GEOTECHNICAL ISSUES ASSOCIATED WITH METHANOL STORAGE TANK DESIGN

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SUMMARY

When completing a geotechnical assessment of a tank site, the importance of observation and factual data should be realised. Data from adjacent tank sites can give a more representative picture than data from laboratory testing of how the tank will perform, especially when considering tank settlement. One dimensional oedometer test data often gives conservative data for settlement estimation, leading to over conservative foundation design. This is especially true for over consolidated soils.

INTRODUCTION

Two steel tanks were constructed for Methanex New Zealand Ltd for the purpose of methanol containment on a site comprising complex soils of volcanic origin. The tank site was selected by Methanex and Works Consultancy Services Ltd were commissioned to provide a geotechnical assessment and design the foundations and civil works for the site. The primary geotechnical issues considered as part of the project were:

- Bearing capacity
- Settlement
- Methanol containment

SITE DEVELOPMENT

Two steel methanol storage tanks, 58m in diameter and 17.5m high were constructed in September 1994 at the southern end of the Omata Tank Farm in New Plymouth (see location plan, Figure 1).

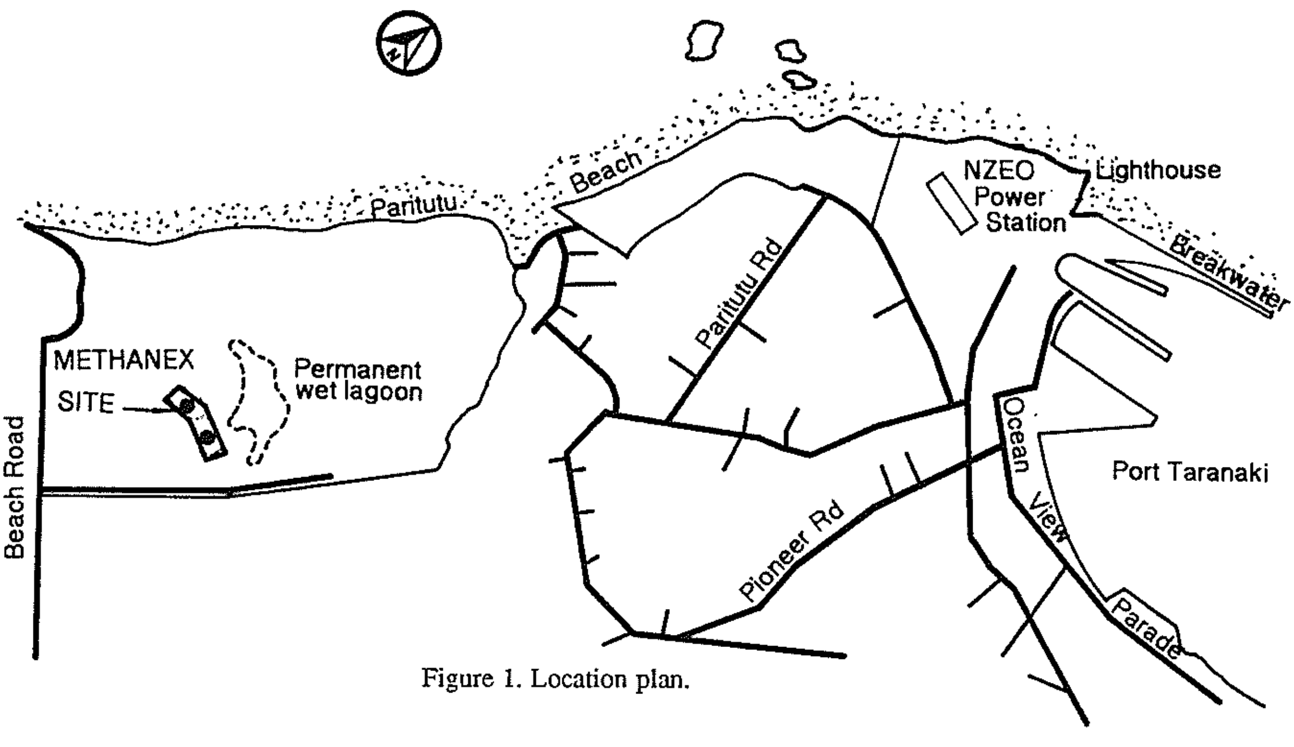

Figure 1. Location plan.

Under normal operational conditions there is a fluctuating level of Methanol in the tanks, up to 13.5m deep. The tanks were filled with water to 15.8m above the tank base during tank commissioning.

Construction of the foundation platform for the two tanks involved cut to waste, with minor excavation on the north side of the site rising to about 5m on the south side. The tank foundations comprise a concrete annular ring beam placed on a ring of compacted fill. The tanks are constructed over a HDPE liner and are surrounded by a 3m bund for the purpose of methanol containment in the case of tank leakage.

SITE INVESTIGATIONS AND LABORATORY TESTING

Investigations completed at the Methanex site comprised the following:

- Two boreholes to a depth of 45m
- Sixteen Cone Penetrometer Tests (CPTs) to refusal
- Three test pits excavated to a depth of 3.6m

Standpipe piezometers were installed in Borehole 1 and CPTs 2 and 12. The positions of the site investigations completed at the Methanex site are shown on Figure 2. Laboratory testing of samples taken from Borehole 1 and Test Pits 1 and 2 was completed.

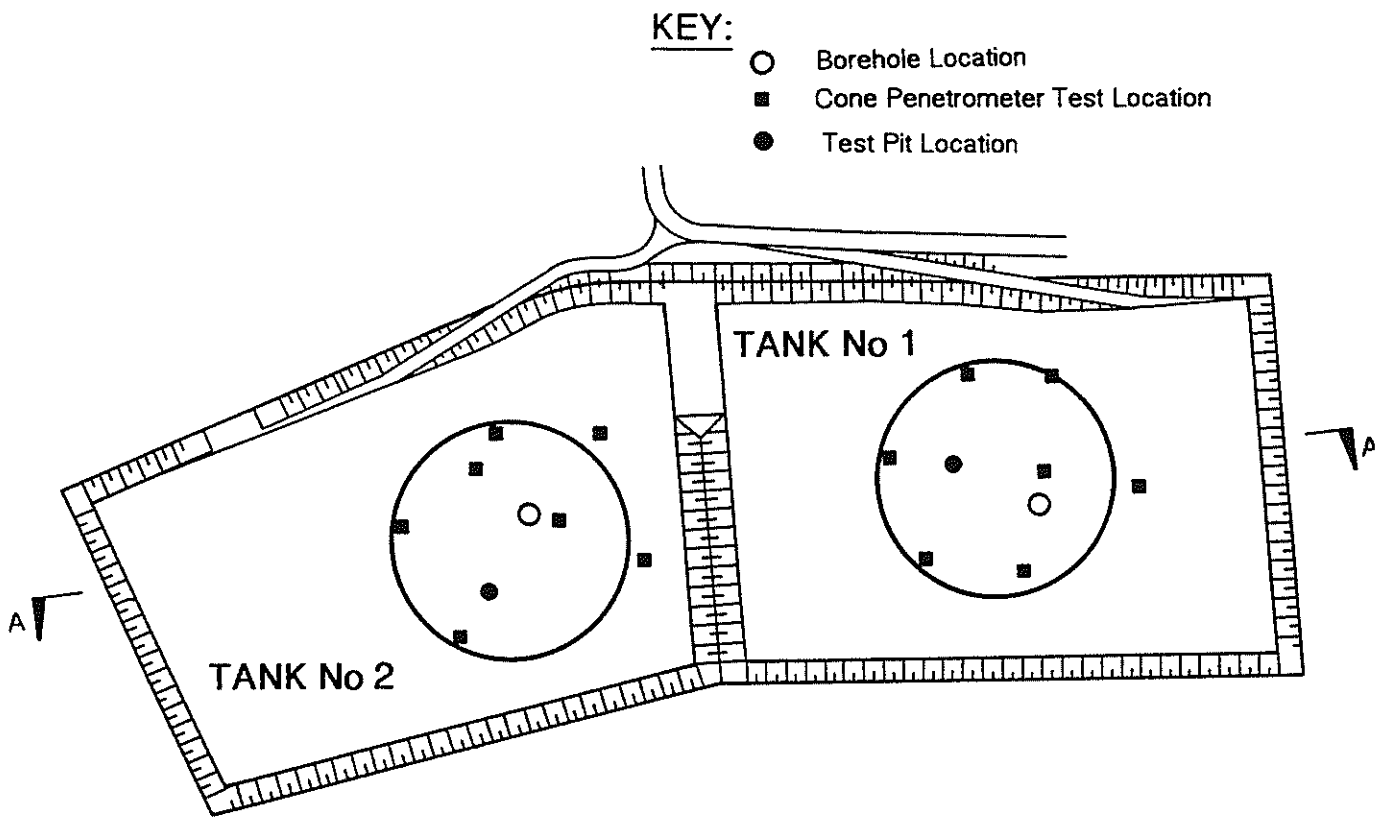

Figure 2. Site plan.

Results from site investigations and laboratory testing for an adjacent tank in the Omata Tank farm were used for the Methanex tank assessment.

SITE DESCRIPTION

The Omata Tank Farm is situated on the northern flank of Mt Egmont, south-west of New Plymouth City (Figure 1). Prior to site development the Methanex site was grassed, and sloped at a grade of approximately 1 in 20 to the north.

GEOLOGY

Material covering the site is summarised as follows:

0 - 20m	Ash deposits (a soft SILT layer was identified at 8m to 10m)
20 - 37m	Variable gravels and organic sand silt mixtures
37 - 45m	Boulders and cobbles in a gravelly silty sand matrix (Laharic agglomerate)

Within these primary material units there is some variability as demonstrated in a typical cross section through the site presented as Figure 3. A soft SILT layer which exists at a depth of 8m to 10m was identified within Borehole 1 and CPT testing indicated the layer was persistent across the site.

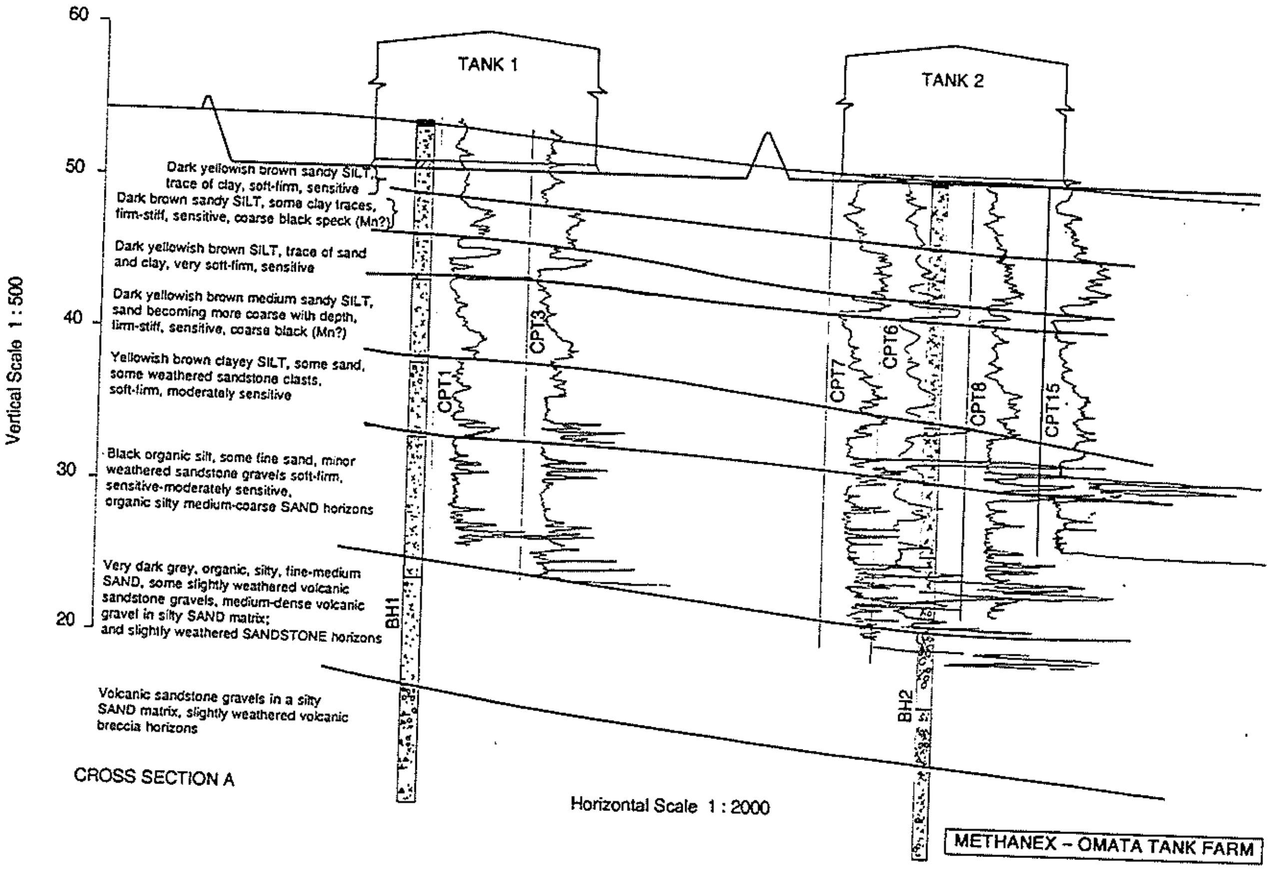

Figure 3. Cross section A-A.

The ash deposits at the site have been correlated to the New Plymouth ash group and are considered to have their origin from the Pouakai Range, to the south of the site. Generally they can be described as an orange-brown, coarse andesitic ash with a predominantly clayey silty sand texture.

A summary of material properties is given in Table 1 below.

Table 1. Summary of Material Properties

Depth (m)	Material description	Shear strength (kPa)	Atterburg results				Oedometer results for stress range of σ_o' to σ_o'+160kPa	Permeability (m/s)	
			w/c	LL	PL	PI		Water	Methanol
0 - 8	Volcanic Ash	100 (min)	80	90	65	30	m_v = 0.072 - 0.17m^2/MN c_v = 62 - 78m^2/yr	2.3x10^{-8}	3.1x10^{-8}
8 - 10	Volcanic Ash (soft SILT layer)	80 (min)	100	110	55	50	m_v = 0.3 - 0.4m^2/MN c_v = 17 - 25m^2/yr		
10 - 20	Volcanic Ash						m_v = 0.058 - 0.082m^2/MN c_v = 27 - 44m^2/yr		
20 - 37	Variable gravels and organic sand silt mixtures						m_v = 0.16 - 0.27 m^2/MN c_v = 58 - 72m^2/yr		

BEARING CAPACITY

Bearing capacity was assessed using the method of Duncan and D'Orazio [4] to check for bearing capacity failure modes under the base and edge of the tank. An average undrained shear strength value of 80kPa was adopted for all the soils underlying the tank. Edge shear was found to be critical. The following Factors of Safety for edge shear were given by analysis:

Product loading (107kPa)	= 4
Hydrotest (158kPa)	= 2.7
Seismic loading (210kPa)	= 2

The Factors of Safety given all satisfied named engineering criteria and the clients brief.

A total stress analysis using Bishops Method was used to check stability of a circular failure surface associated with the tank loading. The proposed levelling of the site meant that the depth of the soft SILT layer would vary across the site. Two situations were analysed:

Soft layer 5-7m under Tank Base
Soft layer 3-5m under Tank Base

Shear vane testing within this soft layer, completed during the drilling of Borehole 1, gave an undrained shear strength value of 82kPa. A correction for the plasticity index, as given by Bjerrum [2] for this layer (PI = 50 as given by laboratory testing), was then applied, reducing the undrained shear strength to 64kPa.

For the material overlying the layer, an undrained shear strength of 80kPa was used. This value was adopted conservatively from shear vane results within the overlying layer, where refusal was met in all tests. The hydrotest loading of 160kPa (16m water head) was applied with a minimum Factor of Safety of 2.5.

An effective stress analysis was also performed using Bishop's Method for long term loading using effective stress values given by triaxial testing. Soil parameters of effective friction angle of 37° and effective cohesion of 0kPa were adopted for the analysis which gave a Factor of Safety greater than 5.

SETTLEMENT

Estimated settlement at the site was assessed on the basis of oedometer testing, CPT interpretation, and field observations from an adjacent tank. Settlement was calculated for both the hydrotest and product loading under the centre of the tanks and under the tank edges. Settlement was calculated using (lower bound) coefficient of volume compressibility values (m_v) given by oedometer testing at representative stresses for each layer. Six layers were adopted coinciding with representative oedometer test results for each layer (m_v values varied between 1×10^{-7} and 8×10^{-8} m^2/N). The method described by Duncan and D'Orazio [5] was used to calculate total settlements assuming full consolidation under sustained loads.

To assess the differential settlement potential around the edge of Tank 1 and Tank 2, the method of Belshaw [1] was used to calculate settlement using CPT data. The effect of a possibly compressible organic layer indicated by borehole data was used to adjust settlement predicted by Belshaw (ie difference between settlements for CPT 1 calculated using Belshaw and settlements calculated using oedometer results for Borehole 1, situated adjacent to CPT 1).

This method of calculating settlement assumed that the organic layer below 30m (limit of CPTs) is consistent across the site which is indicated to be a fair assumption by Boreholes 1 and 2. This method also assumed that settlement calculated using one dimensional Terzaghi consolidation theory using oedometer results is more or less equal to total settlement as given by Burland et al [3].

Measured settlement values at an adjacent tank during a month long hydrotest period (waterhead of 16m for 11 days) were much smaller that those calculated for the tank. The actual observed settlements for this tank were approximately 35% of the settlements calculated to occur during the hydrotest and approximately 55% of the settlement calculated to occur during the product loading. Survey information also demonstrated that total settlement did not alter significantly (±3mm) over the 8 year period after the end of the hydrotest. It was anticipated that similar behaviour would occur at the Methanex tank site.

Test results for the soils underlying the Methanex tank site demonstrated very low permeability and a slow rate of consolidation. The model adopted for the analysis assumed an average value for c_v of $42m^2$/year over the entire layer, a 20m compressible layer and one free draining boundary. The calculated time for consolidation was 8 years. If the layer was assumed to be free draining from both boundaries (eg if a gravel blanket was placed under the tank) the time for consolidation was calculated to be 2 years. Total settlements calculated during hydrotest loading were not expected to occur during the hydrotest period, as the period of testing was not long enough for the soil to consolidate this amount. Total settlements calculated for product loading were therefore more appropriate.

The product loading over time is a variable which was not defined by Methanex. Because of this it was necessary to design on the basis of continuous full product loading. As the site will not be fully loaded at all times, predicted product settlements were also not expected to occur. This behaviour has been observed at the adjacent tank where surveying indicated no significant settlement during product loading over the 8 years since hydrotesting. 60% of the settlements calculated under product loading were therefore recommended by WCS for design purposes.

Actual settlements have been monitored at the two Methanex tanks using eighteen survey points and three settlement tubes at each tank. Actual maximum settlement monitored are summarised in Table 2 below, along with design settlements:

Table 2. Actual settlement for the Methanex tanks.

Tank	Actual Tank Centre Settlement	Design tank centre settlement	Actual tank edge settlement	Design tank edge settlement	Maximum settlement
1	170mm	170mm	15mm	70mm to 150mm	191mm (38m across tank)
2	70mm	220mm	26mm	130mm to 210mm	82mm (23m across tank)

METHANOL CONTAINMENT

A requirement of the project was that the tank was founded on a platform relatively impermeable to methanol. Site investigations demonstrated that volcanic ash forms a thick, low permeability blanket across the whole Methanex site.

Permeability tests in fixed ring permeameters have shown that the permeability of methanol ($k_{methanol}$) can be 10^3 greater than k_{water} (Mitchell et al [7]). Similar testing in flexible membrane permeameters indicate $k_{methanol} = 10$ k_{water}. It appears that the soil structure changes with methanol as a result of damage/shrinkage of the soil in which situation the fixed ring permeameter test is unlikely to provide a representative result (ie there may be possible leakage along the sample in the permeameter). Methanol concentrations less than approximately 80% have little effect on permeability (Mitchell et al [7]).

Permeability testing (method of Head [6]) completed for the Methanex Project indicated the permeability of recompacted Taranaki ash to water and methanol to be very similar. A sample of Taranaki ash from a depth of 1-2m was firstly tested for water permeability then methanol permeability. The sample was slightly less permeable to methanol and no erosion or deterioration of the soil after methanol permeation was observed.

DISCUSSION

The most important point gained from the geotechnical assessment completed for the Methanex tank site is the importance of observation and factual data. The tank adjacent to the Methanex site was of similar size and loading and the site investigations demonstrated similar materials types underlying both sites. It was therefore considered appropriate to reduce the design settlements at the Methanex site to a similar level monitored at the adjacent tank site.

It is realised that factual data such as observations from an adjacent tank is not always available when completing a geotechnical tank assessment. The loading pattern of the tank, expected consolidation of the underlying soils, and stress history of the underlying soils should then be taken into account when estimating settlement. Oedometer testing completed for the Methanex site indicated low permeability, overconsolidated soils.

The oedometer test is one dimensional in both strain (k_o conditions) and drainage and this test condition is not strictly applicable to field conditions. Sample disturbance may also lead to laboratory results that are not representative of field conditions.

Laboratory testing completed at the site indicated the thick ash layer overlying the site would provide a robust containment medium for the tank compound and the natural ash permeability satisfied the client's brief. Allowing for some soil damage on exposure to the methanol the expected flow rate through the ash could increase, however the penetration by methanol would be restricted by the need to displace soil pore water (hence the soil permeability with water would dominate) and also by the fact that the deposit of low permeability ash across the site is very thick.

CONCLUSIONS

1) The prediction of performance in stratified volcanic soils is complex and difficult.

2) Field observations of previous performance can enable significant improvements in the prediction of future performance.

3) Environmental aspects of containment prevent a further challenge in the definition and understanding of geotechnical performance.

ACKNOWLEDGEMENTS

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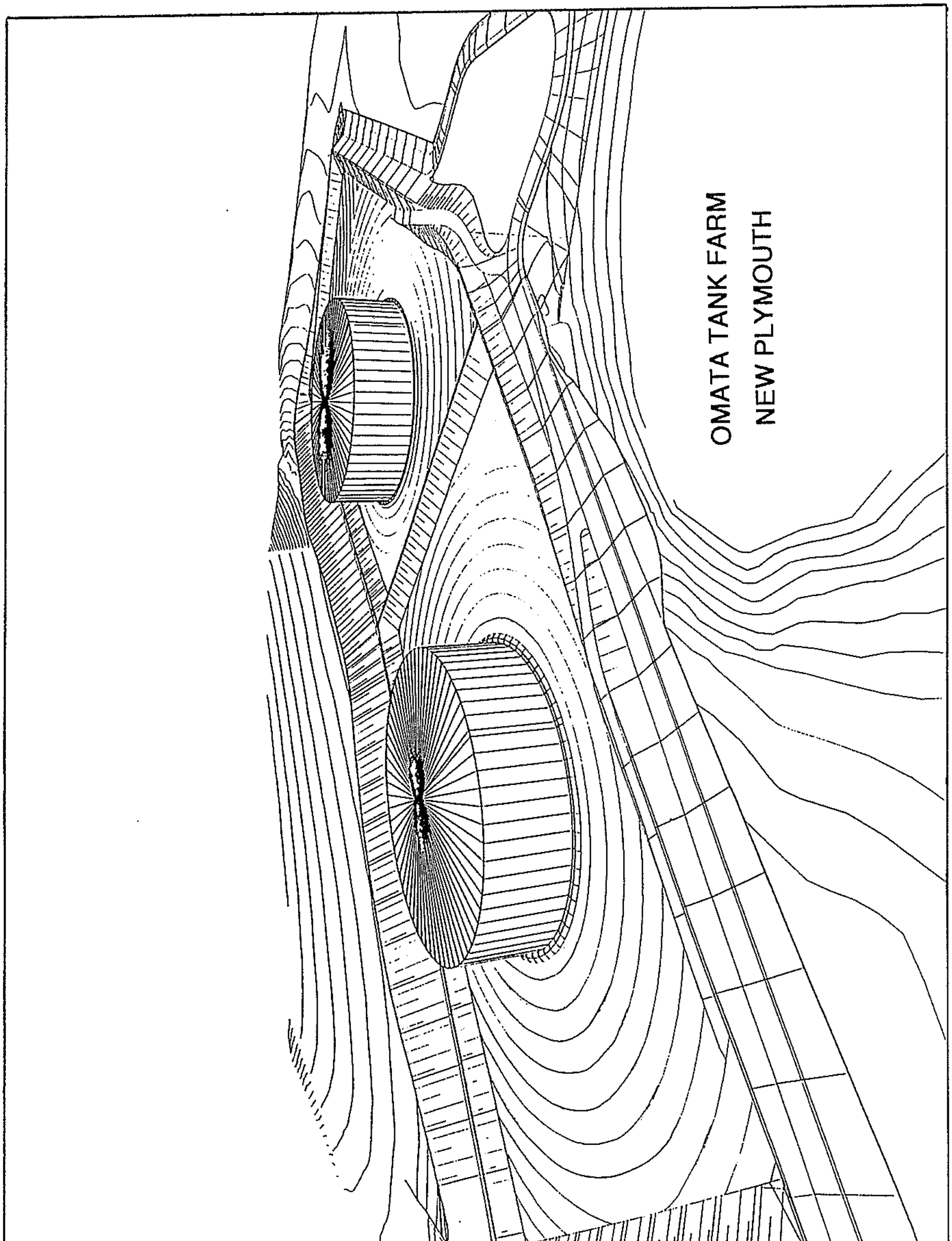
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