamentally inversely proportional to Ip and supports values of G / s_u as high as 1,000 or even 2,500, particularly for cases where Ip is less than 20% and OCR is low.

Figure 9-1 : Plasticity Indices at the Measurement Sites

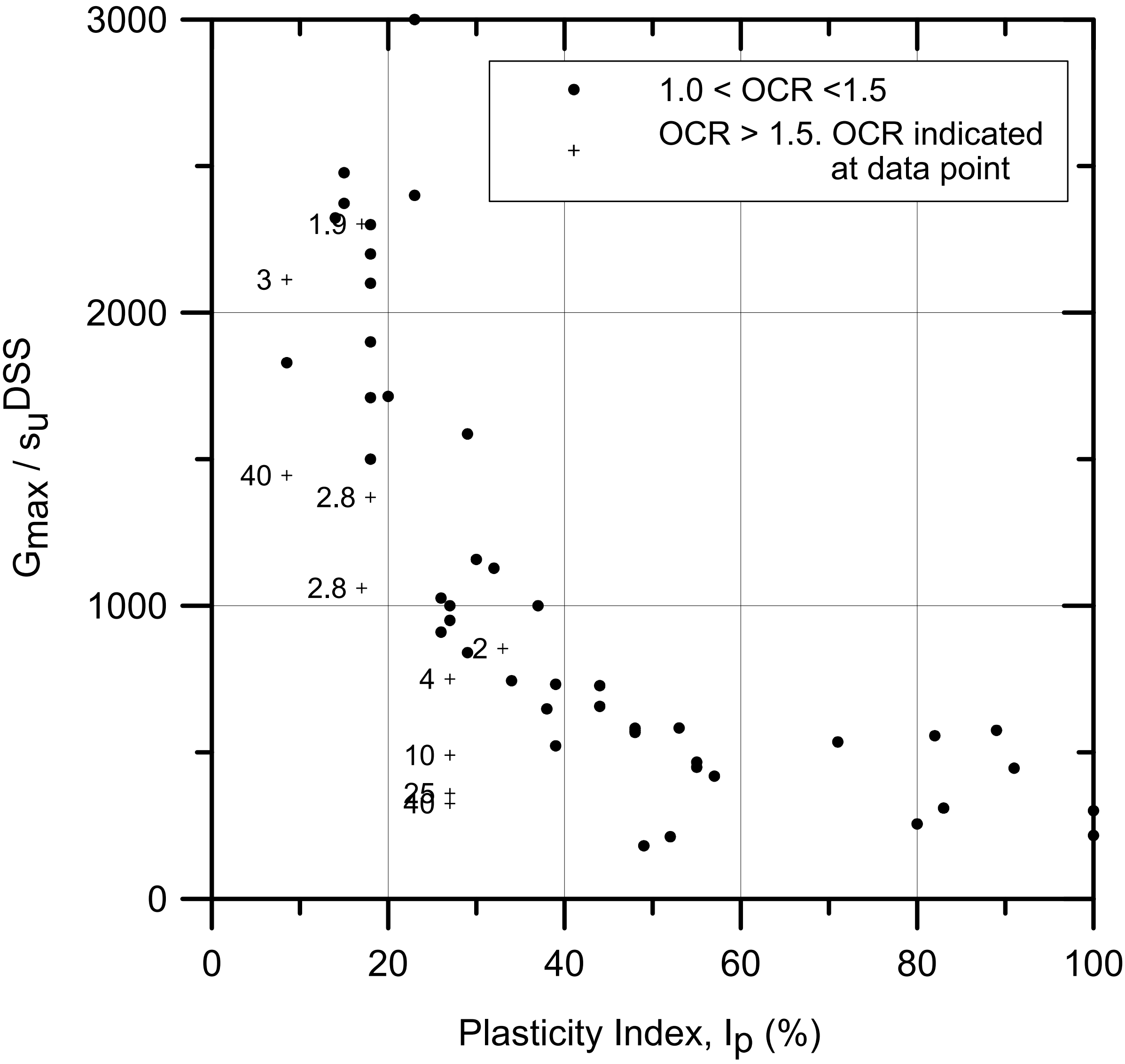


Comments: The above implies that $G = s_u \dfrac{600}{OCR^{0.25}}$, $K_\theta = \dfrac{GB^3}{1.5}$ (assuming Poisson's ratio of 0.5 for clay) and that, ignoring other terms and factors, $V_{Lo} = \dfrac{\pi B^2}{4} N_c s_u$ where V_{Lo} is the effective preload. This simplifies to:

$$K_3 = V_{Lo} B \left(\frac{4 \times 600}{1.5 \pi N_c} \right) \frac{1}{OCR^{0.25}}$$

So that the rotational stiffness is directly proportional to the diameter, directly proportional to the preload, and depends weakly on the OCR. The bracketed term is almost a constant factor in the region of about 80. Since full embedment will usually apply, so that neither preload nor diameter will vary very much for any one unit, in fact the OCR is the only factor that alters the stiffness significantly.

Figure 9-2 : Normalised initial shear modulus as a function of Plasticity Index, Ip, for 11 different clays. Figure 10.2 from Anderson [55]



Note: On the vertical axis G_{max}, the initial shear modulus from shear waves generated by bender elements in direct simple shear (DSS) tests is normalised against S_u^{DSS}, the undrained montonic shear strength from the DSS tests. Whilst these parameters may differ from those determined by other means, due to rate effects, etc., the differences are expected to be sufficiently small that the data and trends remain applicable.

9.9.2 Model C for sand

On the basis of laboratory tests, the estimated initial value of the dimensionless stiffness factor g was 4000. The three case histories which give a clear indication of suitable value of g for the field (Cases 3, 4 and 7) give values of about 500, 400 and 300 i.e. a factor of about 10 lower on average. Two cases (5 and 6) are inconclusive, but not entirely inconsistent with a value of 400. The reasons for the discrepancy are thought to be:

a) In the laboratory tests it was difficult to resolve the very small displacements which would be expected as a consequence of high stiffness, hence the initial stiffnesses may have been inherently inaccurate.

b) The laboratory tests are approximately 1/40th of full scale. The appropriate shear modulus is assumed to vary with the square root of the mean stress level, which is in turn given by a representative value 2Rγ′. The result is that the modulus is proportional to the square root of the radius. An alternative procedure might take the representative stress level as V/A = V/πR^2. For the laboratory the typical values of V are 1600N, and R=50mm, giving a stress level of about 200kPa. In the field the (effective) radius is typically about 4m and V about 40MN, giving a stress level of about 800kPa. Thus the factor of 40 on radius is related to a factor of only 4 on stress level, and therefore about 2 on stiffness. In contrast the original scaling would imply a factor of 40 on stress level and therefore a factor of about 6.3 on stiffness. The scaling by vertical load would therefore imply stiffnesses rather more than 3 times lower than expected from the laboratory tests, this implies that stress level should be scaled by use of the vertical load rather than 2Rγ′. (The reason why these give different values in this case is that the bearing capacity factor for the field cases was much lower than for the laboratory).

Because of the above it is felt best that the scaling of the stiffness should be on the basis of the mean stress under the spudcan, rather than the scaling on density and diameter. The increase of stiffness with the square root of stress is suggested.

No justification was seen for continuing the practice of using different stiffness values for vertical, horizontal and moment stiffness. The calibration herein was therefore undertaken on the basis that the moment stiffness is the dominant component and the same G value was then used for determining moment, lateral and vertical stiffnesses.

All the above case histories are on medium dense through to very dense sands, although there is little quantitative evidence about precise densities at different locations. The shear stiffness of soils is in fact rather insensitive to changes of density. SNAME T&RB 5-5A (Refs. [1]a, [1]b) suggests an adjustment factor based on voids ratio: $\frac{(2.973-e)^2}{(1+e)}$.

Which is a variation on the factor $\frac{(2.17-e)^2}{(1+e)}$ introduced by Hardin and Black [9]. (For a discussion of stiffness factors for sands see Wroth *et al.* [10].) An alternative, which is in many cases more satisfactory, is to base the adjustment on the relative density rather than on the voids ratio. The suggested factor is $1.23\left(0.9+\frac{D_R}{500}\right)$ which varies from 1.23x0.9 at a relative density of 0% to 1.23x1.1 at a relative density of 100%, suggesting that the effect of the density is only to change the stiffness by a maximum of 20%. The factor of 1.23 arises simply so that the two factors, based on voids ratio or relative density, are equivalent.

Recommendation: For sands the shear modulus should be taken as the same for the computation of all stiffnesses (vertical, horizontal and moment). The stiffness should be computed from the formula $G/p_a = g(V_{swl}/Ap_a)^{0.5}$, where V_{swl} is the spudcan vertical load in still water conditions, A the spudcan area and p_a atmospheric pressure. The recommended value of g is 250 for the case histories considered. Estimating that the average relative density at these sites was say 80% or more, the suggested final form is:

$$g = 230\left(0.9 + \frac{D_R}{500}\right).$$

Comments: The above gives $\frac{G}{p_a} = 230\left(0.9 + \frac{D_R}{500}\right)\left(\frac{4V_{swl}}{\pi B^2 p_a}\right)^{0.5}$.

Combining this with $K_3 = \frac{GB^3}{3(1-\nu)}$ and $V_{Lo} = \frac{\pi B^2}{4}\frac{\gamma' B}{2}N_\gamma$ (for a partially embedded foundation in sand, and also approximately true for the fully embedded case if one ignores the N_q term, and depth and shape factors) gives

$$K_3 = \frac{2 \times 230\left(0.9 + \frac{D_R}{500}\right)p_a^{0.5}V_{swl}^{0.5}}{3(1-\nu)\pi^{0.5}}B^2 \text{ and } B^3 = \frac{8V_{Lo}}{\pi\gamma' N_\gamma}.$$

Two cases emerge. If there is (rarely) full embedment then the rotational stiffness is proportional to the square of the diameter and the square root of the load - since for any particular unit not much can be done about either, this results in almost constant rotational stiffness in the embedded case.

In the partially embedded case we substitute for the diameter and get:

$$K_\theta = \frac{2 \times 230\left(0.9 + \frac{D_R}{500}\right)p_a^{0.5}V_{swl}^{1.17}}{3(1-\nu)\pi^{0.5}}\left(\frac{V_{Lo}}{V_{swl}}\right)^{0.67}\left(\frac{8}{\pi}\right)^{0.67}\frac{1}{\gamma'^{0.67}}\frac{1}{N_\gamma^{0.67}}$$

This shows that the stiffness depends on the vertical load (increasing slightly more than linearly) and <u>reduces</u> with increasing bearing capacity factor. Note that of course N_γ increases much more rapidly than D_R as relative density increases. The rather surprising effect of density is due to the reduced penetration and hence reduced effective diameter. In the limit an infinitely strong soil would result in point contact, and no rotational stiffness! None of the other factors in the above equation vary much.

The overall story is that weaker soils (NC rather than OC clays, loose rather than dense sands) in each case result in, paradoxically, higher rotational stiffnesses.

9.10 OTHER RECOMMENDATIONS

It is recommended that the formulation of the yield surface in SNAME be updated. The reasoning for this proposal is as follows:

Writing $v = V/V_{Lo}$, $m = M/M_{Lo}$, $h = H/H_{Lo}$ the original form of yield locus for SNAME was:

$$f = 16v^2(1-v)^2 - h^2 - m^2 = 0$$

This was modified in to:

$$f = 16v^2(1-v)|1-v| - h^2 - m^2 = 0$$

to account for difficulties for $v > 1$ (hypothetical cases which arise during computation). There are three problems with the above:

a) in solving the problem on $v > 1$ in fact it causes some problems elsewhere,
b) it uses the opposite convention from the usual one that the yield function should be negative within the yield surface,
c) the expression is somewhat cumbersome.

These problems are all resolved by replacing the surface by

$$f = \sqrt{h^2 + m^2} - 4v(1 - v) = 0$$

which is simpler, negative within the surface and entirely positive outside. The positive square root must always be taken. The modified expression also has the property that its differentials with respect to v, m and h are better behaved for special cases.

9.11 EVALUATION OF STIFFNESS RECOMMENDATIONS

In Table 9-2 overleaf, the stiffness values resulting from the proposed formulations are given and compared with stiffnesses based on SNAME. It can be seen that the rotational stiffness has increased by a factor of at least 2, in all cases except those for Clay, when the lowest OCR is used. The vertical stiffnesses increase by a factor of at least 2 for clay foundations, but reduce slightly for sand foundations. The horizontal stiffnesses increase by a factor of at least 2 for clay foundations and approaching 10 for sand foundations.

Using the Noble Denton JUSTAS program, the benefits of the increased foundation fixity from the proposed 'calibrated' formulations were evaluated. The cases considered are those that were previously selected and reported in Section 5. The results are shown in Table 9-1 below, where key parameters and the resulting utilisation checks from the proposed formulation are compared against those obtained from SNAME '97. The information is presented in terms of the ratio of the 'calibrated' case to the SNAME '97 case i.e. 'calibrated/SNAME '97.

Table 9-1 : Benefits from Foundation Stiffness Increases

	Monitor at Joule	Magellan at North Everest		Magellan at Franklin
Site Details:				
Water depth (m)	28.0	88.5		91.2
Airgap (m)	18.6	19.0		23.5
Soil type	Sand	Clay (1)	Clay (2)	Clay on Sand (3)
Ratios of Key Parameters:				
Foundation Stiffness	2.33	1.92	2.52	3.34
Natural period	0.90	0.90	0.86	0.83
Hull side-sway	0.88	0.96	0.95	0.89
Ratios of Unity Checks:				
Preload UC	0.99	0.98	0.97	0.94
OTM UC	0.98	0.95	0.93	0.90
Leg Strength UC	0.97	0.97	0.95	0.92
Percentage Gains In UC:				
Including preload check	1%	2%	3%	6%
Ignoring preload check	2-3%	3-5%	5-7%	8-10%

(1) For Lower bound OCR values. (2) For Upper bound OCR values. (3) Lower bound clay selected.

Table 9-2 : Effect of Proposed Foundation Stiffness Formulations

	Magellan at Franklin			Magellan at North Everest		Magellan at Joanne	Monitor at Halley	Monitor at Joule	Galaxy at Ranger	Galaxy at Judy	Galaxy at Shearwater	
Water depth (m)	91.2			88.5		77.0	83.5	28.0	91.8	74.5	89.4	
Airgap (m)	23.5			19.0		21.0	19.8	18.6	19.8	20.1	19.2	
Soil type	Clay on Sand			Clay		Sand	Sand	Sand	Sand	Sand	Clay	
Original Foundation Stiffnesses to RP '97												
Vertical Stiffness (t/m)	0.071×10^{6} [3]			0.067×10^{6}		0.170×10^{6}	0.168×10^{6}	0.171×10^{6}	0.213×10^{6}	0.221×10^{6}	0.186×10^{6}	
Horizontal Stiffness (t/m)	0.033×10^{6} [3]			0.045×10^{6}		0.016×10^{6}	0.016×10^{6}	0.016×10^{6}	0.020×10^{6}	0.021×10^{6}	0.124×10^{6}	
Rotational Stiffness (tm/rad)	2.926×10^{6} [3]			3.680×10^{6}		2.279×10^{6}	2.258×10^{6}	1.900×10^{6}	3.551×10^{6}	1.924×10^{6}	10.19×10^{6}	
Proposed Foundation Stiffnesses:												
C_u (kPa)/ OCR / RD	98.9 / 15 - 60 / 80%			56.6 / 5 - 15 / -		- / - / 80%	- / - / 80%	- / - / 80%	- / - / 80%	- / - / 80%	163 / 10 – 20 / -	
	Clay (1)	Clay (2)	Sand	Clay (1)	Clay (2)	Sand	Sand	Sand	Sand	Sand	Clay (1)	Clay (2)
Vertical Stiffness (t/m)	0.224×10^{6}	0.158×10^{6}	0.163×10^{6}	0.168×10^{6}	0.128×10^{6}	0.164×10^{6}	0.158×10^{6}	0.160×10^{6}	0.195×10^{6}	0.195×10^{6}	0.408×10^{6}	0.343×10^{6}
Horizontal Stiffness (t/m)	0.149×10^{6}	0.105×10^{6}	0.154×10^{6}	0.112×10^{6}	0.085×10^{6}	0.156×10^{6}	0.150×10^{6}	0.152×10^{6}	0.185×10^{6}	0.185×10^{6}	0.272×10^{6}	0.229×10^{6}
Rotational Stiffness (tm/rad)	12.3×10^{6}	8.71×10^{6}	8.96×10^{6}	9.27×10^{6}	7.04×10^{6}	5.53×10^{6}	5.30×10^{6}	4.43×10^{6}	8.10×10^{6}	4.25×10^{6}	22.5×10^{6}	18.9×10^{6}
Ratios of RP '97 Stiffnesses to Proposed Stiffnesses:												
Vertical Stiffness	3.14	2.22	2.29	2.52	1.92	0.97	0.94	0.93	0.91	0.88	2.20	1.85
Horizontal Stiffness	4.48	3.17	4.63	2.52	1.92	9.71	9.39	9.32	9.15	8.87	2.20	1.85
Rotational Stiffness	4.21	2.98	3.07	2.52	1.92	2.43	2.35	2.33	2.28	2.21	2.20	1.85

(1) For Lower bound (smaller) OCR values.
(2) For Upper bound (larger) OCR values.
(3) The original assessment used the sand Gr value for both rotational and vertical stiffness and the a clay G value for horizontal stiffness

10 RECOMMENDATIONS FOR FUTURE WORK

Although the present project has gone some way to clarifying the uncertainties about the appropriate stiffness for the analysis of jack-ups, there remain a number of issues to be resolved, and further work in this area would be valuable. In some cases further research would probably lead to long-term economies, as the removal of uncertainty removes the need for over-conservative procedures. However, it should also be borne in mind that there is the possibility that research might show that present practice is insufficiently conservative.

Specific items (in no particular order) where future work is required are:

a) **Further monitoring and back analysis of field cases**.
To be of value the information at the sites should include sufficient site investigation data so that the soils can be properly characterised, adequate knowledge of the structural properties of the rig (presumably not a problem), monitoring of environmental conditions (wind, wave and current) and monitoring of deck movements. Additional information, such as monitoring of forces in selected leg members, would clearly be of enormous value, but would probably be difficult to achieve. Some direct indication of stiffness, *e.g.* the measurement of soil rebound as the preload is dumped, would also be a valuable check, but again is difficult to obtain. It would be better to devote limited resources to a relatively small number of well-monitored sites than to attempt to cover too many sites and compromise on the quality of the data.

b) **Development of theoretical models**.
The models used here for the analysis of jack-ups (Models B and C) are probably the most advanced currently available, but they are inadequate as far as the modelling of cyclic behaviour is concerned. It is proposed to develop a new family of models which would be based on the recently developed “continuous hyperplasticity” approach. This has been shown to fit the behaviour of several aspects of model tests very well. The advantage is that it provides realistic modelling of behaviour during cycling, including a gradual degradation of stiffness with strain amplitude.

c) **Validation of models against laboratory tests**.
Models B and C were based on small scale laboratory testing of single footings. Whilst validation against field records is of primary importance, a greater coverage of different cases (especially extreme events) can be achieved by validation against further, but independent, model tests. It is proposed to carry out tests at an intermediate scale, with spudcans of the order of 300mm-450mm diameter. A number of possibilities exist, e.g. (i) applying loads to the spudcans calculated on the basis of applied wave loading on the legs of a rig, with the rig itself simulated numerically, or (ii) carrying out tests on a full model of rig and footings. In either case we would propose using the recently installed Structural Dynamics Laboratory at Oxford University. This has a suite of state-of-the-art computer controlled actuators which can apply loads up to about 500kN at a frequency up to about 10Hz.

11 CONCLUSIONS

11.1 Initial studies showed that, for the jack-up units studied here, the foundation stiffness inferred from back-analysis using FE models will not be unduly dependent upon the assumptions made in respect of hull or leg-hull connection stiffness. Furthermore, the anticipated increase in foundation stiffness would produce modest, but useful benefits.

11.2 Eight case records of jack-ups in the North Sea have been back analysed to fit soil stiffness parameters. Six of the eight cases provide satisfactory data.

11.3 On the basis of these records, it is possible to recommend higher stiffness factors than are currently suggested in SNAME T&RB 5-5A (Ref. [1]a, [1]b). Such higher stiffness factors had been expected to be appropriate by some practitioners, but the case records studied here provide a firmer basis than had hitherto been available for recommending higher stiffness factors.

11.4 In clays the value of the shear modulus should be based on the value of the shear strength measured at the depth $z = D + 0.15B$. Where the clay is significantly layered, the average strength within the range $z = D$ to $z = D + 0.3B$ should be used, where B is the diameter of the spudcan and D is the depth below mudline of the lowest point on the spudcan at which this diameter is attained. Except in areas with carbonate clays or clayey silts the shear modulus should be calculated as:

$$G / s_u = 600 / OCR^{0.25} ,$$

with G / s_u generally < 400 for clays with Plasticity Indices up to 60%. Due consideration should be given to the possibility of determining site-specific shear moduli for normally consolidated and slightly overconsolidated clays and/or where the Plasticity Indices exceed 60%. G / s_u = 600 is supported by field data for jack-up response in the Gulf of Mexico (Templeton [12]) on clays with low OCR and Plasticity Index no more than about 40%. For other cases when field data is not available and the Plasticity Indices are in excess of 60%, account should be taken of the inverse dependence of G / s_u on Plasticity Index, Andersen [13]. In some cases higher ratios of G / s_u may be applicable; the data published by Andersen in figure 10.2 of [13], reproduced in Figure 9-2, would support use of values as high as 1000 or even 2500, particularly for Plasticity Indices less than 20%.

11.5 In sands the shear modulus should be taken as the same for the computation of all stiffnesses (vertical, horizontal and moment). The stiffness should be computed from the formula $G / p_a = g \left(V_{swl} / A p_a \right)^{0.5}$, where V_{swl} is the spudcan vertical load in still water conditions, A the spudcan area and p_a atmospheric pressure. The recommended value of g is: $g = 230 \left(0.9 + \frac{D_R}{500} \right)$.

11.6 Proposals for incorporating the recommendations in SNAME 2002, (ref. [1]b) are shown in Appendix E, which also includes a revised presentation of the equation for the yield surface.

11.7 The recommendations result in increased rotational stiffness, by factors of about 2 or more. Example site assessments in deeper water using the increased stiffnesses result in utilisation checks improved by about 2-6%, or 3-9% if the preload check is ignored.

11.8 Recommendations for future monitoring and back-analysis work are made.

NOBLE DENTON EUROPE LTD.

Signed: ______________________

Eur Ing M.J.R. Hoyle, CEng, MA, MRINA, MIMarE

Dated : London, 21st November, 2005

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OXFORD TABLES FOR SECTIONS 8 & 9

Table 1(a): Comparison of mean calculated offsets in NDE and Oxford (JAKUP) analyses

Comparison of mean offsets (mm)							
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7
Pinned footings							
NDE	2.037	1.703	1.270	1.430	0.244	0.794	0.545
JAKUP	2.078	1.720	1.287	1.436	0.231	0.784	0.561
% difference	1.987	1.008	1.340	0.428	-5.845	-1.276	2.800
Linear elastic springs							
NDE	0.684	0.901	0.779	0.683	0.156	0.400	0.381
JAKUP	0.681	0.901	0.772	0.675	0.148	0.391	0.391
% difference	-0.352	0.069	-0.887	-1.211	-5.421	-2.354	2.458

Table 1(b): Comparison of mean calculated offsets in NDE and Oxford (JAKUP) analyses

Natural Periods (sec)								
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8
Pinned T_N [(1)]	11.3	10.2	9.00	8.73	3.95	7.88	6.55	7.93
Fixed T_N [(2)]	5.7	5.45	4.90	4.88	3.03	4.35	3.64	4.35

[(1)] From Figures 1 to 8
[(2)] Estimated from computed results in Appendix D

Table 2: Stiffness Values for Analyses

Case 1

	I_r	D	s_u @ z = 0.15D	G	K_v	K_m	K_h	K_c
		(m)	(kPa)	(MPa)	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
Ir = 80	80	18.2	98.9	7.9	580.9	32434.3	399.2	-27.6
Ir = 150	150	18.2	98.9	14.8	1089.1	60814.3	748.4	-51.7
Ir = 250	250	18.2	98.9	24.7	1815.2	101357.2	1247.4	-86.2
Ir = 300	300	18.2	98.9	29.7	2178.2	121628.6	1496.9	-103.5
Ir = 350	350	18.2	98.9	34.6	2541.3	141900.0	1746.4	-120.7
Fully Fixed					infinity	infinity	infinity	N.A.

Case 2

	I_r	D	s_u @ z = 0.15D	G	K_v	K_m	K_h	K_c
		(m)	(kPa)	(MPa)	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
Ir = 80	80	18.2	56.6	4.5	335.6	18997.0	237.5	-31.6
Ir = 150	150	18.2	56.6	8.5	629.2	35619.4	445.3	-59.2
Ir = 200	200	18.2	56.6	11.3	838.9	47492.5	593.8	-78.9
Ir = 300	300	18.2	56.6	17.0	1258.4	71238.8	890.7	-118.4
Ir = 350	350	18.2	56.6	19.8	1468.1	83111.9	1039.1	-138.1
Ir = 400	400	18.2	56.6	22.6	1677.9	94985.1	1187.5	-157.9
Ir = 500	500	18.2	56.6	28.3	2097.3	118731.3	1484.4	-197.3
Ir = 600	600	18.2	56.6	34.0	2516.8	142477.6	1781.3	-236.8
Ir = 700	700	18.2	56.6	39.6	2936.3	166223.9	2078.2	-276.3
Fully Fixed					infinity	infinity	infinity	N.A.

Case 3

	g	R	unit weight γ'	G	K_v	K_m	K_h	K_c
		(m)	kN/m^3	MN/m^2	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
g = 100	100	7.3	9.1	11.6	448.8	16608.0	389.6	-346.2
g = 200	200	7.3	9.1	23.2	897.7	33216.1	779.1	-692.4
g = 300	300	7.3	9.1	34.8	1346.5	49824.1	1168.7	-1038.6
g = 400	400	7.3	9.1	46.4	1795.4	66432.1	1558.3	-1384.8
g = 500	500	7.3	9.1	58.0	2244.2	83040.2	1947.8	-1731.0
g = 1000	1000	7.3	9.1	116.0	4488.5	166080.3	3895.7	-3462.1
g = 4000	4000	7.3	9.1	464.0	17954.0	664321.4	15582.7	-13848.3
Fully Fixed					infinity	infinity	infinity	N.A.

Case 4

	g	R	unit weight γ'	G	K_v	K_m	K_h	K_c
		(m)	kN/m^3	MN/m^2	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
g = 100	100	7.3	9.0	11.5	446.4	16516.5	387.4	-344.3
g = 200	200	7.3	9.0	23.1	892.8	33033.1	774.8	-688.6
g = 300	300	7.3	9.0	34.6	1339.1	49549.6	1162.3	-1032.9
g = 400	400	7.3	9.0	46.1	1785.5	66066.1	1549.7	-1377.2
g = 500	500	7.3	9.0	57.7	2231.9	82582.6	1937.1	-1721.5
g = 1000	1000	7.3	9.0	115.4	4463.8	165165.3	3874.2	-3443.0
g = 2000	2000	7.3	9.0	230.7	8927.5	330330.6	7748.4	-6886.0
g = 3000	3000	7.3	9.0	346.1	13391.3	495495.9	11622.6	-10329.0
g = 4000	4000	7.3	9.0	461.5	17855.1	660661.2	15496.8	-13772.0
Fully Fixed					infinity	infinity	infinity	N.A.

Table 2: Stiffness Values for Analyses (Continued)

Case 5

	g	R	unit weight γ'	G	K_v	K_m	K_h	K_c
		(m)	kN/m^3	MN/m^2	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
g = 100	100	6.5	9.1	10.9	377.1	11063.3	327.3	-259.0
g = 200	200	6.5	9.1	21.9	754.3	22126.6	654.6	-518.0
g= 300	300	6.5	9.1	32.8	1131.4	33189.9	982.0	-777.0
g = 400	400	6.5	9.1	43.8	1508.5	44253.2	1309.3	-1036.0
g = 500	500	6.5	9.1	54.7	1885.6	55316.5	1636.6	-1295.0
g = 1000	1000	6.5	9.1	109.5	3771.3	110633.1	3273.2	-2590.1
g = 2000	2000	6.5	9.1	218.9	7542.5	221266.2	6546.3	-5180.1
g = 3000	3000	6.5	9.1	328.4	11313.8	331899.2	9819.5	-7770.2
g = 4000	4000	6.5	9.1	437.9	15085.0	442532.3	13092.7	-10360.3
Fully Fixed					infinity	infinity	infinity	N.A.

Case 6

	g	R	unit weight γ'	G	K_v	K_m	K_h	K_c
		(m)	kN/m^3	MN/m^2	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
g = 100	100	7.89	9.5	12.3	515.3	22274.0	447.3	-429.6
g = 200	200	7.89	9.5	24.6	1030.6	44548.1	894.5	-859.2
g = 300	300	7.89	9.5	37.0	1545.9	66822.1	1341.8	-1288.8
g = 400	400	7.89	9.5	49.3	2061.3	89096.1	1789.0	-1718.4
g = 500	500	7.89	9.5	61.6	2576.6	111370.2	2236.3	-2148.0
g = 1000	1000	7.89	9.5	123.2	5153.2	222740.3	4472.5	-4296.0
g = 2000	2000	7.89	9.5	246.5	10306.3	445480.7	8945.1	-8592.0
g = 3000	3000	7.89	9.5	369.7	15459.5	668221.0	13417.6	-12887.9
g = 4000	4000	7.89	9.5	492.9	20612.6	890961.3	17890.2	-17183.9
Fully Fixed					infinity	infinity	infinity	N.A.

Case 7

	g	R	unit weight γ'	G	K_v	K_m	K_h	K_c
		(m)	kN/m^3	MN/m^2	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
g = 100	100	6.8	9.3	11.3	407.9	13097.6	354.1	-293.1
g = 200	200	6.8	9.3	22.6	815.9	26195.1	708.1	-586.2
g = 300	300	6.8	9.3	34.0	1223.8	39292.7	1062.2	-879.3
g = 400	400	6.8	9.3	45.3	1631.8	52390.2	1416.3	-1172.4
g = 500	500	6.8	9.3	56.6	2039.7	65487.8	1770.3	-1465.5
g = 1000	1000	6.8	9.3	113.2	4079.4	130975.5	3540.6	-2931.0
g = 2000	2000	6.8	9.3	226.4	8158.9	261951.1	7081.3	-5862.1
g = 3000	3000	6.8	9.3	339.6	12238.3	392926.6	10621.9	-8793.1
g = 4000	4000	6.8	9.3	452.8	16317.8	523902.2	14162.6	-11724.2
Fully Fixed					infinity	infinity	infinity	N.A.

Case 8

	I_r	D	s_u @ z = 0.15D	G	K_v	K_m	K_h	K_c
		(m)	(kPa)	(MPa)	(MN/m)	(MNm/rad)	(MN/m)	(MN/m)
Pinned					infinity	0	infinity	N.A.
Ir = 80	80	18.2	80.0	6.4	492.8	29412.9	389.1	-137.6
Ir = 150	150	18.2	80.0	12.0	924.1	55149.1	729.5	-258.0
Ir = 200	200	18.2	80.0	16.0	1232.1	73532.2	972.7	-344.0
Ir = 300	300	18.2	80.0	24.0	1848.2	110298.2	1459.1	-516.0
Ir = 350	350	18.2	80.0	28.0	2156.2	128681.3	1702.2	-602.0
Ir = 400	400	18.2	80.0	32.0	2464.2	147064.3	1945.4	-688.0
Ir = 500	500	18.2	80.0	40.0	3080.3	183830.4	2431.8	-860.1
Ir = 600	600	18.2	80.0	48.0	3696.3	220596.5	2918.1	-1032.1
Fully Fixed					infinity	infinity	infinity	N.A.

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Table 3: Displacement Results

Case 1

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-63.1	56.2	119.3	13.9
Pinned	27.2	-308.9	333.5	642.4	97.7
Ir = 80	-25.9	-93.9	113.1	207.0	25.1
Ir = 150	-10.3	-57.9	74.2	132.1	14.9
Ir = 250	-4.3	-49.4	81.1	130.5	12.0
Ir = 300	-2.9	-56.4	83.3	139.7	12.2
Ir = 350	-1.8	-46.1	66.4	112.5	11.1
Fully Fixed	3.0	-35.4	43.1	78.5	8.9

Case 2

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-140.1	169.0	309.1	22.5
Pinned	317.7	-429.4	453.4	882.8	121.5
Ir = 80	245.4	-269.2	283.1	552.3	67.7
Ir = 150	176.1	-175.8	175.2	351.0	34.6
Ir = 200	155.1	-154.9	149.7	304.6	27.7
Ir = 300	133.2	-133.0	124.3	257.3	22.4
Ir = 350	126.7	-126.5	117.2	243.7	21.1
Ir = 400	121.8	-121.6	111.8	233.4	20.1
Ir = 500	114.8	-114.7	102.9	217.6	18.8
Ir = 600	110.1	-110.0	97.5	207.5	18.4
Ir = 700	106.7	-106.6	94.2	200.8	17.7
Fully Fixed	85.4	-85.2	73.8	159.0	12.9

Case 3

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-91.2	66.3	157.5	19.9
Pinned	305.4	-367.0	416.2	783.2	94.6
g = 100	204.6	-218.3	233.7	452.0	46.4
g = 200	171.6	-183.3	158.2	341.5	29.6
g = 300	159.4	-170.4	126.5	296.9	25.1
g = 400	151.6	-162.4	109.9	272.3	22.4
g = 500	145.9	-156.6	99.7	256.3	20.6
g = 1000	138.6	-143.6	83.2	226.8	17.3
g = 4000	127.1	-129.6	63.7	193.3	14.0
Fully Fixed	81.1	-91.5	80.6	172.1	12.0

Case 4

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-58.1	53.6	111.7	14.3
Pinned	38.9	-142.6	161.6	304.2	41.5
g = 100	29.8	-78.4	82.2	160.6	19.6
g = 200	20.3	-66.2	77.9	144.1	13.5
g = 300	16.9	-50.9	52.8	103.7	11.7
g = 400	14.9	-43.4	45.6	89.0	10.4
g = 500	13.9	-59.2	55.7	114.9	10.4
g = 1000	11.0	-33.4	44.5	77.9	7.5
g = 2000	9.6	-30.3	39.2	69.5	6.2
g = 3000	9.2	-23.2	25.0	48.2	6.0
g = 4000	8.8	-26.4	33.5	59.9	5.3
Fully Fixed	8.2	-25.2	30.2	55.4	5.4

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Table 3: Displacement Results (Continued)

Case 5

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-15.0	14.6	29.6	3.5
Pinned	34.4	-43.0	32.6	75.6	3.7
g = 100	39.0	-47.2	38.6	85.8	3.8
g = 200	30.0	-37.7	23.6	61.3	2.5
g = 300	26.2	-33.5	21.7	55.2	2.0
g = 400	24.1	-31.1	19.1	50.2	1.8
g = 500	22.6	-29.5	18.9	48.4	1.5
g = 1000	19.3	-25.3	18.1	43.4	1.4
g = 2000	17.4	-22.9	15.7	38.6	1.2
g = 3000	16.5	-22.1	14.1	36.2	1.1
g = 4000	16.2	-21.6	16.8	38.4	1.2
Fully Fixed	15.0	-20.1	16.1	36.2	1.0

Case 6

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-105.9	95.6	201.5	21.0
Pinned	78.2	-84.2	81.1	165.3	19.2
g = 100	60.2	-71.7	66.2	137.9	15.6
g = 200	47.0	-55.8	48.4	104.2	11.0
g = 300	40.9	-48.7	46.0	94.7	9.9
g = 400	37.0	-44.8	38.6	83.4	8.0
g = 500	34.5	-42.0	38.0	80.0	7.4
g = 1000	28.8	-35.8	35.1	70.9	5.5
g = 2000	25.3	-32.2	21.4	53.6	4.1
g = 3000	24.1	-30.8	24.8	55.6	4.0
g = 4000	23.4	-30.1	29.4	59.5	3.7
Fully Fixed	21.4	-27.9	21.5	49.4	3.6

Case 7

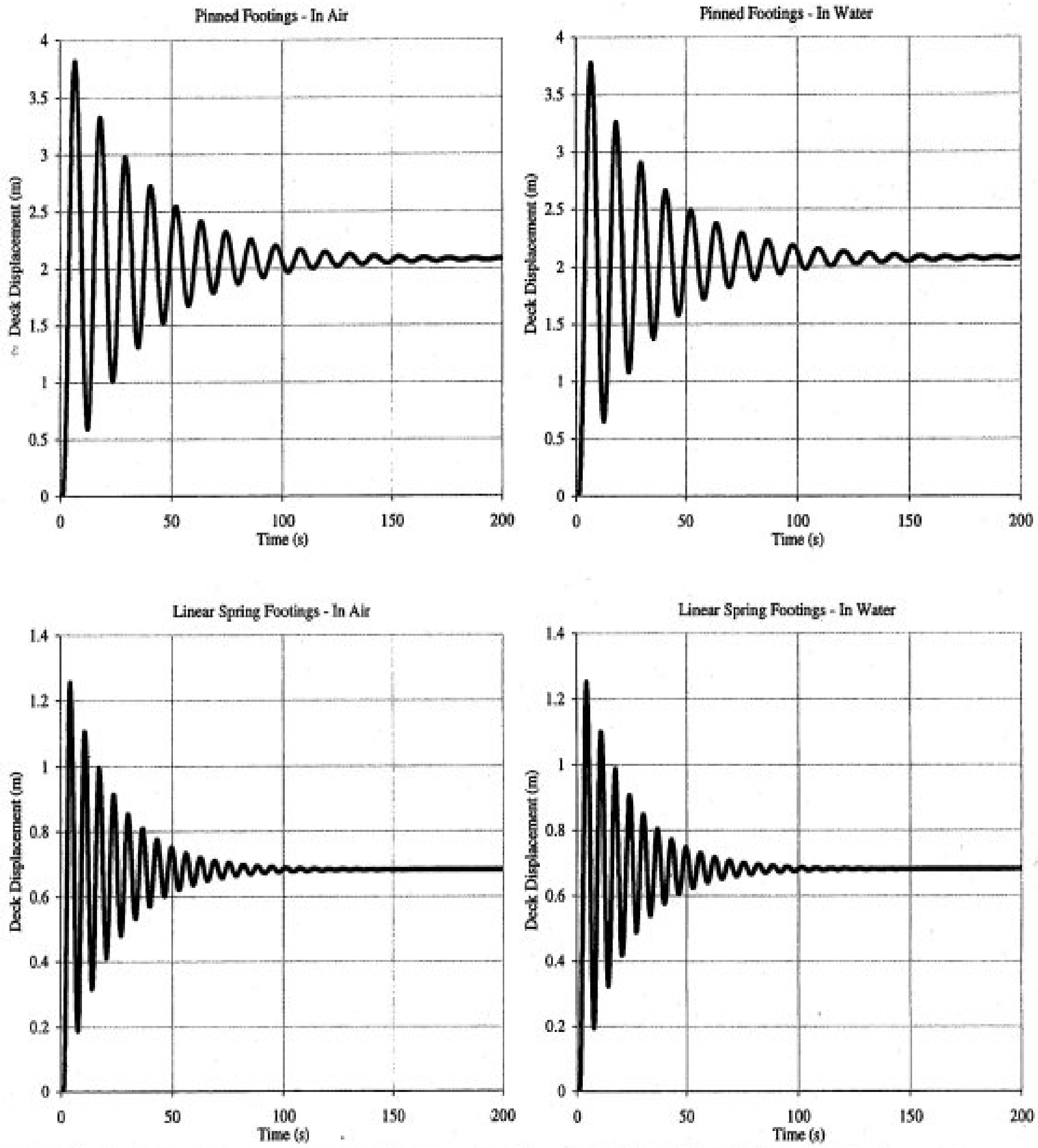
	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-43.7	40.2	83.9	7
Pinned	57.5	-57.5	63.3	120.8	10.6
g = 100	64.0	-64.0	53.5	117.5	11.0
g = 200	48.2	-48.2	41.5	89.7	8.8
g = 300	41.5	-41.6	48.8	90.4	7.4
g = 400	37.7	-37.8	39.1	76.9	6.3
g = 500	30.5	-35.0	31.8	66.8	5.1
g = 1000	28.6	-28.5	20.2	48.7	3.2
g = 2000	24.6	-24.7	21.9	46.6	2.7
g = 3000	23.1	-23.1	18.5	41.6	2.4
g = 4000	22.3	-22.3	18.1	40.4	2.1
Fully Fixed	16.8	-16.9	13.7	30.6	1.6

Case 8

	Mean	Range of displacement		Range	Standard deviation
	(mm)	lower (mm)	upper (mm)	(mm)	(mm)
Observed	N.A.	-42.9	53.8	96.7	10.3
Pinned	90.4	-125.5	198.4	323.9	36.1
Ir = 80	62.3	-103.1	131.8	234.9	27.2
Ir = 150	50.5	-86.9	104.6	191.5	22.3
Ir = 200	46.0	-76.8	109.1	185.9	19.0
Ir = 300	42.6	-72.9	83.3	156.2	15.4
Ir = 350	40.8	-57.5	68.9	126.4	15.2
Ir = 400	37.8	-60.8	79.2	140.0	14.1
Ir = 500	35.9	-69.5	74.7	144.2	13.1
Ir = 600	34.4	-51.9	72.4	124.3	11.9
Fully Fixed	26.2	-39.2	46.8	86.0	8.0

OXFORD FIGURES FOR SECTIONS 7, 8 & 9

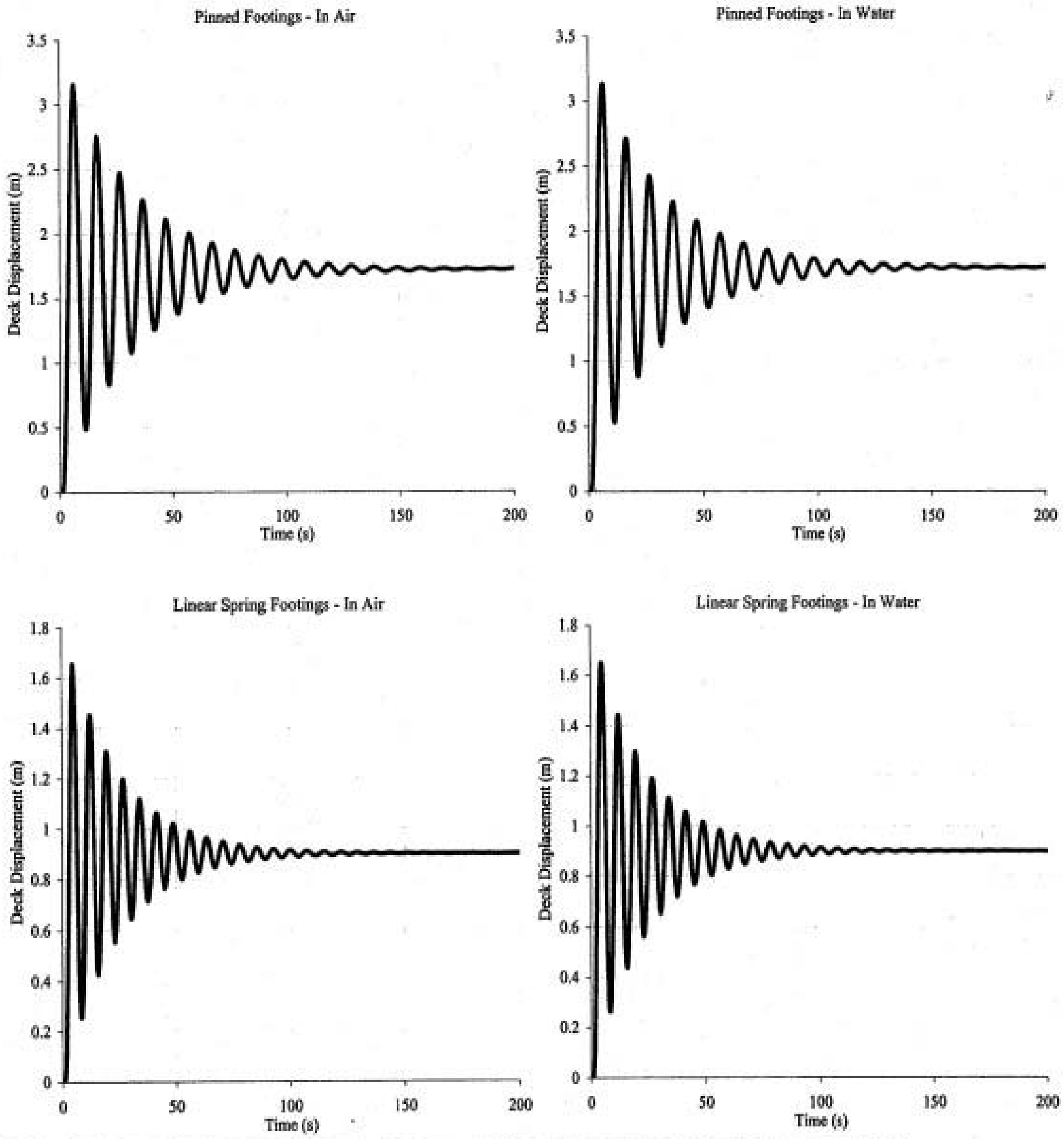
Figure 1



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	3.816	0.0526	3.776	0.0579	1.236	0.0566	1.23	0.0565
2	3.327	0.0525	3.258	0.0564	1.07	0.0567	1.066	0.0577
3	2.976	0.0527	2.906	0.0558	0.9535	0.0568	0.949	0.0574
4	2.723	0.0528	2.661	0.0549	0.8718	0.0569	0.868	0.0580
5	2.541	0.0529	2.491	0.0541	0.8146	0.0570	0.811	0.0570
6	2.41	0.0536	2.372		0.7745	0.0567	0.772	0.0548
7	2.315				0.7466		0.7456	
mean	2.078		2.078		0.6814		0.6814	
T_n (est.) (s)	11.28				6.44			

Case 1: Calibration Test (10MN applied at deck)

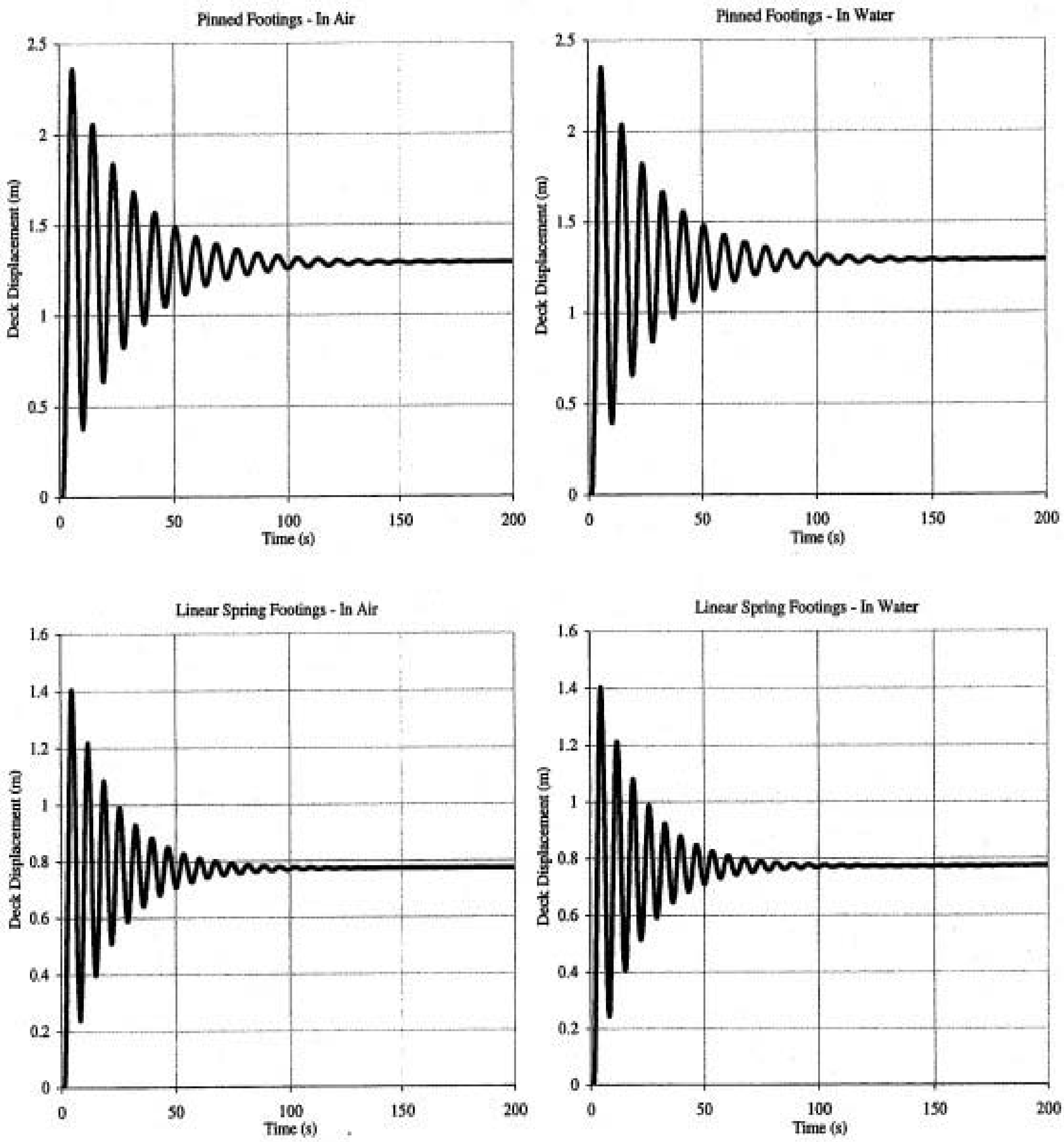
Figure 2



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	3.158	0.0516	3.132	0.0559	1.655	0.0493	1.65	0.0512
2	2.759	0.0515	2.713	0.0546	1.454	0.0492	1.444	0.0506
3	2.471	0.0517	2.424	0.0540	1.307	0.0492	1.296	0.0497
4	2.262	0.0517	2.221	0.0528	1.199	0.0498	1.19	0.0501
5	2.111	0.0517	2.079	0.0528	1.119	0.0492	1.112	0.0501
6	2.002	0.0519	1.977	0.0527	1.061	0.0485	1.055	0.0493
7	1.923		1.904		1.019		1.014	
mean	1.718		1.718		0.901		0.901	
T_n (est.) (s)	10.18333				7.3			

Case 2: Calibration test

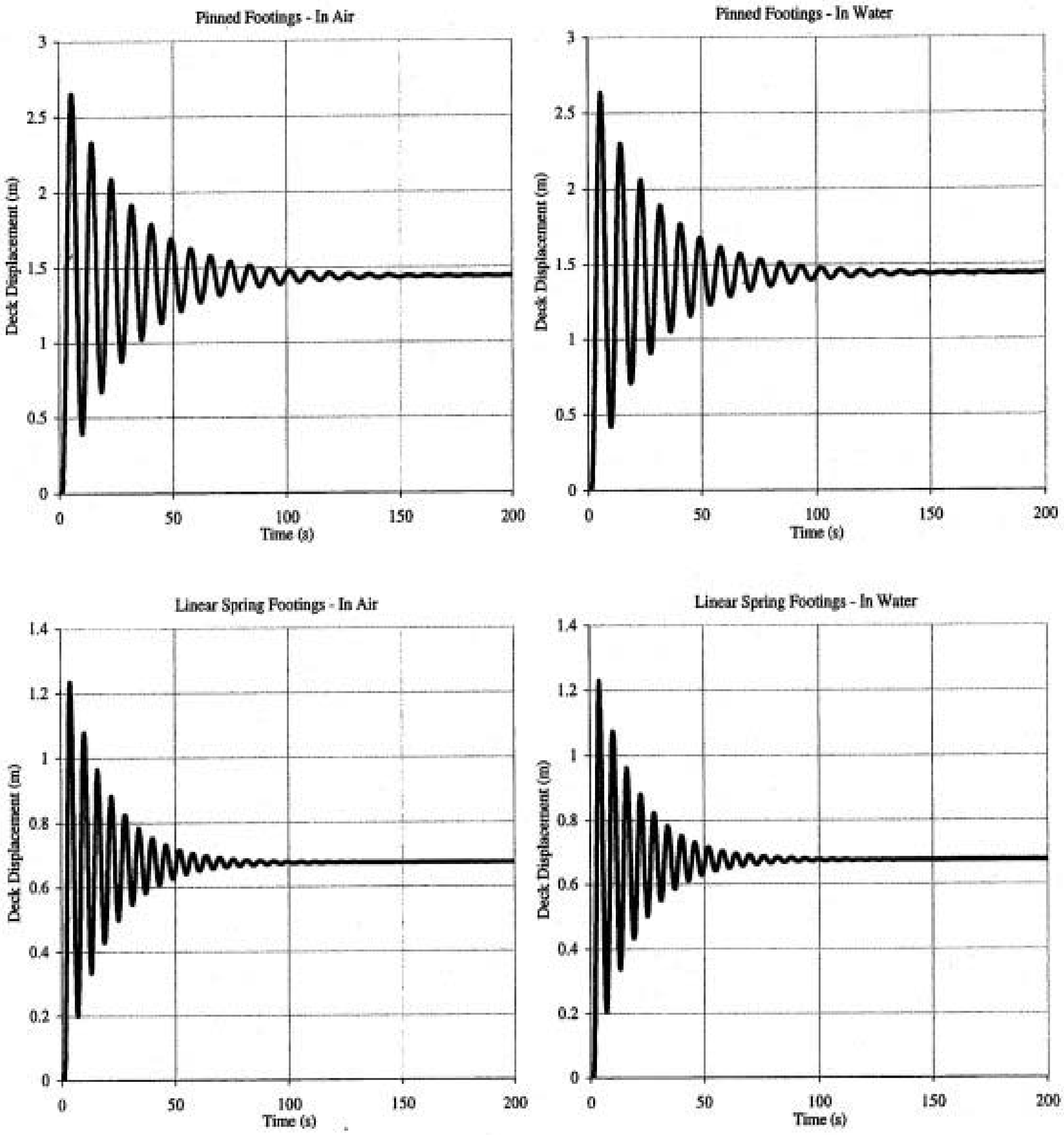
Figure 3



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	2.362	0.0533	2.351	0.0557	1.406	0.0560	1.403	0.0574
2	2.056	0.0536	2.037	0.0553	1.218	0.0563	1.212	0.0568
3	1.836	0.0532	1.817	0.0542	1.085	0.0565	1.08	0.0570
4	1.68	0.0534	1.664	0.0543	0.9914	0.0564	0.9872	0.0567
5	1.568	0.0533	1.555	0.0539	0.9259	0.0565	0.9227	0.0566
6	1.488	0.0531	1.478	0.0541	0.8799	0.0566	0.8776	0.0565
7	1.431		1.423		0.8476		0.846	
mean	1.287		1.287		0.7719		0.7719	
T_n (est.) (s)	9				7			

Case 3: Calibration test

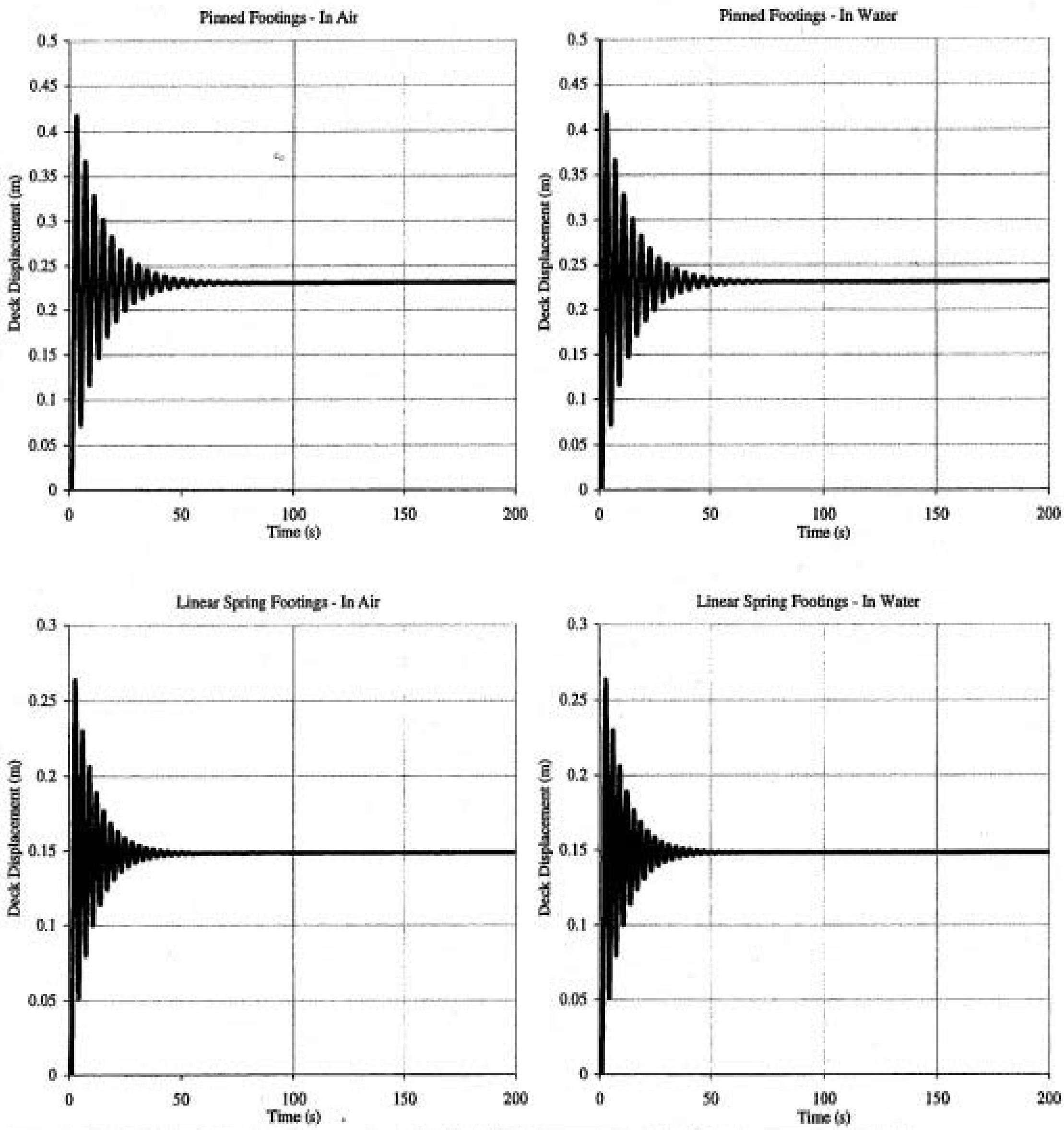
Figure 4



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	2.653	0.0493	2.633	0.0528	1.234	0.0524	1.23	0.0533
2	2.329	0.0493	2.295	0.0519	1.077	0.0523	1.072	0.0532
3	2.091	0.0491	2.056	0.0510	0.9642	0.0521	0.9591	0.0528
4	1.917	0.0492	1.886	0.0503	0.8833	0.0523	0.8788	0.0526
5	1.789	0.0493	1.764	0.0497	0.8248	0.0525	0.8213	0.0524
6	1.695	0.0485	1.676	0.0503	0.7826	0.0519	0.7801	0.0521
7	1.627		1.611		0.7525		0.7506	
mean	1.436		1.436		0.6745		0.6745	
T_n (est.) (s)	8.733333				5.966667			

Case 4: Calibration test

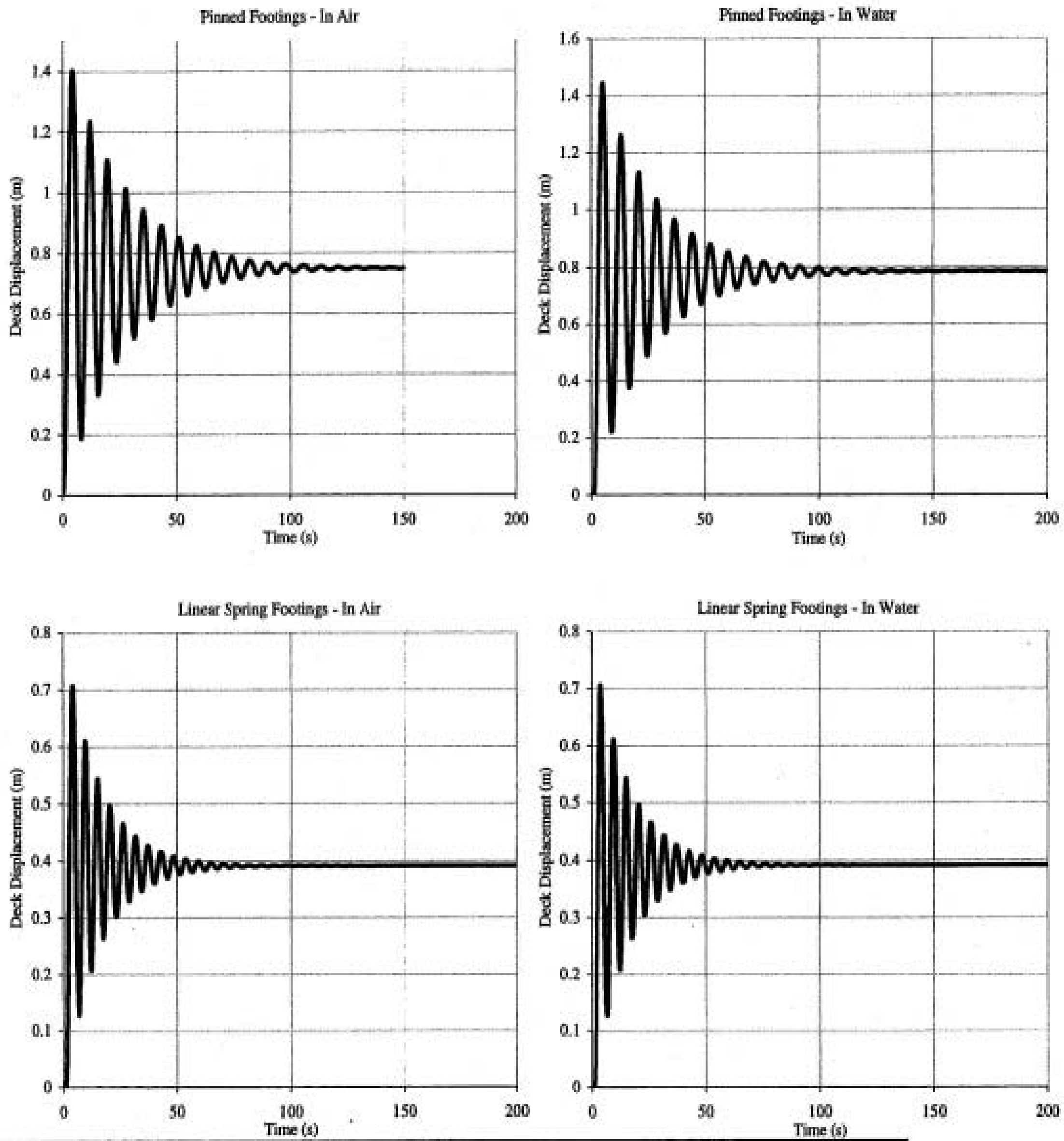
Figure 5



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	0.4177	0.0511	0.4178	0.0513	0.2637	0.0547	0.2636	0.0216
2	0.3664	0.0530	0.3663	0.0529	0.2301	0.0554	0.2301	0.0176
3	0.328	0.0512	0.328	0.0513	0.206	0.0561	0.206	0.0140
4	0.3013	0.0510	0.3012	0.0513	0.1888	0.0550	0.1887	0.0103
5	0.282	0.0531	0.2818	0.0524	0.1769	0.0541	0.1769	0.0078
6	0.2675	0.0496	0.2675	0.0506	0.1686	0.0540	0.1684	0.0057
7	0.258		0.2575		0.1627		0.1625	
mean	0.231		0.2308		0.1481		0.1481	
T_n (est.) (s)	3.95				3.17			

Case 5: Calibration test

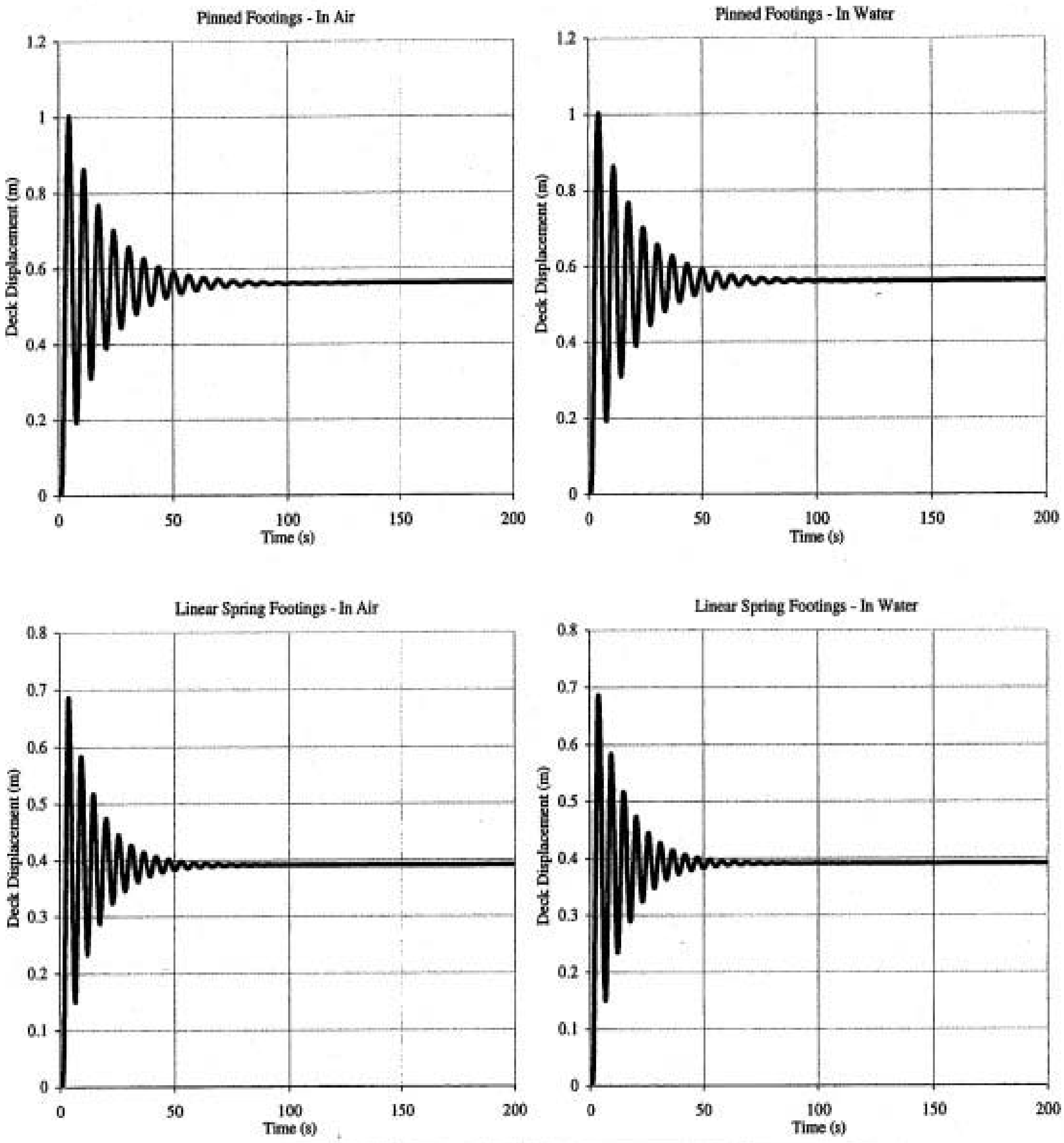
Figure 6



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	1.433	0.0490	1.437	0.0530	0.7069	0.0578	0.7058	0.0583
2	1.261	0.0511	1.252	0.0523	0.6107	0.0579	0.6092	0.0583
3	1.13	0.0510	1.121	0.0520	0.5436	0.0579	0.5422	0.0584
4	1.035	0.0514	1.027	0.0520	0.497	0.0581	0.4957	0.0579
5	0.9656	0.0512	0.9593	0.0515	0.4645	0.0580	0.4637	0.0578
6	0.9156	0.0514	0.9108	0.0516	0.442	0.0583	0.4415	0.0581
7	0.879		0.8757		0.4263		0.426	
mean	0.784		0.784		0.3908		0.3908	
T_n (est.) (s)	7.883333				5.55			

Case 6: Calibration test

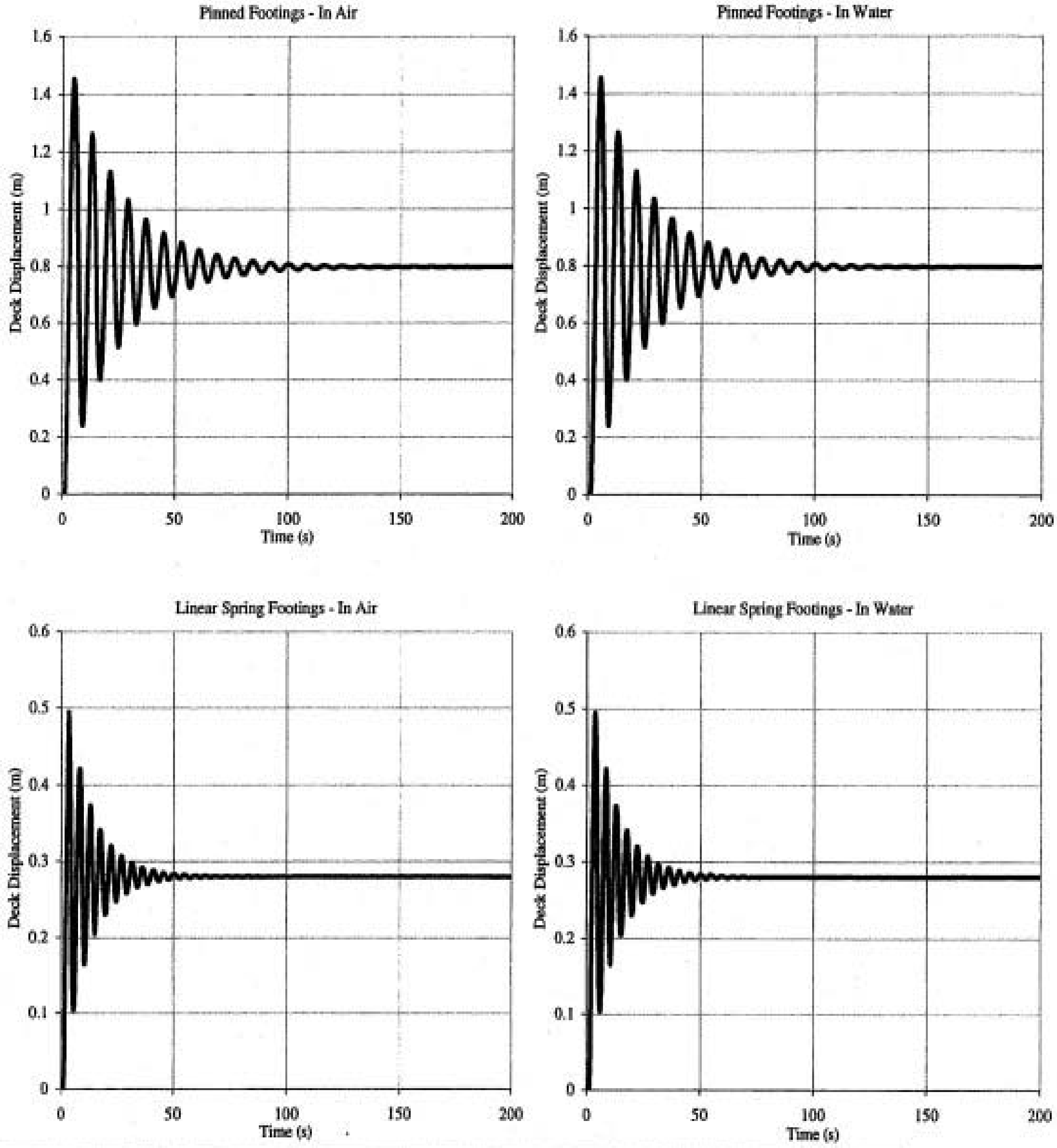
Figure 7



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	1.003	0.0605	1.002	0.0615	0.6858	0.0676	0.6853	0.0678
2	0.8631	0.0610	0.8606	0.0613	0.5837	0.0676	0.5831	0.0681
3	0.7668	0.0609	0.7648	0.0615	0.5169	0.0682	0.5161	0.0679
4	0.7013	0.0611	0.6995	0.0613	0.4729	0.0677	0.4725	0.0681
5	0.6565	0.0610	0.6552	0.0613	0.4444	0.0680	0.444	0.0677
6	0.626	0.0610	0.6251	0.0613	0.4257	0.0680	0.4255	0.0678
7	0.605		0.6046		0.4135		0.4134	
mean	0.561		0.561		0.3906		0.3906	
T_n (est.) (s)	6.55				5.45			

Case 7: Calibration test

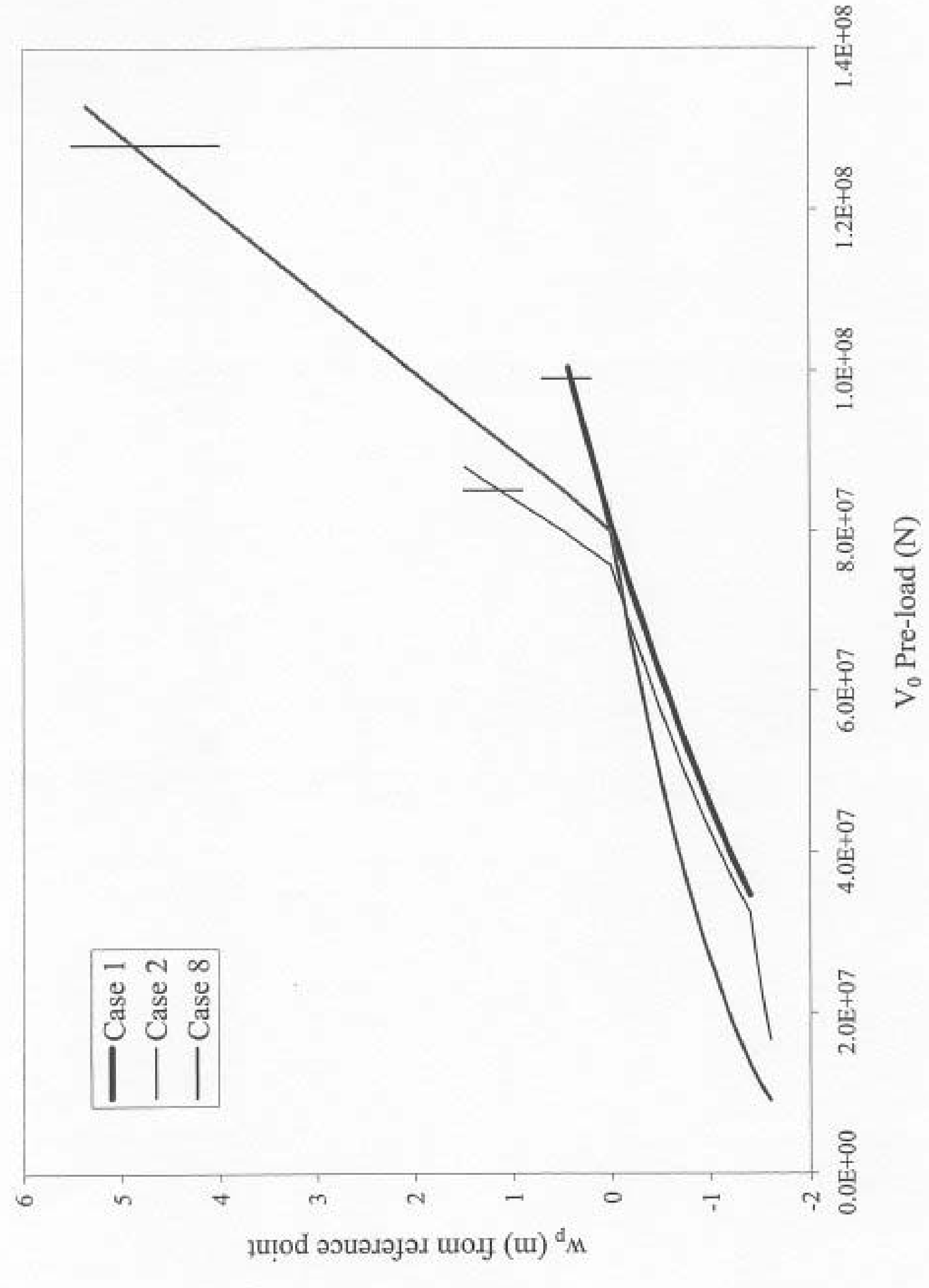
Figure 8



	Pinned Footings				Linear Springs			
	In air		In water		In air		In water	
Peak	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ	Deck Disp. (m)	ξ
1	1.454	0.0546	1.449	0.0565	0.4948	0.0666	0.4946	0.0672
2	1.263	0.0543	1.254	0.0556	0.4214	0.0666	0.4207	0.0668
3	1.128	0.0544	1.119	0.0555	0.3731	0.0671	0.3725	0.0666
4	1.032	0.0548	1.024	0.0548	0.3411	0.0672	0.3409	0.0670
5	0.9633	0.0546	0.9576	0.0549	0.3201	0.0661	0.32	0.0669
6	0.9148	0.0545	0.9105	0.0548	0.3065	0.0673	0.3063	0.0661
7	0.880		0.8772		0.2974		0.2974	
mean	0.796		0.796		0.2801		0.2801	
T_n (est.) (s)	7.933333				4.666667			

Case 8: Calibration test

Figure 9



Case 1
Case 2
Case 8
0.0E+00
2.0E+07
4.0E+07
6.0E+07
8.0E+07
1.0E+08
1.2E+08
1.4E+08
V_0 Pre-load (N)
6
5
4
3
2
1
0
-1
-2
w_p (m) from reference point

Figure 10

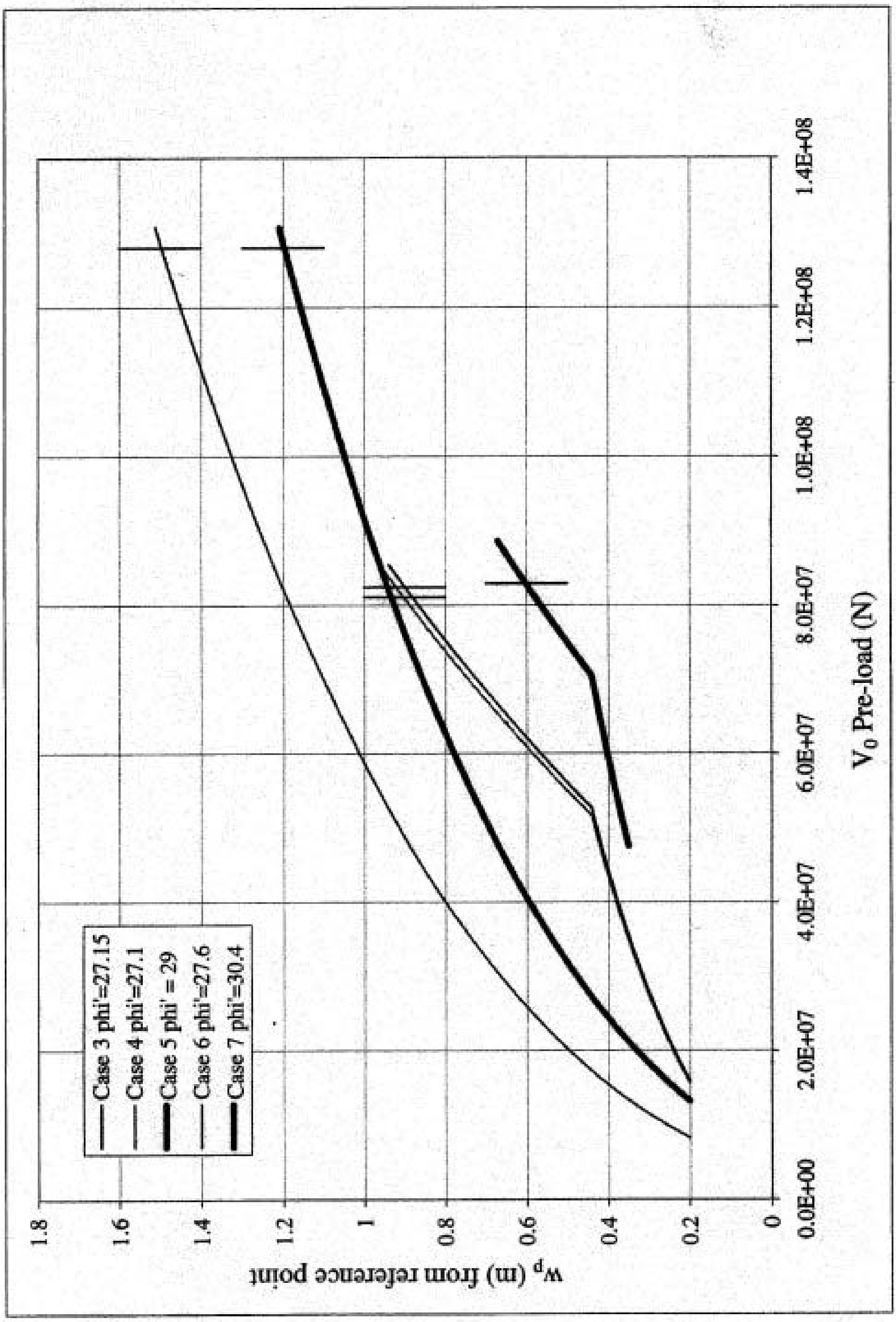

Case 3 phi'=27.15
Case 4 phi'=27.1
Case 5 phi' = 29
Case 6 phi'=27.6
Case 7 phi'=30.4
w_p (m) from reference point
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.0E+00
2.0E+07
4.0E+07
6.0E+07
8.0E+07
1.0E+08
1.2E+08
1.4E+08
V_0 Pre-load (N)

Figure 11: Stiffness factors for clay