

Gibson Island Wastewater Treatment Plant - Foundation Design

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SUMMARY The Gibson Island Wastewater Treatment Plant in Brisbane is founded on up to 15 m of soft foundation soils. Preloading was used to achieve adequate foundation bearing capacity for the larger structures. The paper describes the preload design and performance together with structure features.

1 INTRODUCTION

The Brisbane City Council has almost completed (November 1987) the construction of Stage I of a Wastewater Treatment Plant at Gibson Island. Foundation materials consist of dredged sand overlying interbedded sands and very soft clays up to 15 m thick.

Detailed foundation investigations, laboratory testing and analyses indicated that the foundation material could be adequately strengthened by preloading to support the structures. This paper describes the investigations and analyses carried out, presents results of the preload monitoring and discusses the design of the structures relative to the foundation conditions.

2 PROJECT DETAILS

The treatment plant at Gibson Island is designed to replace the overloaded and economically inefficient treatment plants of the Bulimba Creek and Tingalpa Creek catchments and provide further capacities for some of the fastest developing areas of south-east Brisbane.

The plant is being developed in two stages each of 150 000 EP capacity, with each stage being developed in two modules. The first module of Stage I will be commissioned in February, 1988. The construction of the second module has already commenced and is expected to be completed by the middle of 1989.

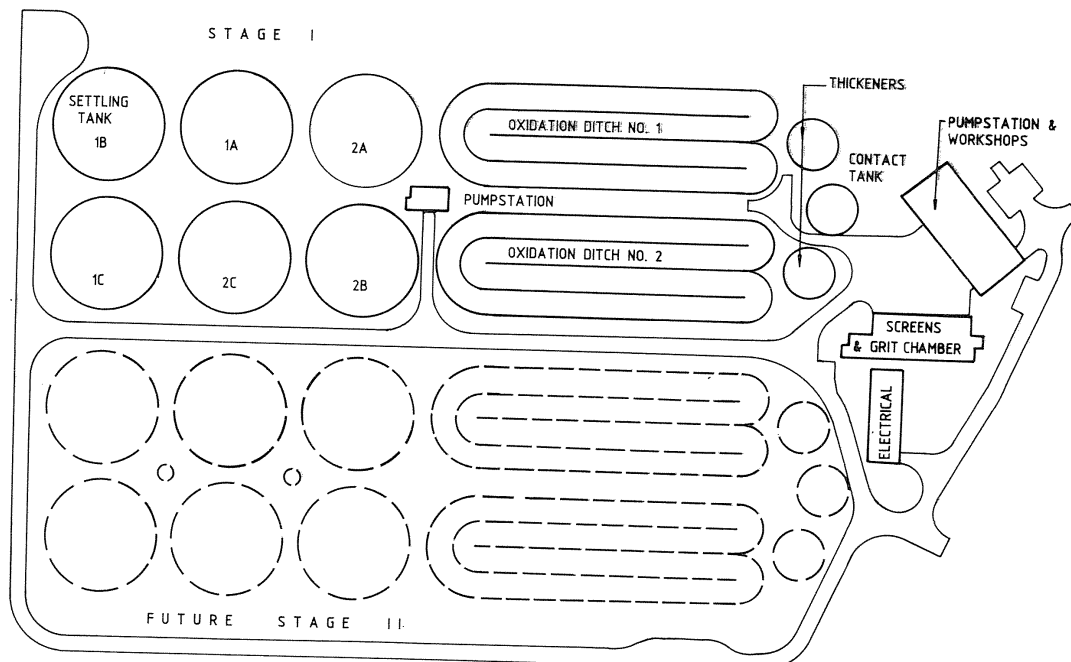


Figure 1 Plant Layout

The 12 hectare site is restrictive in area and is geotechnically difficult. On the positive side, the site is environmentally most suitable because it is adjacent to the Brisbane River where the effluent can be effectively discharged with minimal impact into a large body of water and is on an island zoned for noxious industry.

Plant layout (Figure 1) shows the following two groups of structures:-

GROUP 1 located at the head of the plant and comprises buildings housing: coarse screens, pumps, rotary screens, grit removal channel, blowers, compressors, switchgear, office and laboratory with amenities, sludge belt filter presses, one 15 m diameter contact tank and two 15 m diameter picket fence thickeners.

GROUP 2 comprises -

- Four aeration tanks designed as rectangular structures 127.4 m long and 36.7 m wide with semicircular ends and constant depth of 4.5 m.
- Twelve final settling tanks - 40 m diameter and 4.6 m deep.
- Returned activated sludge pump building designed as reinforced concrete structure with an irregular shape.

3 FOUNDATION CONDITIONS

3.1 General

The site is a reclaimed area of mangrove mud flats on an island near the mouth of the Brisbane River. The natural surface level (1985) varied between 1.0 m and 3.5 m AHD.

3.2 Field Investigations

An initial investigation programme was carried out by the BCC Materials Section in 1984. The work comprised fifteen boreholes and four electric friction cones at the locations shown on Figure 2.

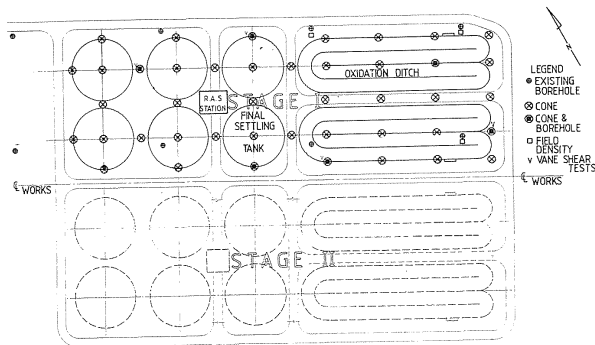


Figure 2 Field Investigations

The following detailed investigation programme (see Figure 2) was put in hand to obtain data for preload design:

- o 43 electric friction cone probes extending to about 20 m depth.
- o Continuous 100 mm diameter undisturbed piston sampling at 6 locations. This work was undertaken to obtain high quality

samples for laboratory testing and for correlation of the cone penetration tests with the actual subsurface soil profile.

- o Vane shear testing at four locations to develop strength profiles.
- o Groundwater levels were measured in standpipes installed at six locations.
- o Field density tests were carried out at 3 locations (a total of 7 tests)
- o A detailed laboratory test programme.

3.3 Soil Profile

The subsurface soil profile consisted of the following distinct units extending from the surface downwards:

a. Fill:

Previous reclamation of the land was by deposition of hydraulic fill dredged from the Brisbane River. It comprises coarse to medium, light brown sand varying in thickness from 1 to 3 m.

The site is adjacent to Bulimba Creek Power Station and ash fill overlies the sand fill in some areas.

b. Younger Alluvial Sequence:

This sequence comprises 11 to 15 m thickness of loose to moderately dense sand with closely spaced interbeds of very soft to firm clay and sandy clay. The thickness of the compressible clay strata varies markedly from place to place. The close interbedding makes it difficult to interpolate between holes/probes.

c. Older Alluvial Sequence:

This sequence comprises stiff to hard sandy clays and dense to very dense sands with a fine to coarse gravel basal layer of variable thickness. The upper surface of this sequence was found to vary from RL -10.5 to -15 m AHD.

d. Bedrock:

Sandstone and basalt bedrock occurs at elevations varying from RL -31 m to -45 m AHD.

3.4 Groundwater Levels

Groundwater levels were not unduly affected by tidal movement. However, they do vary seasonally and a maximum value of RL 0.88 m AHD (in the range RL 0.09 to 0.88 m) was measured.

4 GENERAL FOUNDATION CONSIDERATIONS

The design decisions related to structural supports were partly made on the experience obtained from the Luggage Point site located on the opposite side of the river four (4) km downstream where nine years ago a considerably larger wastewater treatment plant was constructed.

High compressibility of the soils there precluded the use of any shallow foundation. Consequently piles driven to the depths of 30 to 35 m were used for all structures except the sludge drying beds (which subsequently sank in some sections by more than 240 mm).

Furthermore, considerable difficulties were experienced in containing the lateral forces in the rectangular liquid retaining structures. The large size of these structures demanded provision of a complex system of joints to relieve strains produced by temperature shrinkage and uneven foundation settlements without loss of lateral structural integrity. As a result the whole foundation system and particularly the floors, designed as suspended slabs, became difficult and expensive to build.

Therefore, when the investigations carried out by GHD Wood Pty Ltd indicated that the compressibility of the soils at Gibson Island site was considerably lower than at the Luggage Point site and that the preloading can be successfully effected, the idea was considered structurally rather attractive.

However, it was decided that only the structures of GROUP 2, that is, the structures with essentially uniform distribution of the loads could effectively be supported on shallow compensated foundations constructed on a preloaded site. All the structures within GROUP 1 were to be supported on piles.

The comparative cost study subsequently made of piled foundations versus mat foundation showed that savings of the order of \$2 000 000 could be achieved if the cost of preloading was kept within the estimated \$10/m³.

The Council first approached the Harbours and Marine State Government Department to negotiate pumping sand from the Aquarium Passage across the road from the site. The costs were favourable but the volume of deposits of available sand were not. As a result the overburden material from the nearest Council's quarry was used at an overall cost to deliver, handle and dispose of, of the order of \$11.60/m³. The volume of imported fill was of the order of 160 000 m³.

Notwithstanding the economy, piled foundations were still preferred because of the earlier indications that the time required to preload the site could not be accommodated by the construction programme. It was only when 120 days of preloading was estimated to be adequate that it was finally adopted.

5 PRELOAD DESIGN

5.1 Design Intent

The design intent was to preload the foundation so as to achieve foundation settlements (at 90% consolidation) equal to the sum of primary and 25 years of secondary settlement due to structure loads.

In order to minimise preload levels and quantities the bases of the structures were founded at a minimum base level of RL 1.0 AHD (0.12 m above maximum measured groundwater level). Foundation design loads were 55 and 48 kPa for the oxidation ditches and settling tanks respectively.

To further minimise fill quantities the preloading was carried out in two stages.

An extensive field monitoring programme was included to measure settlements, pore pressures

and consolidation of the foundation under the preload in order to check design assumptions.

5.2 Settlement Magnitude

The results of the laboratory consolidation tests are summarised in Table 1. Mean values were generally adopted for design purposes. A somewhat higher value was taken for the coefficient of secondary consolidation because of recent experience on other jobs. The tests also indicated that the soft clays were approximately normally consolidated at a ground surface level of RL 2.0 AHD.

TABLE 1
SUMMARY OF CONSOLIDATION TEST RESULTS

	No. of Results	Range	Mean	Adopted
			Value	Value
Cc/1+e ₀	7	0.11-0.27	0.20	0.20
Ce/1+e ₀	6	0.030-0.058	0.051	0.05
C	15	0.0044-0.014	0.008	0.01
Cv m ² /year	6	0.38-1.30	0.76	0.76

Cc = Virgin Compression index
 Ce = Rebound Compression index
 C = Coefficient of secondary consolidation
 e₀ = Void ratio at preconsolidation pressure
 Cv = Coefficient of compressibility

In-situ densities, required to estimate the in-situ stress state, were obtained by sand replacement tests in the sand fill and from undisturbed samples at lower levels. Adopted design values are shown in Table 2. A value of 1.8 tonnes/m³ was assumed for the preload fill.

TABLE 2
DESIGN DENSITIES

Material	Density (tonnes/m ³)	
	Moist	Saturated
Sand Fill	1.70	2.00
Soft Clay	1.60	1.65
Loose Sand	1.70	2.00
Stiff Clay	1.80	1.85

Settlement of clay layers was calculated in three stages as follows:

- (i) Immediate Settlement was calculated assuming an undrained modulus equal to 300 times the undrained shear strength (see Figure 3) and linear elastic behaviour.
- (ii) Settlement due to Primary Consolidation was calculated using the usual one dimensional formula with $\frac{Cc}{1 + e_0}$ equal to 0.20.
- (iii) Settlement due to Secondary Consolidation was calculated for two log cycles of time estimated as equivalent to about 30 years of project time. The settlement was taken as proportional to thickness of the soft layers and log time.

An estimate was also made of the instantaneous settlement of the loose sand layers but this was relatively small.

The settlements of the structures were calculated and preload designed to achieve the sum of

primary and secondary settlements at 90% consolidation. Preload levels of RL 8.0 m and 7.0 m AHD were determined for the oxidation ditch and settling tank areas respectively.

5.3 Period of Preloading

The time required for 90% consolidation is notoriously difficult to predict from laboratory test data. Hence, settlement data available from the Fisherman Islands Grain Terminal (GHD-Wood) and the Brisbane Airport (Department of Housing and Construction) were used to obtain time estimates. These indicated times to 90% consolidation of 70 to 100 and 90 to 120 days respectively. This is roughly equivalent to a maximum clay layer thickness of 2 m (using the C_v value in Table 1) which is reasonable. Hence a time to 90% consolidation of 120 days was adopted.

5.4 Stability Considerations

The foundation strengths for stability analyses were based on field testing. A friction angle of 28° was adopted for both the sand fill and alluvial sand based on a Standard Penetration Test "N" value of 4.

Shear strength of the alluvial clay was based on vane shear tests reduced by a Bjerrum factor of 0.85 based on a PI of 40. Details are shown on Figure 3.

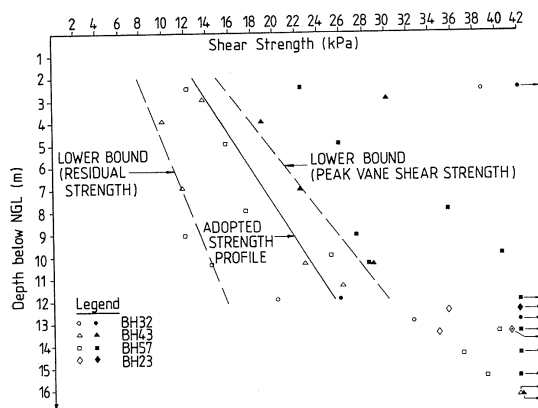


Figure 3 Vane Shear Strength Profile

A friction angle of 35° was adopted for the preload fill.

The overall stability of the preload fill was determined using the computer programme "STABL". Side slopes of 3.0 H:1 V and 3.5 H:1 V were required to achieve a factor of safety of 1.2 for fills with a top level of RL 7.0 m and 8.0 m AHD respectively.

It was also necessary to eliminate local shear failure within the foundation soils to avoid remoulding of foundation soils and to enable proper assessment of settlement measurements. Local shear stresses in the foundation were determined for a variety of side slopes using the curves produced by Jorgenson (1934) and compared with available shear strength. Details are shown on Figure 4. This gave significantly flatter slopes than the overall stability considerations and resulted in the following side slope geometries being adopted:

Preload Fill Slope 4H:1V to RL 3,
level RL 7.0 AHD: 8 m wide berm at RL 3

Preload Fill Slope 4.5H:1V to RL 4,
level RL 8.0 AHD: 11 m wide berm at RL 4

6 PRELOAD PERFORMANCE

6.1 Instrumentation

The following instrumentation was installed to monitor foundation behaviour:

Instrument Type	Number Installed
Settlement Plates	42
Magnetic Extensometers	3
Pneumatic Piezometers	14
Standpipe Piezometers	6

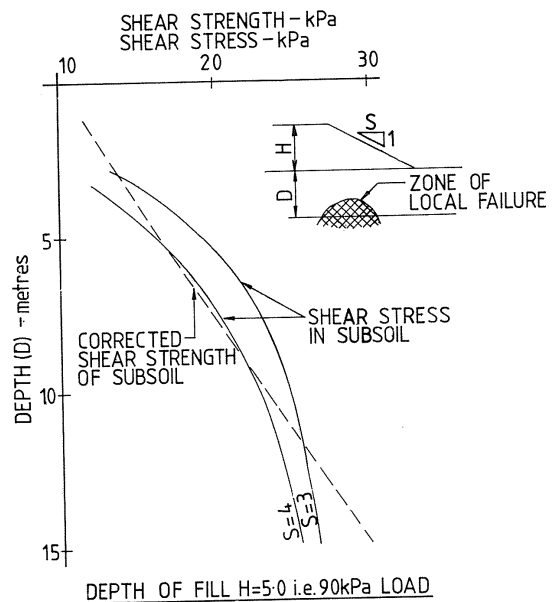


Figure 4 Local Shear Strength Considerations

The locations of the instruments are shown on Figure 5. Settlement plates were installed at RL 0.5 m AHD. Holes drilled to install piezometers were continuously logged and each piezometer located at the centre of a clay layer as far as practical.

Monitoring of preload performance was carried out at approximately weekly intervals.

6.2 Settlement Plates

Measured settlements from the 42 plates were plotted against time and 100% consolidation estimated using the (Asaoka and Suzuki, (1979)) method. A typical example is shown on Figure 6. Using these data percent consolidation was calculated for various times from end of fill placement. These are plotted in Figure 7 and indicate that there is a 99% probability that 90% consolidation occurred with 97 days (i.e. slightly less than the 120 days predicted). Preload removal commenced 97 days after placement was completed.

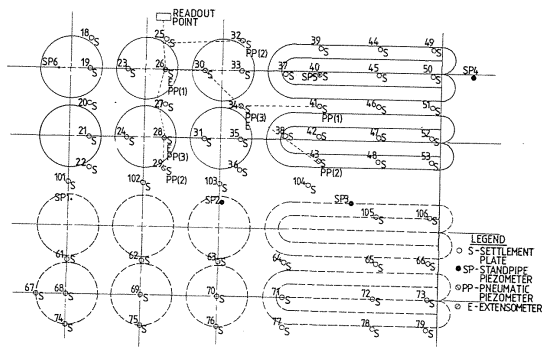


Figure 5 Layout of Instruments

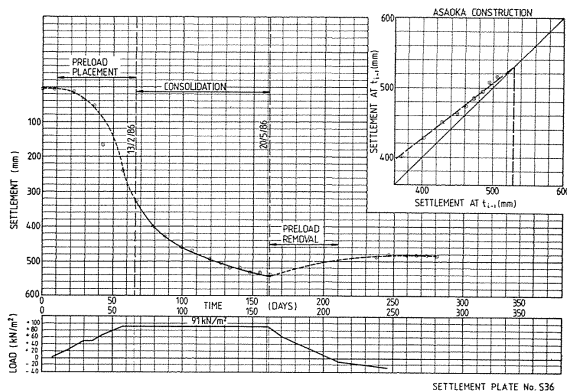


Figure 6 Typical Settlement Plots

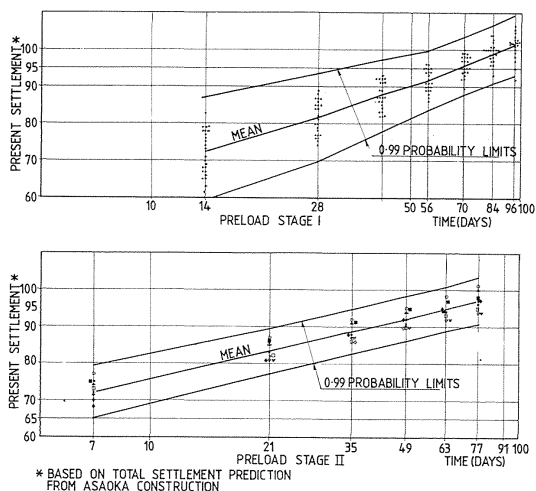


Figure 7 Graph of Percent Settlement Vs Log Time

The measured immediate plus primary settlements ranged from 200 to 770 mm. Predicted values are plotted against actual settlements on Figure 8. The accuracy of prediction was about $\pm 30\%$.

Measurements taken during unloading indicate that the rebound ranged from 5 to 15% of the immediate plus primary settlement, averaging about 10%. This is considerably less than the 25% predicted using the rebound compression index obtained from laboratory tests (see Table 1). The modulus of subgrade reaction ranged from 1 600 to 3 000 kN/m^3 which agreed well with the recommended design value of 1 500 kN/m^3 .

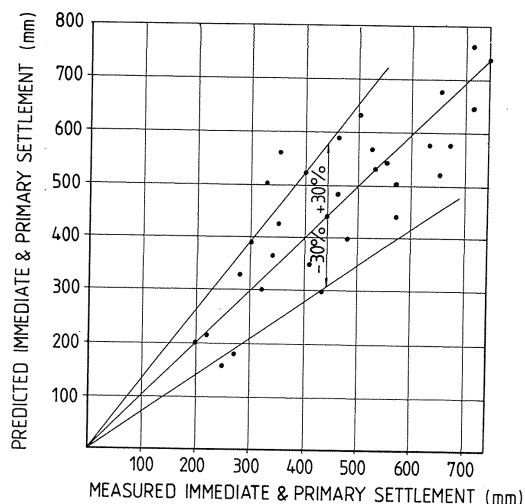


Figure 8 Predicted Vs Measured Settlement

6.3 Magnetic Extensometers

The ASAOKA method applies to single clay layers only, not the multi-layered system at Gibson Island.

In order to better estimate the time required for 90% consolidation extensometers were installed to measure consolidation of 2 m thick layers of the foundation. Compression between magnets (spaced at 2 m centres) ranged from 12 to 143 mm. Asaoka plots indicated that consolidation was at least 90% complete in all 2 m layers.

6.4 Pore Pressures

Pore pressures were measured at 14 locations. Typical plots are shown on Figure 9. Pore pressure increases due to the preload ranged from 5 to 40% of the applied load.

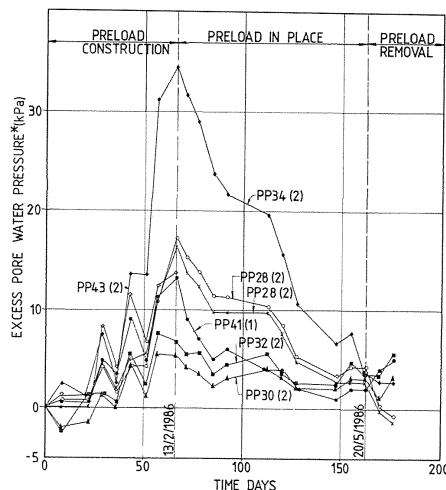


Figure 9 Summary of Pore Pressure Measurements

From one dimensional consolidation theory the theoretical pore pressure at the centre of the consolidating layer for 90% average consolidation would be about 15% of the applied load. Immediately prior to removal of the preload all

pore pressures were less than 15% of the applied load with most being zero within the accuracy of measurement.

7 FOUNDATIONS ON PRELOADED SOIL

The adopted foundation system for preloaded soils is not truly generic to this class of structures and geotechnical conditions. The more normal system would have been a stiff raft foundation with the capacity to bridge over softer areas. However, such a foundation, if adopted, would have been complex and expensive and probably economically inferior to piled foundations.

Consequently the following design decisions were made:

- (i) The stresses generated on the structure-soil interface not to exceed allowable soil stresses.
- (ii) The soil resistance to sliding, rotation and torsion not to be accounted as a factor assisting stability of structures.
- (iii) The walls of circular structures to be a continual shell supported on mat foundation designed as a glorified slab on ground thickened in the region of the wall base. Such a slab constructed without joints was to have sufficient stiffness to bridge over local soft zones and still have adequate flexibility to cope with gradual changes of the foundation modulus more likely to occur on this site.
- (iv) The rectangular, semicircular or other shaped footing/floor slabs to be provided with minimum joints. The lateral stability of walls to be maintained with special floor joints designed to restrain lateral movements of the monoliths.
- (v) Foundation levels to be set above the water table so that the footings and floors are protected from effects of uplift or intermittent saturation of underlying soils. This, also, allowed the footings and floors to be constructed with the minimum use of dewatering pumps.
- (vi) The structures to be sufficiently separated so that the soil stress and settlement interface between the adjoining structures is minimised.
- (vii) Nonuniform settlement can cause distortion and cracks in underground pipes. In severe cases the slope of the pipe can be lost or reversed. To prevent damage from excessive differential settlement sufficient pipe length or a special flexible joint was provided between structures to ensure that any movements can be accommodated by the pipe's overall or local flexibility. In addition pipes were made of flexible materials.
- (viii) Due to high permeability of the underlying soils floor and wall underdrains were considered unnecessary.

8 BEHAVIOUR OF STRUCTURES

At the time of writing there were no signs of structural distress or of irregular behaviour of

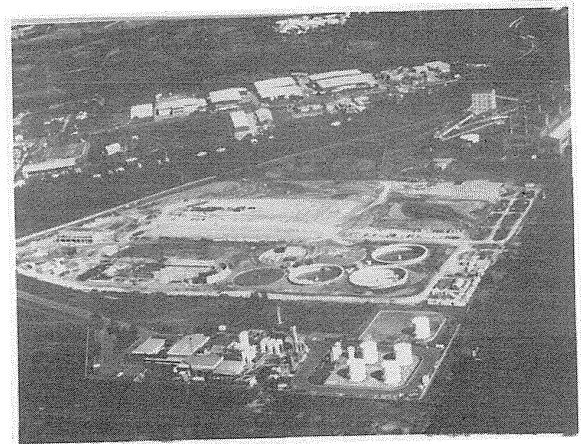
the completed structures. This was to be expected since the foundations at the construction stage are considerably understressed. The intended post construction monitoring of the structures should reveal any structural problems. However reliable information will become available only after the structures have been subjected to a prolonged full loading.

9 CONSTRUCTION

In order to construct the oxidation ditches and settling tanks it was necessary to remove the preload and excavate to RL 1.0 AHD. Soft clay was exposed over part of the site at that level and the water table was at or just below RL 1.0 AHD in many areas. Hence care was necessary to avoid remoulding the clay on exposure.

A full description of construction methods is beyond the scope of this paper. However the following precautions were adopted:

- o General excavation methods were discontinued at RL 0.5 AHD.
- o Dewatering was carried out where required and all pipes below the structures carefully installed in trenches.
- o Excavation to 100 mm below base level was carried out in strips 5 m wide using a small bulldozer with low ground pressure tracks. This strip was protected with 100 mm of concrete.
- o Reference points were established on concrete structures as soon as practical and movements monitored.



Note Stage II Preload in Background.

Figure 10 Plant Under Construction

10 CONCLUSIONS

Details have been given of design of the foundation preload for the Gibson Island Wastewater Treatment Plant. Measurements of settlements and pore pressures during preload have been shown to compare reasonably well with design estimates. It has not been possible to present data for the structures as they have not yet been subject to design loads.

11 ACKNOWLEDGEMENTS

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Thanks are also due to Mr P O'Flaherty of GHD who analysed much of the data presented herein.

12 REFERENCES

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