

HARDSTAND LOAD TESTING FOR HEAVY LIFT CRANES

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SUMMARY

Large capacity plate load tests were used to predict settlement of heavy lift cranes. The rear counterweight crawlers of the heavy lift cranes will have a mass of 2200 tonnes, and will apply a track bearing pressure of 490 kPa. The load testing program involved applying a 1000 tonne load over a 6 m long by 2.4 m wide area to result in a maximum applied bearing pressure of 680 kPa. Piezocone penetration testing and magnetic extensometers were used to assess deformation properties of the different soil units encountered at the site. This paper describes the field component of the load test program, the procedure for the assessment of soil deformation properties, and the predicted settlement behaviour for the heavy lift cranes.

INTRODUCTION

Two heavy lift cranes will be used to lift large prefabricated sections onto a large structure. Each of the heavy lift crane comprises a front crawler supporting a 140 m long boom, connected with a 35 m long strut to a rear counterweight crawler weighing 2200 tonnes.

Large-scale load testing was required to check the ground deformations which would occur in the hardstand area during operation of the heavy lift cranes. Preliminary analyses based on borehole data and conventional plate load tests indicated that settlements beneath the crane tracks of 250 mm to 290 mm may occur.

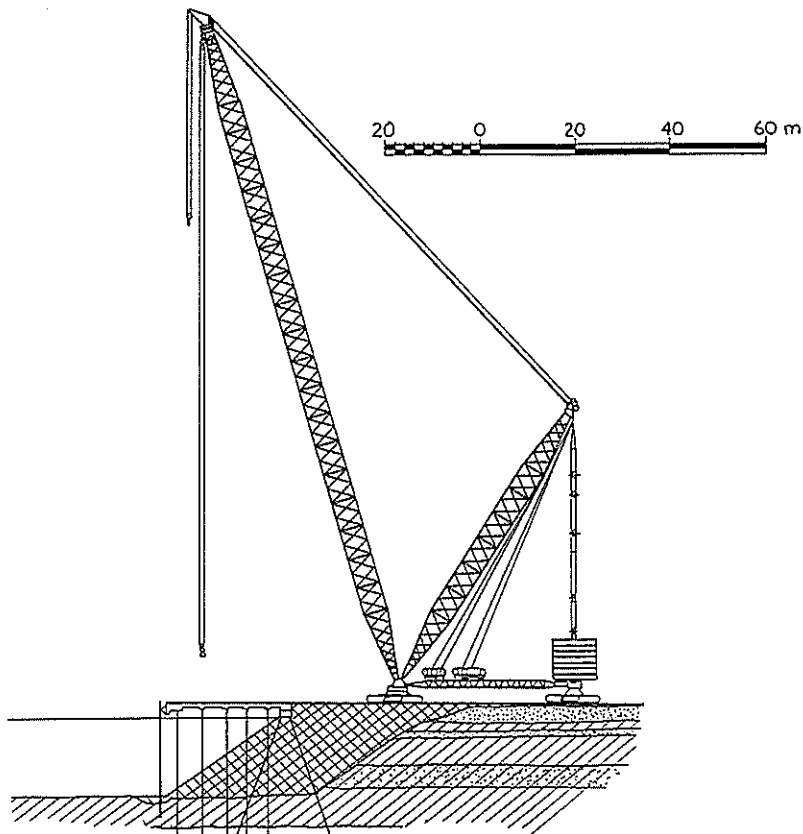


Figure 1 Heavy lift cranes on hardstand area.

This paper discusses the following aspects of the load test program:

- Additional site investigations performed to better define sub-surface conditions
- Installation of magnetic extensometers to measure vertical movement at various depths beneath the ground surface
- Load test methodology and results
- Assessment of soil deformation parameters
- Predicted settlement behaviour of the heavy lift crane

SITE CONDITIONS

The hardstand area is approximately 90 m by 60 m in area, and situated about 60 m from the shoreline of Port Kembla Inner Harbour. Site investigations performed prior to 1995 comprised four wash-bored boreholes, with undisturbed tube samples taken of clay soils, Standard Penetration Tests (SPT) performed in sands, and 15 in-situ vane shear tests performed in the soft clay soil.

Generally less than a metre thickness of blast furnace slag gravel occurred over the surface of the site. Underlying the slag fill was a 2.5 to 6 m thick layer of medium dense to dense sand and silty sand. Beneath the sand a very soft to soft high plasticity estuarine clay unit was found, which was up to three metres thick. Loose sand and another soft clay layer occurred below this depth, though these were less important with respect to crane settlement behaviour than the soils nearer to the surface.

LOAD TEST PROGRAM

The load test program involved additional geotechnical investigation of the hardstand area and the installation of instrumentation in preparation for the load tests. Three large scale load tests were subsequently performed at locations chosen based on the results of the additional investigation. Large scale tests were necessary in order to simulate as closely as possible the proposed crane loading, so that the same magnitude of stresses were developed in the soil profile, and non-linear stress-strain behaviour identified.

Piezocone Penetration Tests

Four boreholes drilled previously had shown variability in the near surface soils. Piezocone penetration testing was performed at ten locations within the hardstand area to more accurately assess the soil unit boundaries and to better assess (compared with SPT results) the relative density of the near-surface sand soils. The use of the piezocone (rather than the more common electric friction cone) enabled greater confidence in soil identification, with pore pressure behaviour highlighting the occurrence of clay layers. The piezocone test results were also used as a basis for the correlation of elastic parameters with cone resistance.

The ten tests were performed to depths of up to 16 m from the back of a conventional truck-mounted drilling rig. The use of a drill rig enabled cemented slag gravel on the surface to be easily pre-drilled using conventional augers, and enabled quick testing and retrieval of the probe when testing the soft clay soils. The same drill rig was also used to install the magnetic extensometers.

The piezocone test results indicated that the soft clays which underly the site occurred in two distinct layers, with a 1 to 3 m thick layer of silty sand and sand separating the two clay layers. Total cone resistance, q_t , was used to assess undrained shear strength of clays using the following relationship:

$$s_u = \frac{q_t - \sigma_v}{N_{kT}} \quad (1)$$

where

| | | |
|------------|---|----------------------------------|
| s_u | = | undrained shear strength |
| q_t | = | total cone resistance |
| σ_v | = | total vertical overburden stress |
| N_{kT} | = | piezocone factor. |

The parameter N_{kT} was found to range from 15 to 20, based on correlating it with the field vane test results. This range of N_{kT} is typical of high plasticity clays.

Of the 15 vane shear tests performed in the soft clays, all but one was performed below 6 m depth, and indicated shear strengths in the range 32 to 66 kPa. Using Eq. (1) it was found that the uppermost clay layer had shear strengths as low as 15 kPa, which was significantly less than that of the deeper clay unit, though of similar magnitude to the 19 kPa result from the one vane shear test which was performed above 6 m depth. The vane shear tests had not clearly identified that the shallow clay layer was considerably softer than the deeper clay unit.

The locations of the proposed load tests were based on the results of the piezocone tests, with load tests located in the areas considered to present the most severe conditions for settlement and bearing capacity of the heavy lift crane.

Magnetic Extensometers

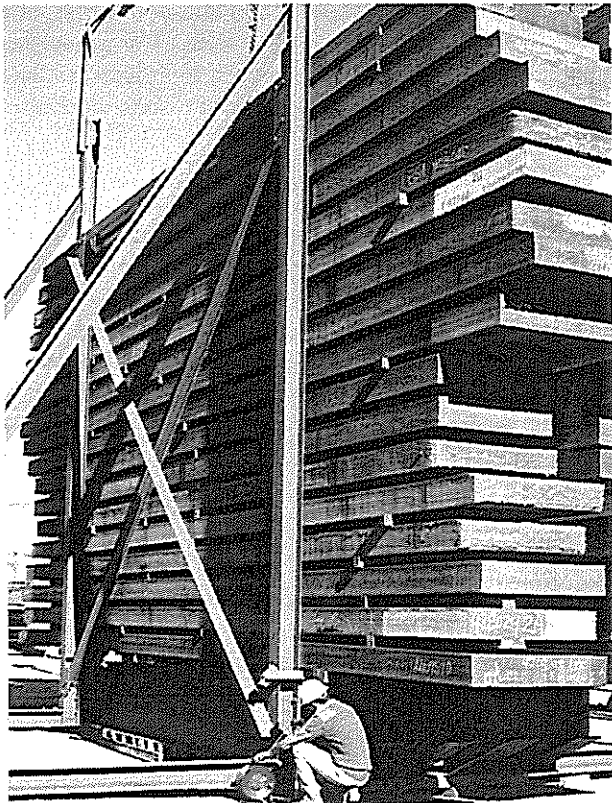
Magnetic extensometers were installed at the location of two of the three load tests. The extensometers allow for the measurement of vertical movements within the soil, thus allowing identification of the soil layers which contribute to the observed surface settlement.

Three extensometers were placed around the perimeter of the proposed location of each of the two load tests. Extensometers comprise magnetic targets within a flexible vertical tube. A magnetic sensor is lowered into the tube, and a reed switch is triggered when the sensor approaches a target, emitting a sound at the ground surface. It was possible to measure the location of magnetic targets to about 1 mm vertical accuracy. Extensometers were installed to depths of about 12 m, with four magnetic targets installed in each extensometer. The spacing of the targets was increased with depth.

Extensometers were installed about one month prior to the load tests being carried out to allow for consolidation of soil around the extensometer, so that the extensometer tubing would deform with the surrounding soil.

Load Tests

The load tests comprised the loading of a purpose-built steel frame with up to 44 steel ingots, each with a mass of 20 to 25 tonnes. The loading frame comprised four large parallel steel beams connected with smaller sections, resulting in a 2.4 m wide platform with a 6 m length in contact with the ground. The steel ingots were placed in layers, with each layer comprising two ingots. A maximum of 22 layers was placed in the frame, corresponding to a load of 995 tonnes, and an average ground bearing pressure of 680 kPa. Surface settlements were measured using a vernier automatic level, which was capable of measuring to an accuracy of 0.1 mm.



The first load test resulted in the least settlement of the three tests, with less than 50 mm observed at the surface. About 1.3 m of compacted slag fill occurred at the first test location, and surface cracks were observed to extend up to 15 m from two corners of the load frame, indicating that the slag fill was capable of transmitting the load over a considerable area. Extensometer data indicated that half of the settlement occurred below 2 m depth, and that the permanent deformation of 30 mm was attributable to soil units above 6 m depth.

The second test was performed at a location half way between the two test locations where extensometers had been installed. Settlement of 100 mm occurred immediately after loading the frame to 680 kPa, with an additional 70 to 100 mm occurring overnight. Permanent deformation of 160 mm was measured. The slag thickness was 0.6 to 0.9 m, significantly less than for the first test, resulting in reduced capacity of the slag to spread the load. As a result surface cracking was over a much smaller area than for the first test.

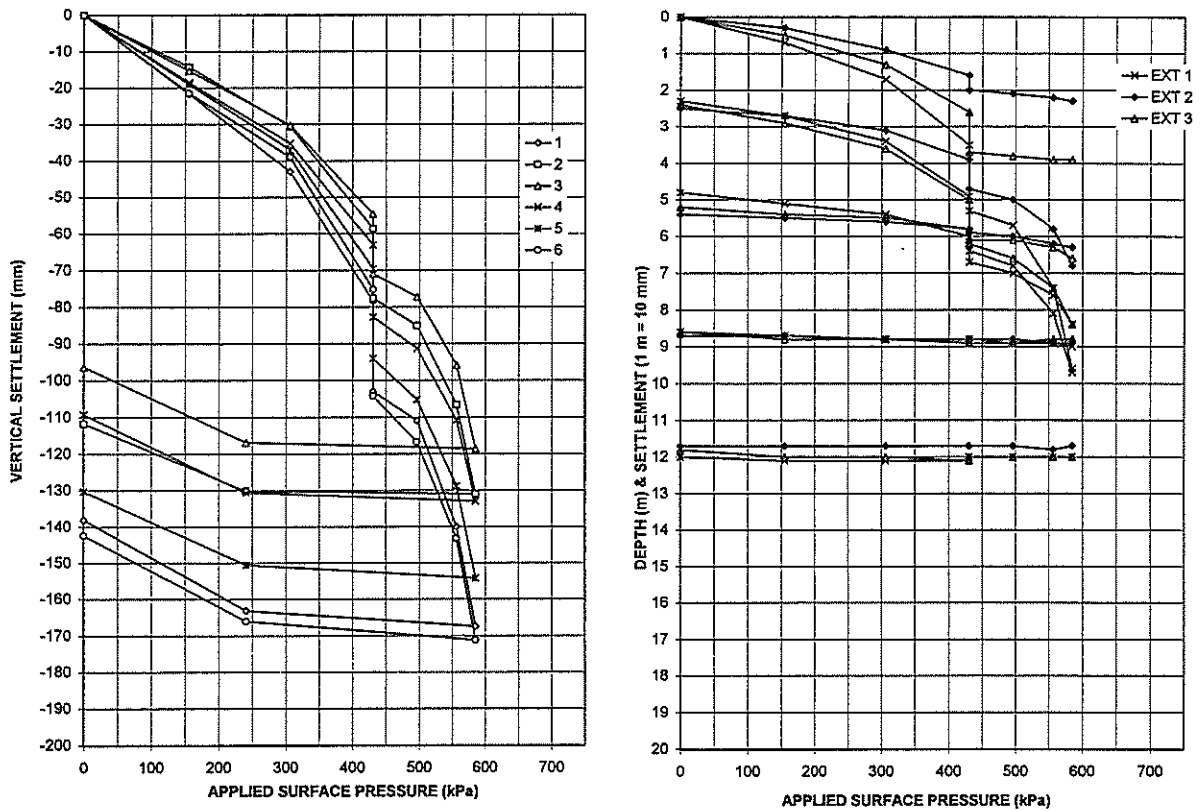


Figure 2 Loading frame settlement and magnetic extensometer settlement for the final load test.

The final test was loaded to only 585 kPa, and resulted in maximum surface settlements of about 150 mm. Extensometer targets indicated that only 60 mm of the total 150 mm settlement was attributable to soil deformation below 2 m depth.

ASSESSMENT OF DEFORMATION PARAMETERS

The methodology adopted for the settlement analyses resulted in a set of site-specific parameters which could be applied to other locations within the site where load testing has not been performed. In particular, the piezocone parameters total cone resistance (q_c) and undrained shear strength (s_u) were used to assess undrained and drained elastic moduli for sands and clays, such that a piezocone log could be used to assess soil deformation properties at other locations around the site.

Methodology

Elastic moduli relationships were assessed based on extensometer and piezocone data, and involved analysis of the load test data. An iterative procedure was adopted for each test which gradually fitted equivalent elastic parameters to the soil layers. The methodology adopted was proposed by Poulos (personal communication), and involved the iterative use of strain influence factors, with actual measured strains used to correct calculated strains. Initially an elastic analysis was performed for a uniform material underlying the applied surface loading, and the vertical strain distribution assessed. Influence factors for each soil layer (whose depth and thickness was defined by the actual location of the extensometer targets) were calculated as follows:

$$I_{e(i)} = \frac{E_{(i)} \cdot \epsilon_{ave(i)}}{\rho_{surface}} \quad (2)$$

where

- $I_{e(i)}$ = strain influence factor for the i th iteration
- $E_{(i)}$ = elastic modulus used in the calculation of $\epsilon_{ave(i)}$ in the i th iteration
- $\epsilon_{ave(i)}$ = average strain of the layer for the i th iteration
- $\rho_{surface}$ = total surface settlement

For the next iteration the elastic modulus was calculated as:

$$E_{(i+1)} = \frac{\rho_{\text{surface}} \cdot I_{e(i)}}{\epsilon_{\text{real}}} \quad (3)$$

where

- $E_{(i+1)}$ = elastic modulus used for the next iteration
- $I_{e(i)}$ = strain influence factor calculated for the i th iteration
- ϵ_{real} = real strain of the layer (based on extensometer results)

The iteration procedure was repeated until the calculated values of E converged for each layer. This usually occurred after only a few iterations.

Deformation Parameters

Once the equivalent soil moduli were assessed for each layer, simple relationships with piezocone data were established. Typical published relationships between elastic modulus, E , and q_t , for sandy soils indicate a range from $E = (1.5 \text{ to } 5) q_t$. For clays, relationships between undrained elastic modulus, E_u , and s_u range from $E_u = (300 \text{ to } 1500) s_u$.

The load tests indicated that deformation of the sand units had a time-dependent component, possibly due to dissipation of excess pore pressures within thin silty or clayey lenses in the sand unit. Adopted settlement parameters for the sand soils gave an initial "undrained" secant modulus of $E_u = 2.3 q_t$, with a (24 hour) drained modulus of $E' = 1.4 q_t$. The 24 hour "drained" modulus presented is not necessarily the same as the normal drained modulus, since it was unclear whether full dissipation of excess pore pressures had occurred. However, this approach was considered to be appropriate for crane loadings of up to one day duration.

Time-dependent deformation was not observed for the clay soils during the 24 hour duration of the load tests, probably due to the very low permeability of the clay units. Only the undrained secant modulus was assigned; $E_u = 900 s_u$.

For the surface slag fill, the secant modulus was found to vary, depending on the thickness of the slag fill at the particular test location. This was thought to be as a result of localised overstressing and failure of the slag particles, and also the breaking of the cementation which develops in compacted blast furnace slag over time. Thinner slag layers were also subjected to much greater tensile stresses at the base of the layer than thick layers, resulting in earlier loss of any tensile strength which the slag may have developed. The slag moduli was assessed to range from 15 MPa for 0.6 m thick slag to 85 MPa for 1.3 m thick slag. The value of 15 MPa was considerably less than would normally be expected, and was likely to be due to the non-linear behaviour of the material combined with the high stresses the material had been subjected to in the load tests.

Layer Equivalencies

When using continuous penetrometer data for assessment of elastic soil parameters it was important to use the harmonic mean, rather than a simple average, since deformation is inversely proportional to the elastic modulus. Therefore thin compressible layers had a much greater effect in reducing the equivalent modulus of a soil profile compared to the effect of thin incompressible layers increasing the equivalent soil modulus. The equivalent elastic modulus was calculated as:

$$E_{\text{equiv}} = \frac{t}{\sum_{i=1}^n (t_i/E_i)} \quad (4)$$

where

- E_{equiv} = equivalent elastic modulus for the soil layer whose thickness is t
- t = soil layer thickness
- t_i = thickness of soil sub-layer i
- E_i = elastic modulus of soil sub-layer i
- n = number of soil sub-layers within the soil layer

Where soil sub-layer thickness is uniform, such as when using digital data from electric penetrometer tests, the harmonic mean was simply calculated using a spreadsheet program, and Eq. (4) can be simplified to:

$$E_{\text{equiv}} = \frac{n}{\sum_{i=1}^n (1/E_i)} \quad (5)$$

PREDICTED HEAVY LIFT CRANE SETTLEMENT BEHAVIOUR

Based on the back-analysed parameters presented in the previous section, the initial settlement of the rear crawler of the heavy lift crane was calculated to be between 85 and 100 mm, with the settlement over one day expected to increase to between 110 and 150 mm.

Bearing capacity was addressed using the methodology presented by Meyerhof [6]. Effective friction angle of the upper sand soils was assessed based on a relationship presented by Lunne and Christoffersen [5] which uses the piezocone parameter q_c . Back-analysis of the load tests indicated that an effective friction of at least 33° was appropriate, since bearing capacity failure had not occurred. A design effective friction angle of 35° was adopted.

Bearing capacity analyses indicated that Factors of Safety of less than 1.0 would occur for the heavy lift crane where the very soft clay layer occurred at only three metres depth. Various remedial foundation treatments were designed, based on the soil parameters derived in the load test program.

CONCLUSIONS

Large scale plate load tests were used to assess soil deformation properties when subject to high vertical stresses. Magnetic extensometers provided a useful tool in assessing deformation properties of different types of soil, and, when combined with piezocone test data, allowed site-specific parameters to be established, such that a piezocone log could be used to assess soil deformation properties at other locations around the site.

ACKNOWLEDGMENTS

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