06 Apr 1995, 10:30 am - 12:30 pm

North Morecambe Terminal, Barrow, Ground Stabilisation and Pile Foundations

C. A. Raison
Keller Foundations

B. C. Slocombe
Keller Foundations

A. L. Bell
Keller Limited

J. I. Baez
Hayward Baker Inc.

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 2-7, 1995, Volume I, St. Louis, Missouri

North Morecambe Terminal, Barrow, Ground Stabilisation and Pile Foundations

Paper No. 3.02

C.A. Raison and B.C. Slocombe
Chief Engineer and Engineering Manager, Keller Foundations

J.I. Baez
Geotechnical Engineer, Hayward Baker Inc.

A.L. Bell
Director, Keller Limited

SYNOPSIS Very poor ground conditions and soils that were susceptible to liquefaction to depths of up to 20m beneath the site were major problems for the construction of the largest natural gas processing terminal in Europe. An innovative foundation solution combining deep vibro compaction with short cast in place piles driven into the treated soils was adopted as both cost and programme effective. The Design and Build project also included advance civils work comprising earthworks, drainage and road construction. The contract started in September 1991 and was completed by October 1992, with the process plant programmed to be on-stream by October 1994. This paper describes the seismic aspects of the foundation solution.

INTRODUCTION

The North Morecambe Terminal is currently under construction close to Barrow-in-Furness which lies in the north west of England. The project comprises a natural gas processing plant being built by British Gas adjacent to their existing South Morecambe unit. The project is being built to cope with the effects of localised earthquake events.

The United Kingdom is located within the Eurasian Plate in a stable intraplate region remote from the boundaries. Although characterised as being of relatively low seismicity there is an extensive UK historical database for earthquake risk. This has revealed that the closest recorded earthquake occurred in 1865 less than 2km from the site during which liquefaction and sand boils were seen on the seashore.

British Gas carried out an extensive seismic risk assessment that concluded that two levels of earthquake should be applied to the design of the process plant and foundations. The study designated a number of selected critical plant items that had to resist a 1 in 10,000 year earthquake. All other plant had to resist a lower level 1 in 500 year event. The two levels of earthquake were represented by Richter scale events of 6.0 and 5.25 respectively with an epicentre at 15km distance and 10km depth. Accelerations of 0.20g and 0.05g were anticipated at bedrock with amplification due to soil conditions to 0.28g and 0.08g at the ground surface.

Keller Foundations successfully bid for civils and foundation works for the project. This paper describes the unusual pile design and foundation solution proposed by Keller to overcome the earthquake risk. The paper also presents a

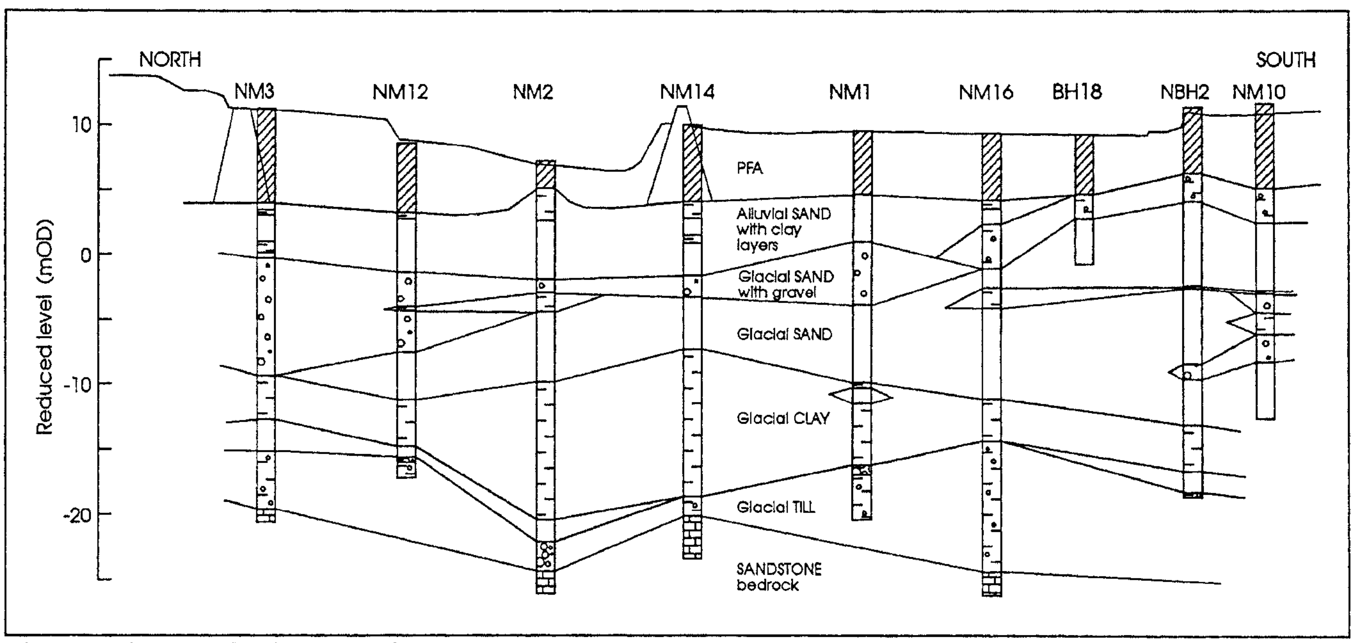

Fig.1 Typical Geological Section

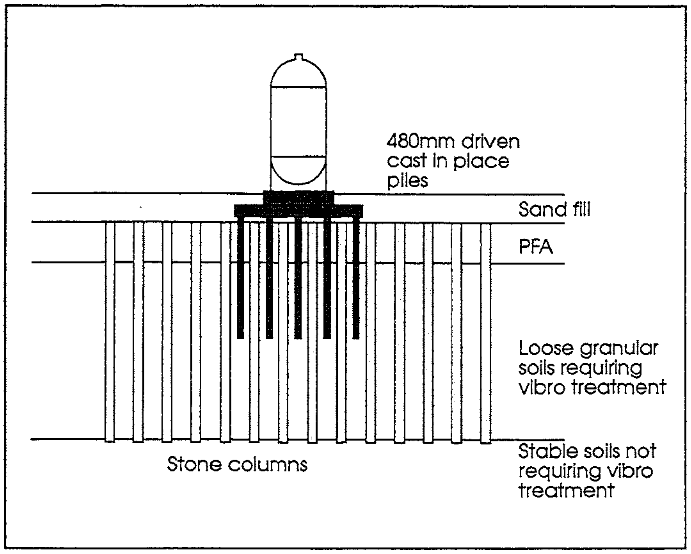

Fig.2 Proposed Foundation Solution

summary of the detailed seismic analyses carried out to justify the design.

GROUND CONDITIONS

Before development, the majority of the site comprised three former settlement lagoons containing pulverised fuel ash (PFA) that had been placed as a slurry. Site investigation included boreholes, static cone penetration (CPT) and piezocone tests, electrical resistivity and seismic cone soundings together with loading tests. An extensive laboratory testing programme included cyclic load tests.

Beneath the PFA, up to 7m of weak alluvial soils varying from loose silty sands to weak silty clays were present over large areas of the site, (see Fig. 1). Glacial soils were also encountered comprising upper glacial till, fluvioglacial sands or sand and gravel, lacustrine lake clay, with a lower glacial till. Much interlayering of these materials is shown but with the cohesive deposits being more common at depth. Sandstone bedrock was reached at a depth between about 28m to in excess of 40m below ground level.

THE PROBLEM

The technical problems facing Keller related primarily to the presence of PFA to depths of up to 6m below the proposed finished grade underlain by loose granular soils. The seismic assessment had suggested that the PFA would liquefy during both levels of earthquake, and during a 0.20g earthquake the loose sands could liquefy to depths of up to 20m below ground. These conclusions were subsequently confirmed by the analyses carried out as part of the design.

THE SOLUTION

The adopted solution was a combination of vibro densification techniques and the use of short piles driven into the treated soils, (see Fig. 2). The aim was to remove the potential for liquefaction in the deep deposits by vibro treatment, utilise surface drainage to control PFA liquefaction during the 500 year event, and to use piles to carry loads from the plant items through the very weak PFA to the underlying

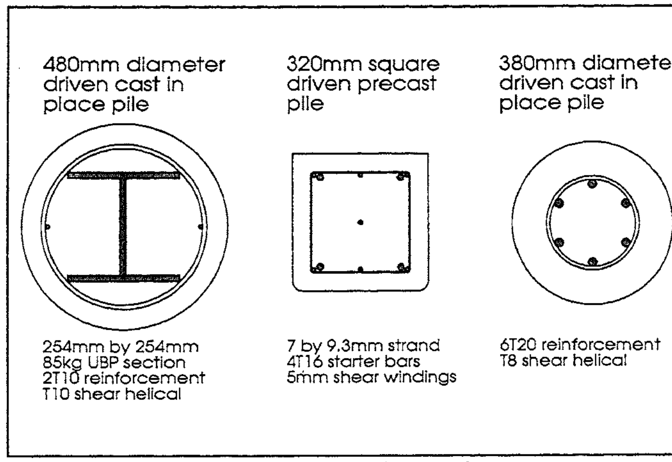

Fig.3 Section Details of Driven Piles

soils. The design also required the piles beneath the critical structures to cope with the effects of the PFA liquefying during the 0.20g earthquake.

For critical plant items and structures Keller proposed 480mm diameter cast in place driven piles. These piles incorporated a 254mm by 254mm Universal Bearing Pile steel section to act as reinforcement, (see Fig. 3). This would ensure continuity of support during the anticipated earthquake with allowance for up to 100mm lateral displacements. 320mm square prestressed single length precast piles were proposed to be driven for the remaining plant items. As an alternative to the precast piles, less heavily reinforced 380mm driven cast in place piles were also used.

The key advantages of the dual system solution to the client were:

An assured construction programme resulting from linking two low cost high production techniques.

A more predictable seismic behaviour.

The approach also enabled the installation of additional low cost piles to be installed to accommodate plant design changes without the need for more vibro stone columns.

During the contract works over 6,000 vibro stone columns were installed together with 1,700 480mm cast in place piles with UBP reinforcement, 2,400 precast piles and 1,600 smaller diameter cast in place piles.

VIBRO DENSIFICATION REQUIREMENTS

Vibro densification techniques have been applied successfully to many projects worldwide to reduce the potential for soil liquefaction. Assessment of the required level of treatment is usually based on analyses performed using simplified procedures to assess liquefaction potential. These are used together with knowledge of the soil densification achievable with the available equipment and proposed treatment grid. Of particular value to Keller for this project was the experience gained carrying out similar work on the adjacent British Gas South Morecambe Terminal site.

Design soil profiles were obtained from the site

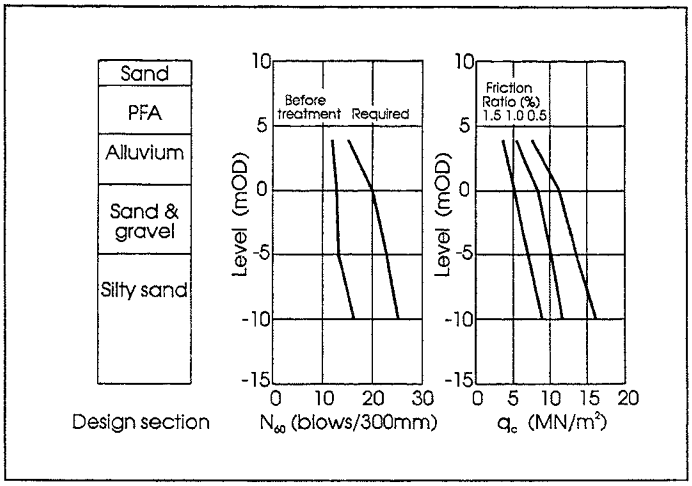

Fig.4 Vibro Treatment Densification Requirement

investigation borehole and CPT data. Fig. 4 shows a simplified soil section together with a typical before treatment SPT $(N)_{60}$ profile. Based on this, the required penetration resistance to prevent liquefaction during the 10,000 year earthquake event was obtained using Seed et al (1985). The result is shown in terms of a clean sand $(N)_{60}$ blow count. As it was proposed to carry out all control of the vibro densification using cone testing, it was necessary to convert the SPT $(N)_{60}$ values into CPT cone resistance profiles. Typical q_c profiles for friction ratios of 0.5%, 1% and 1.5% are shown. These were then used together with an extensive programme of pre-treatment CPTs to define the required depth of vibro works.

SEISMIC ANALYSIS

Although vibro techniques have been used worldwide to reduce the likelihood of soil liquefaction, this project is believed to be the first where piles have been designed to be supported by the treated ground. In view of this, more rigorous analyses were considered necessary to justify all aspects of the foundation design under seismic conditions. This was carried out by Keller in close liaison with the University of Southern California who provided much advice and carried out the detailed seismic analyses.

EARTHQUAKE RECORDS

Selection of suitable earthquake records was based on the hazard analysis response spectra defined by British Gas in their seismic assessment report. Three earthquake events were chosen for site studies:

> The 1979 Imperial Valley earthquake, Superstition Mountain N135 Component.
>
> The 1966 Parkfield earthquake, Array No 8, N50 WA.
>
> The 1988 Saguenay earthquake, Station 20, N-S Component.

Each earthquake time history was scaled to give a base peak acceleration of 0.05g and 0.20g for the 500 year and 10,000 year events respectively. Fig. 5 shows a comparison of the 5% damping response spectra for each record

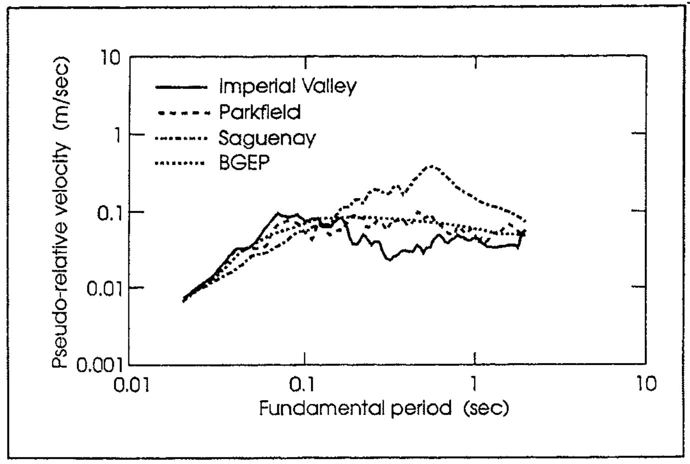

Fig.5 Selected Design Earthquake Spectra

compared with the British Gas spectrum.

DYNAMIC RESPONSE ANALYSES

Dynamic site response analyses were performed using the effective stress site response analysis code DESRA-II developed by Lee and Finn, (1978). The program enables calculation of the stress, build up of pore pressures and acceleration throughout a vertical column of soil divided into appropriate layers. Four pore pressure constants are required for each layer to define the liquefaction strength at a given cyclic stress ratio. Additional soil properties specified included rebound parameters, low strain maximum shear modulus, damping and residual shear strength following liquefaction. Earthquake records or sinusoidal input motions can be defined.

To obtain suitable input parameters for analysis, liquefaction strength curves were needed for the soils requiring treatment. The site investigation data suggested that typically these soils were defined by $(N_1)_{60}$ of about 15 blows/300mm or q_c of about 6MN/m² for a friction ratio of 1%. Early field trials suggested ground improvement could increase the relative density to give $(N_1)_{60}$ of about 25 blows/300mm or q_c of about 10MN/m². Typical pre and post-treatment CPT profiles together with an assessed soil

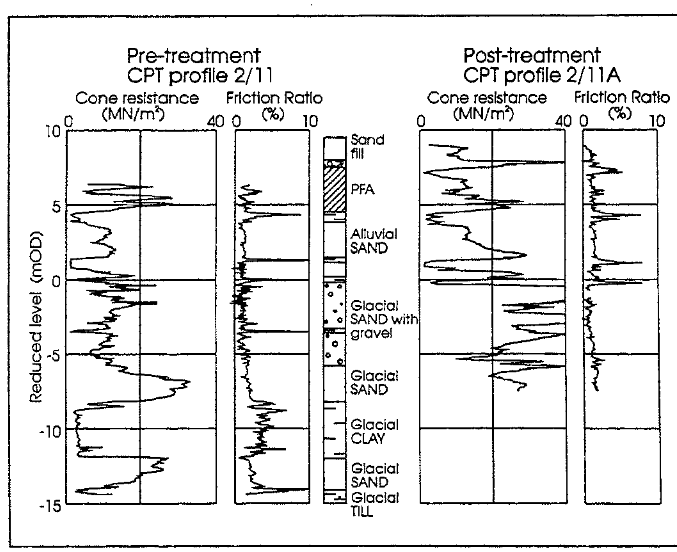

Fig.6 Typical Pre and Post-Treatment CPT

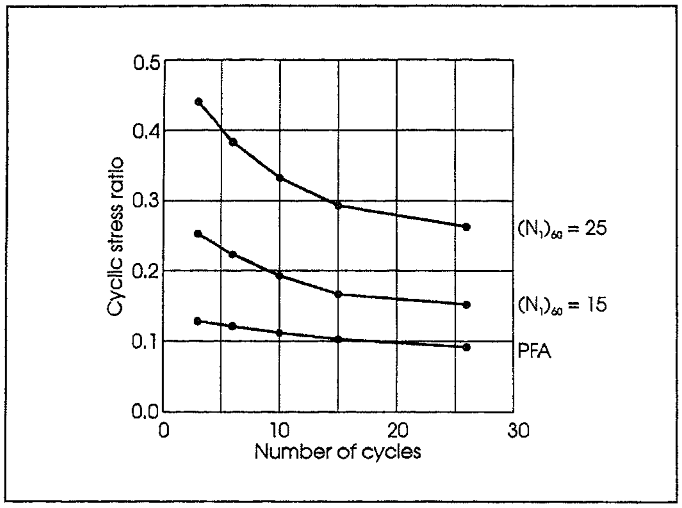

Fig.7 Liquefaction Strength Curves

section are given in Fig. 6. Generated liquefaction strength curves are shown in Fig. 7 together with a liquefaction strength curve for the PFA based on laboratory cyclic testing. These curves were calibrated for use by DESRA-II following the procedures given by Martin et al (1981).

RESULTS OF ANALYSES

DESRA-II analyses were carried out for both the treated and untreated condition with the three earthquake records noted above.

Results from the site response analyses assuming untreated soil properties show that the pore pressures in the PFA are unlikely to cause liquefaction during a 500 year event but exceed 35% of the effective overburden stress, (see Fig. 8). Pore pressures for the natural soils are significantly lower. For the higher level 10,000 year event the earthquake is expected to induce rapid liquefaction in the PFA. The underlying natural soils also show an unacceptable build up of pore pressure in excess of 50% overburden suggesting the possibility of significant ground movements and confirming the need for vibro treatment.

Analyses using post treatment soil parameters show little difference for the 0.05g event, but confirm that the vibro densification is able to control significant build up of pore pressures in the natural soils during the 0.20g earthquake. A build up of pore pressure in the layer immediately below the PFA is shown suggesting that some reduction in strength and stiffness is warranted in this layer.

Of particular interest is the output acceleration at ground level computed by the DESRA-II program. Fig. 9 shows a comparison between the input and output accelerograms for the Imperial Valley record 0.20g treated case. This shows a ground surface acceleration of about 0.27g indicating amplification of the bedrock motion through the superficial deposits. Output from the other DESRA-II runs show similar amplification.

The DESRA-II non linear effective stress site response analyses were also able to provide valuable insight on the likely post liquefaction lurch displacement across the liquefied zone of PFA. As a simplifying assumption, the shear movements will approximate the maximum displacements for the site response of a very long period single degree of freedom elastic system. However, the actual movement will depend on the ground velocity of the upper soil levels prior to liquefaction, the post liquefaction ground response of the soil beneath the liquefied layer and the residual strength of the PFA. Analyses indicated that post liquefaction movements would not exceed the design value of 100mm.

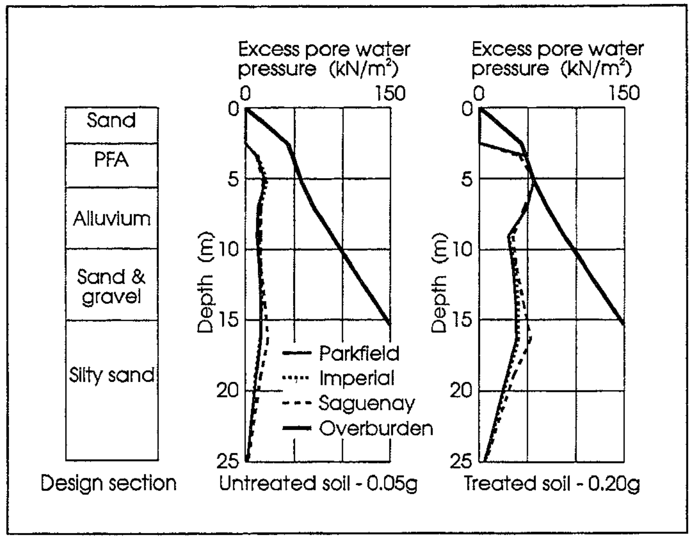

Fig.8 Computed Build Up of Pore Pressures

DRAINAGE

The DESRA-II site response analyses were based on the assumption that pore pressure build up would be dependent solely on the relative density of the soils. However, the geometry of the vibro treatment and size of the stone columns will allow significant drainage towards the columns during an earthquake. Computations based on the methodology of Seed and Booker (1977) suggest a reduction of pore pressure from about 50% to 13% of overburden pressure near to the level of the pile toes.

The presence of stone columns will also provide a ready drainage path for thin silty layers to dissipate local excess pore pressures to more permeable layers thus preventing local liquefaction problems.

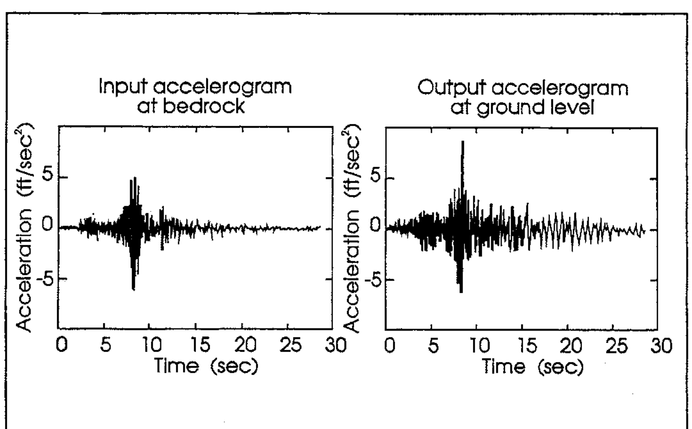

Fig.9 Computed Ground Motion Amplification

The effect of the partial replacement of the upper layers of PFA with a drainage layer and sand fill will be to maintain the water table below this level and prevent the spread of liquefaction into the upper fill. Increased vertical effective stresses also improve the resistance to liquefaction in the PFA and natural soils.

EXTENT OF VIBRO TREATMENT

The extent of the vibro treatment was defined by the need to protect critical plant items specified by British Gas. These items were scattered throughout the process plant and ground treatment was therefore performed beneath many non critical structures. Two criteria were used for assessing the extent of treatment; the required margin beyond the edge of the item and the minimum plan dimension of the treatment zone.

The required treatment margin beyond the edge of the critical plant was based on experience gained from the 1989 Loma Prieta earthquake in California. Use was also made of large shaking table tests performed in Japan, Iai et al (1988), to assess the potential edge slumping of stabilised ground into adjacent liquefied ground. Provided treatment extends beyond a margin defined by an angle of about 35 degrees to the vertical through the depth of potential liquefaction, overall stability will not be impaired.

The minimum effective size of the treatment zone is dependent on the stress concentration within the stabilised block. Two cases were analysed using two dimensional finite element methods. The first case considered full liquefaction of the soil surrounding the stabilised area where the ratio between the shear modulus of the treated and liquefied soils was taken as 1000. For the no liquefaction case, a ratio between the shear modulus of the treated and untreated soil of 2 was adopted. The maximum shear stresses were computed for various treatment widths between 20m and 125m. Typical results are shown in Fig. 10. For the full liquefaction case these analyses indicated that a width of treated area greater than 25m will ensure that the increase in shear stress is kept less than 50%.

PILE DESIGN

Seismic design of the piles was carried out considering three cases corresponding to different stages of the earthquake. The first stage, (see Fig. 11), was a pseudo elastic case assuming no liquefaction with an induced horizontal load equal to the mass of the plant item and foundation times the peak acceleration at the ground surface. For routine design, peak accelerations of 0.08g or 0.28g were assumed. Generally this is inappropriate as the inertial load will depend on the response spectra at ground level and the natural frequency of the foundations and superstructure. However, most of the plant items have massive pile caps and substructure which would tend to dominate the behaviour with small fundamental periods. The frequency response of certain critical structures, such as the pipe racks, together with their foundations, were checked more rigorously.

Assessment of pile deflection and bending moment was made using a suitable soil-pile interaction analysis computer program. Inertial loads to be applied were assumed to act at the head of the pile. Allowance was made for head fixity, and relationships established between lateral load and the resulting pile deflection and bending moment.

The second or partial liquefaction phase considers shear movements between the sand fill, drainage blanket, upper layers of the PFA and the underlying sands through a thin liquefied layer at the base of the PFA. A design shear of 100mm was taken which approximates to the maximum displacement for the site response of a very long period elastic system. The DESRA-II site response analyses confirmed that this value was reasonable.

For the 1 in 10,000 year event soil shearing movements were expected to cause the development of plastic hinges at the junction between pile and cap, and possibly at a lower level in the pile section. However since no mechanism could form, this would not result in failure. This effect was analysed using the same soil-pile interaction software and imposing soil shear movements over a 1m thick zone at the base of the PFA fill.

The last stage considers complete liquefaction of the PFA. The pile section is largely unrestrained by the soil and loads depend on the natural frequency of the pile. However, complete

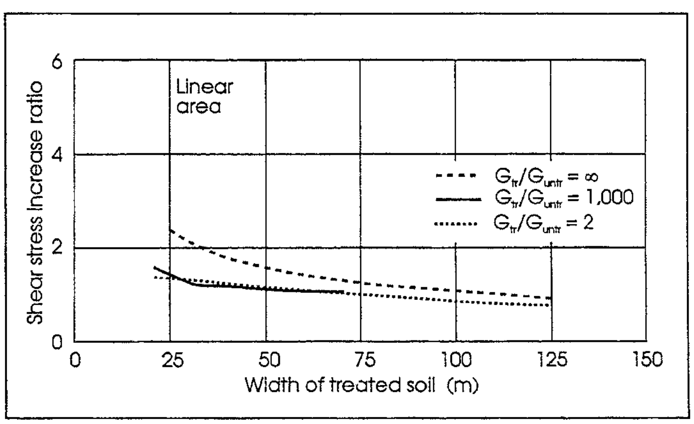

Fig.10 Minimum Extent of Vibro Treatment

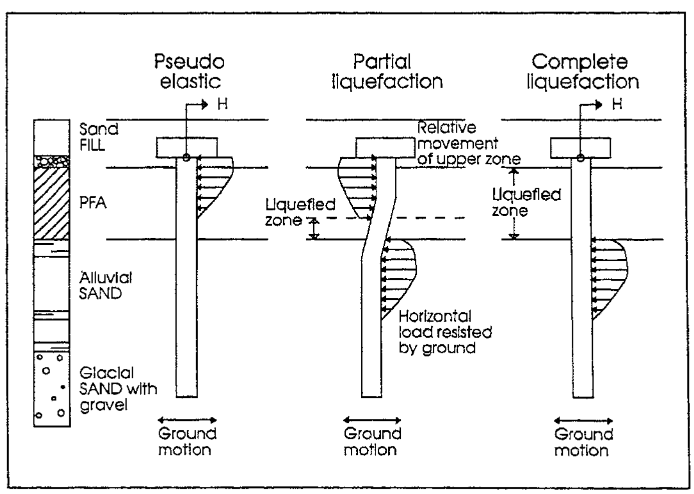

Fig.11 Idealised Behaviour of Piles

liquefaction does not occur and soil will always retain some residual strength. Soil-pile interaction analyses show that this case is less onerous than the partial liquefaction situation unless major lateral flow or lurching movements take place.

FOUNDATION SETTLEMENTS

Shakedown settlement of the alluvial and glacial soils will occur as a result of dissipation of pore pressures following cessation of the earthquake. The magnitude will be a function of the predicted maximum pore pressures and assumed stress strain unload reload relationship. Settlements based on the DESRA-II analyses and assumed soil properties are expected to be about 12mm and 4mm for the respective 10,000 year and 500 year events. These would result in small pile settlements due to negative shaft friction loading on pile shafts.

Following complete liquefaction of the PFA during the 10,000 year design earthquake, the material will reconsolidate under self weight. Reported results from the laboratory testing give maximum volumetric strains from the virgin consolidation curve of about 2.2%. Post earthquake settlements in the PFA could therefore approach 80mm to 100mm. Negative shaft friction loading on piles from the PFA has been taken into account.

More serious to the piling are potential settlements within lenses of loose soils immediately beneath the toes of the pile foundations or within the depth of the shaft. Great effort was taken to ensure that any thin layers of looser more silty soils were identified, and if necessary, retreated. Settlements within remaining layers were assessed by applying the Tokimatsu and Seed (1987) method using the post treatment CPT results performed prior to the piling. Typically settlements of about 5mm to 10mm were computed, substantially less than the 50mm which was allowed for.

CONCLUSIONS

The paper has described the foundation solution developed by Keller to the problems of poor ground conditions and the need to design for specified levels of seismic risk. It has also presented the background to a complex and unusual pile design involving different aspects of seismic, geotechnical and structural engineering.

ACKNOWLEDGEMENTS

This paper has been written with the permission of British Gas Exploration and Production.

The writers also wish to acknowledge the contribution made by Juan Baez and Geoff Martin of the University of Southern California who provided much advice during the progress of the design work and who carried out the detailed seismic analyses.

REFERENCES

Iai, S., Noda, S. and H. Tsuchida (1988), "Basic Considerations for Designing the Area of the Ground Compaction as a Remedial Measure Against Liquefaction", U.S.-Japan Joint Workshop on Remedial Measures for Liquefiable Soils, UJNR, Jackson, Wyoming, May 11-13.

Lee, M.K.W. and W.D.L. Finn (1978), "DESRA-II, Dynamic Effective Stress Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential", University of British Columbia, June 23, 1978.

Martin, G.R., Lam, I.P., McCaskie, S. and C. Tsai (1981), "A Parametric Study of an Effective Stress Liquefaction Model", Int. Conf. on Recent Advances in Geotechnical Engr. and Soil Dynamics, Vol. II : 699-705.

Seed, H.B., and J.R. Booker (1977), "Stabilisation of Potentially Liquefiable Sand Deposits Using Gravel Drains", Journal of Geotechnical Engr., ASCE, Vol. 103, GT7 : 757-768.

Seed, H.B., Tokimatsu, K., Harder, L.F. and R.M. Chung (1985), "Influence of SPT procedures in Soil Liquefaction Resistance Evaluations", Journal of Geotechnical Engr., ASCE, Vol. 111, GT12 : 1425-1445.

Tokimatsu, K. and H.B. Seed (1987), "Evaluation of Settlements in Sands due to Earthquake Shaking", Journal of Geotechnical Engr., ASCE, Vol. 113, GT8 : 861-878.