Heavy Structures Founded in Aeolian Soils

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SUMMARY Case histories relating to the foundations for three R.C. tanks are presented. Details of the site investigations are given. The design criteria, construction methods and the performance of the treated soil foundations are recorded.

1 INTRODUCTION

The treatment of the foundations for tanks at Waikerie, Woolpunda and Loveday provide the basis for these case histories. The tanks are all close to the River Murray in South Australia and are founded on "fossil" sand dunes. The locations of the tanks are shown in Fig. 1.

In the past the practice has been either to found at the surface or to use reinforced concrete piles to carry the structural loads through the aeolian soils. The former solution was not always successful, and the latter, although sound, was not necessarily the most economical.

In order to understand the behaviour of these soils, a study was made of the local geology and aeolian landforms.

2 RELEVANT GEOLOGY OF THE REGION

The aeolian soils of the Murray Basin of South Australia, are derived from sedimentary marine deposits laid down, during the Tertiary period. During this period limestones were formed, visible in river cliff formations downstream of Morgan.

Next an impervious layer of marl was laid down followed by the bright yellow Loxton Sand, typically 16 m thick. These unconsolidated sands were laid down in estuarine conditions.

More sands overlie the Loxton Sand. These occur as sandstones, sandy limestones and calcareous sands and were deposited in an estuary which more or less follows the present course of the Lower Murray. They are visible in cliff exposures at Waikerie and are found as partly consolidated sands in the river cliffs upstream of Loveday.

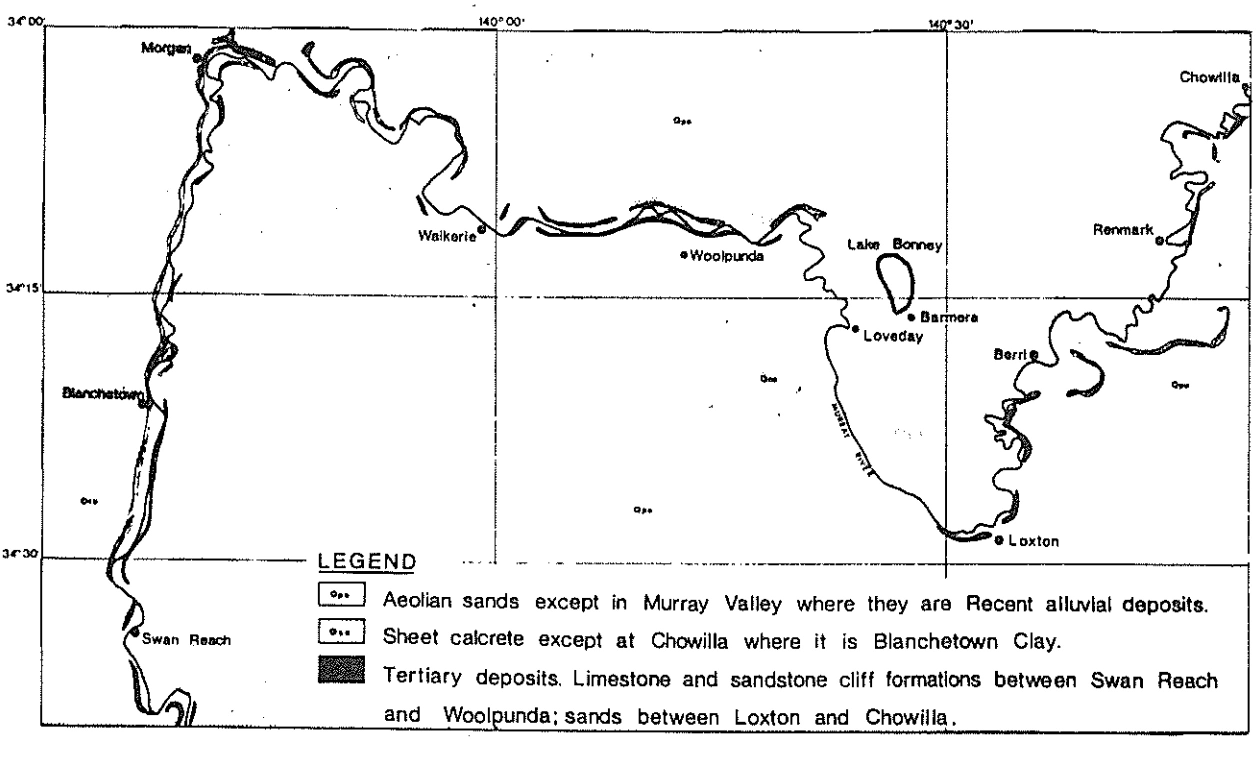

Figure 1 - Location plan and broad geology

Later, in the Quaternary, clays and limestones were formed as lake deposits. The eroded surface of these deposits is overlain by dune sands and pebble conglomerates derived from them and older rocks. This is the present-day blanket of loess and associated calcrete which forms the foundations for the Departmental tanks.

3 DESIGN TASK

3.1 The Structures

The dimensions of the 1.25 ML elevated tank at Waikerie and Woolpunda and of the surge tank at Loveday are shown in Fig. 2.

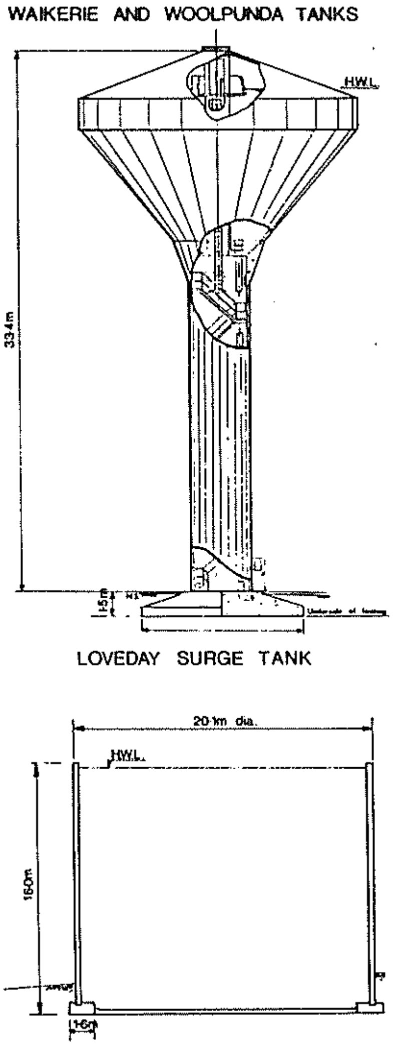

Figure 2 - Tank dimensions

3.2 Soil Loading

The load diagram for the Waikerie and Woolpunda Elevated Tanks is shown in Fig. 3.

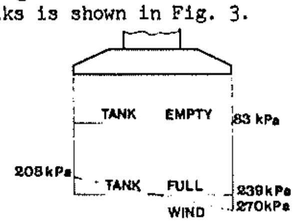

Figure 3 - Load diagram

3.3 The Foundation

It was realised that, should the foundation for any of the three structures become partly or completely wet then collapse of the open-structured soil would be inevitable. The foundation treatments considered were to:

Prevent the soil from becoming wet,
Excavate and recompact with fill for a selected depth,
Use concrete or steel piles,
Form sand or gravel piles,
Densify the soils by Vibroflotation,
Construct stone columns by Vibro-compaction,
Pre-load the site.

4 WAIKERIE ELEVATED TANK

4.1 Site Investigation

Typical soil profile and test results are given in Fig. 4.

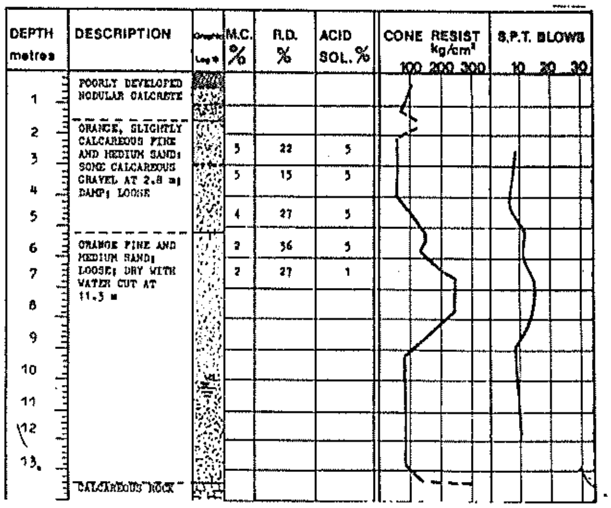

Figure 4 - Soil profile and test results

4.3 Design

Settlement predictions, based initially on the static cone soundings, gave 46 mm total settlement. The soundings showed the ground to be uniform. Hence, with the rigid tank base and small wind loading component, the differential settlement was expected to be small. Initially the natural soil foundation was therefore considered acceptable.

The inspection of the 1.5 m deep excavation for the foundation revealed aeolian deposits. A pit, then a shaft, were dug to obtain soil densities and determine the degree of cementation of the sand. This revealed how loose and also how weakly cemented the sand foundation was.

Plans to provide an adequate foundation by preventing the soil from wetting-up were therefore scrapped as the soil in its dry state was considered insufficiently cemented to prevent excessive settlement under the tank load.

Further deeper exploration was then done, the Standard Penetration Test being used to define the sands density. Low densities were found to 12 m with a small increase at 6.4 m. A rocky layer was found at 13.4 m.

Cost estimates for excavation and recompaction, cast-in-situ piles and sand piling came out in favour of sand piling and a design, based on this method, was prepared.

The design aim was to produce a mass of compacted sand, 15 m in diameter and 5 m thick beneath the footing. A triangular grid of piles, spacing 1.7 m, was used. The Contractor was required to displace 650 kg of sand per metre of pile into the ground. This "method" specification, was used as an alternative to requiring the Contractor to work to a performance specification.

The specification called for the first five piles to form a trial to ascertain whether the pile spacing or the amount of sand to be displaced should be altered.

The predicted settlement for the sand piled foundation was 50 mm. It was expected that this would be derived largely from the soil below 6.4 m.

4.4 Construction of Foundation

The sand piles were placed from a 380 mm tube which was driven percussively with a 3 tonne hammer to 6.4 m. The Contractor elected to backfill the excavation and form the piles from 1 m above foundation level, thereby ensuring the piles immediately below foundation level would be heavily compacted. The sand was hammered out of the tube as it was withdrawn, the Contractor being required to pull the tube in 250 mm increments. The first five piles, which formed the trial, indicated that the pile spacing and the amount of sand displaced were adequate to achieve the required sand density.

During the work a general tightening up of the sand occurred ahead of the piles being formed, particularly below a depth of 5.5 m. Piles became increasely hard to drive. Because of this the driving depth, for all except the perimeter piles, was reduced to 5.5 m.

The treated ground was extremely tight. Standard Penetration Test values in it ranged from 32 at 1.8 m to 100 at 6.5 m.

4.5 Performance of Structure

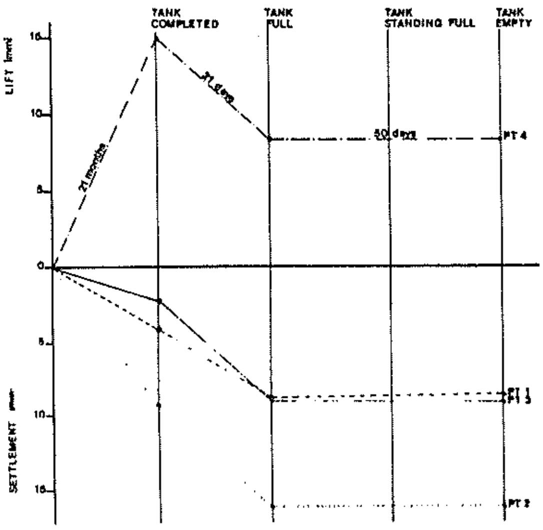

Figure 5 - Settlement data for Waikerie Tank

Settlement data for each loading stage is given in Figure 5. The measurement points are at the quarter points on the perimeter of the base and number consecutively around it.

These measured settlements imply a mean settlement of 5 mm with an overall tilt of 10 mm.

5 WOOLPUNDA TANK

5.1 Site Investigation

Typical soil profile and test results are given in Figure 6.

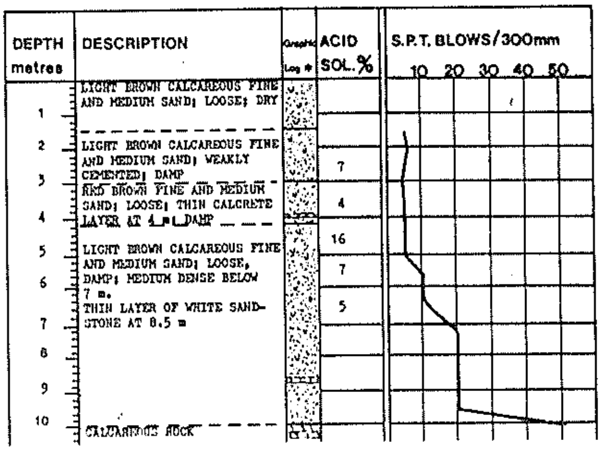

Figure 6 - Soil profile and test results

The soil investigation, confirmed the site as the crest of an aeolian dune. The sands were loose and weakly cemented. Sand densities started to improve at 7.5 m and a hard layer was hit at 9.5 m. Sand sizes to 6.5 m depth were typically 35% medium (200 to 600 microns) 50% fine (60 to 200 microns) and 15% less than 60 microns (with 8% less than 2 microns).

5.2 Design

Cost estimates based on foundations of concrete or steel piles, excavate and recompact and sand piling again came out in favour of sand piling. A treated volume of 15 m dia. by 4.7 m thick beneath the footing was selected.

An alternative tender based on vibro-compacted "stone columns" was submitted by one tenderer. This was cheaper than the lowest sand-piling tender. The "stone columns" were to extend from the surface to a depth of 6.5 m. Pile spacing varied between 2.35 m close to the centre of the foundation to 1.35 m at 4 m from the centre. The spacing then opened out until it became 2.08 m, 6 m from the centre.

The " stone column" design called for the removal of the top metre of the prepared foundation and its replacement with mat of compacted crushed rock. This was to ensure the adequate transfer of load into the "stone columns" and also to remove the less compact stone from the top of the columns.

It was assessed that the total settlement for the tank founded on the columns would not exceed 75 mm with a maximum differential settlement of 20 mm.

5.3 Construction of Foundation

The stone columns were selected both because they were cheaper and because the contractor had doubts whether sand piling would be completely successful in soil containing 15% less than 60 microns (silt size) and 8% less than 2 microns (clay size). It was accepted that total settlement would be greater with stone columns as the process does little to densify the natural soil between the piles. Pipework was therefore designed to allow for the settlement.

The stone columns were formed by the "Vibro-compaction" process, a 300 mm dia. x 5 m long vibro-flot being used to penetrate 6.5 m below ground surface. The vibro-flot was equipped with a 35 kVA electric motor and used up to 7 000 litres of water to form each column - which varied in diameter between 600 and 1 100 mm. An average of 6 cubic metres of 40 mm crushed rock was used to backfill each "vibro-flot crater". Density was confirmed by ensuring the vibro-flot's electric motor peaked at 75 amperes, there being an established correlation between the amperage and the density of the crushed rock forming the columns.

5.4 Performance of Structure

Settlement data for each loading stage is given in Figure 7.

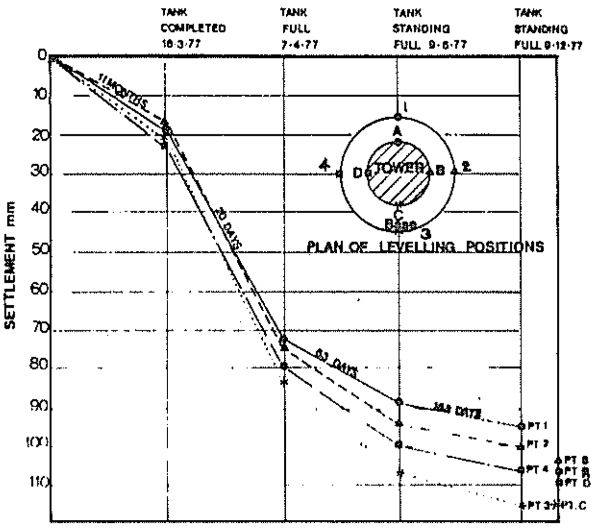

Figure 7 - Settlement data for Woolpunda Tank

6 LOVEDAY TANK

6.1 Site Investigation

Typical soil profile and test results are given in Figure 8.

The site investigation revealed very weak soils between depths of 3 and 7 m. The sands were calcareous and below 4 m and became increasingly clayey. This formation is compatible with certain aeolian landforms. A water table was intersected at 3 m, this being perched on the clay layer found at 5.5 m.

6.2 Design

A rough calculation of the settlement of the proposed tank, if founded on the natural soil, gave a figure of 250 mm under the uniformly distributed "tank-full" loading of 160 kPa. This was not acceptable, it being necessary to limit the settlement to 75 mm total and 25 mm differential.

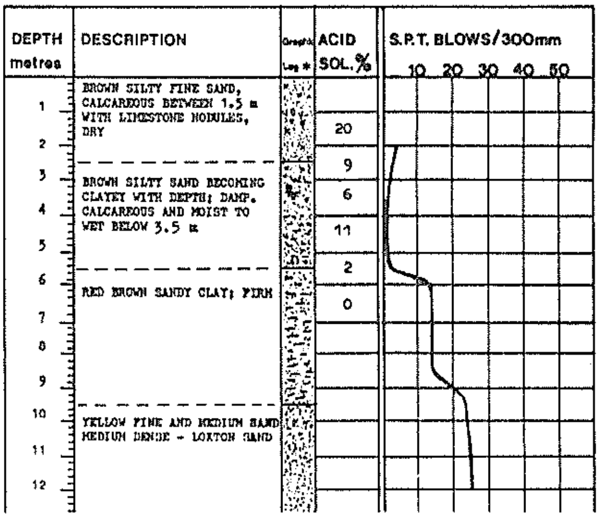

Figure 8 - Soil profile and test results

The various ground-treatment options were analysed and costed. Gravel piles constructed by the sand-piling process were initially shown to provide the most economical solution.

On receipt of tenders it was found that the number of gravel piles required had had to be doubled; also although the contractor expected the settlement criteria to be met he could not guarantee the differential settlement would be within the tolerance specified. Alternative foundation treatments were reconsidered. A causeway was to be constructed close to the tank site. It was found that a pre-load solution could be made economical by temporarily diverting the material destined for a causeway to the tank site. The pre-loading option was therefore selected.

To ensure total wetting of the foundation soils prior to pre-loading, the tank foundation was required to be flooded.

The shape of the rockfill pre-load is shown in Fig. 9. Also shown are the elevations and locations of the settlement measuring points.

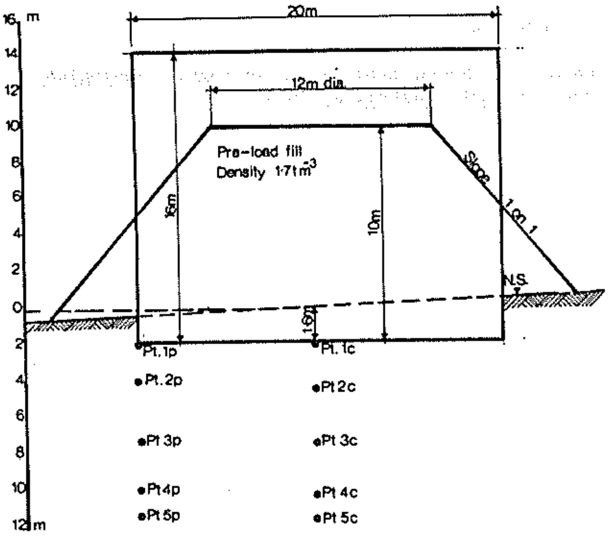

Figure 9 - Loveday pre-load and settlement points

6.3 Construction of Foundation

Construction started with the flooding of the shallow excavation. This lasted 14 days.

Sand drains were installed on a 5 m grid to accelerate the consolidation of the foundation soil under the pre-load. The complete saturation of the soil profile was confirmed during the drilling for the sand drains.

Settlement measuring points were established for three purposes.

1. To check when settlement under pre-load was effectively complete.

2. To measure ground movements at selected levels in the natural ground below the pre-load.

3. To monitor fill placement. This was done because a low safety factor had been used for the pre-load slope design and a failure of the pre-load foundation had to be guarded against.

Pre-load was placed within 3 weeks and remained for a further 6 weeks. No sign of any slope failure was observed despite using the 1 on 1 rock fill slope above the weak soils.

The surface settlements for points under the centre and the perimeter of the proposed tank are shown in Fig. 10, plotted against time and height of the preload. Ground movements below the surface are also shown.

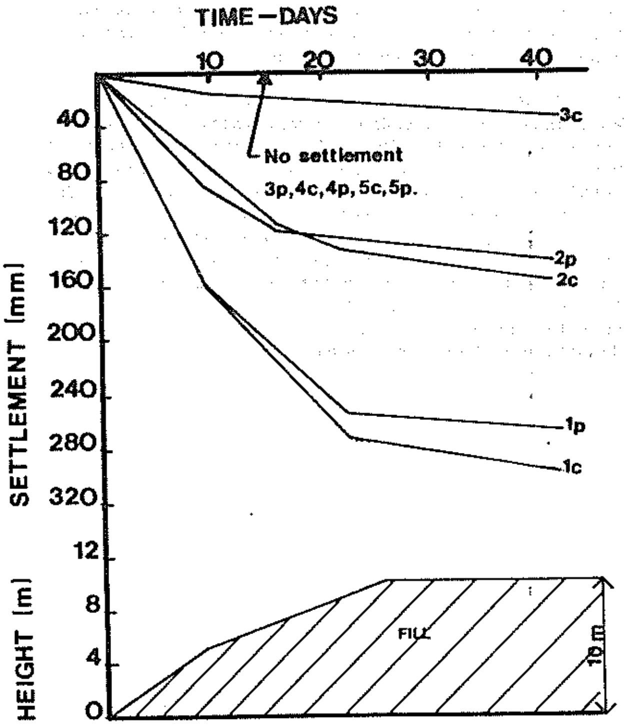

Figure 10 - Settlement data for Loveday preload

7 COSTS

The table below compares the actual costs of the treatments used with those for an equivalent reinforced concrete pile foundation (figures in brackets).

Tank	Date	Treated Area - m²	Cost $	
			Actual	1979 Basis
Waikerie	1973	177	10 000 (17 000)	28 000 (47 600)
Woolpunda	1975	177	12 000 (14 000)	20 400 (23 800)
Loveday	1979	314	15 000 (48 000)	15 000 (48 000)

It is noted that the very favourable cost of the pre-loading is probably unique to this job. This is because the intensity of loading did not require a high fill to be placed; also because the filling was so readily available costs were reduced.

8 CONCLUSIONS

The flexible approach, which considered a wide range of engineering techniques, enabled a low cost foundation to be selected.

Technically, the foundation treatments achieved their purpose. The total settlement at Woolpunda was more than forecast but would in part have been derived from settlement of the untreated ground below the piles. Deepening of the treatment could therefore have reduced the settlement. The Loveday tank has still to be constructed but its settlement is not expected to exceed 10 mm.

The interpretation of the geology of the sites was valuable when evaluating the tank sites. The probable density of the soils, the degree of cementation of the soil and the value of a potential soil (or rock) "basement" stratum could all be inferred. It was also a useful aid when planning the site investigations.

The Dutch deep sounder was not used in the Woolpunda and Loveday investigations. It was concluded from the Waikerie investigation that it was unreliable as a means of estimating settlements in these dry cemented soils. The Standard Penetration Test, although limited as a design tool, was a more reliable sounding device. The shaft at Waikerie provided valuable density information but was relatively expensive. In future work 75 mm dia. thin-walled push-tubes will probably be used to obtain an indication of soil density and degree of cementation.

9 ACKNOWLEDGEMENTS

The Director General and Engineer-in-Chief, Engineering and Water Supply Department has agreed to the publishing of the histories of these departmental structures. His permission to do this is gratefully acknowledged.

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10 REFERENCE

Handbook of South Australian Geology. Geological Survey of South Australia, 1969.