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A Particular Foundation Problem on a Waste Fill

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ABSTRACT

The construction of a new aerobic water treatment system for polluted water in the ENICHEM chemical plant at Mantova (N. Italy), emphasized a particular foundation problem; the complete treatment system, covering a surface area of 12000 m^2, lies on a zone once used as a waste fill, 4-6 m deep and composed of various materials from industrial processing, with highly variable origins and poor resistance characteristics. The main structures are: steel tanks with fixed roofs, and gauge and settling tanks, of 30 and 26 m diameter respectively, each containing a 15-m water head, reinforced concrete sediment thickeners and degassing basins with diameters of 15 and 35 m, each containing a 3-4 m restricted water head. The requirement to avoid contact between waste materials and the natural soil beneath excluded the possibility of deep foundations on piles. Shallow foundations were therefore necessary; 4-8 m preloading embankments were designed in relation to the characteristics of the fill and the subsoil, working loads, and the need to limit settlements and distortions. The design criteria of preloading, measurements of embankment settlements (400-700 mm) and main structures during hydraulic testing are reported and discussed.

KEYWORDS

Waste fill, preloading, settlement, steel-tank foundation.

1. SOIL INVESTIGATION

Chemical investigations showed that the waste materials were contaminated in several points both by mercury in insoluble form and also highly and diffusely by aliphatic hydrocarbons and alkylic aromatics. These materials, lying at depths varying from 4 to 6 m, could not therefore be remixed or transported. In order to avoid the spread of contamination to the subsoil and surrounding environment, the hydrological situation was analysed by placing deep piezometers in the natural soil and shallow piezometers in the waste fill. Two water-tables were found: one within the waste fill, with varying depths, indicating stagnant waters and separated from the natural soil by a silt and peat layer of low permeability; and another true water-table in the sandy layer beneath, flowing north-south.

Cone penetration tests (C.P.T.) and deep boreholes to a depth of 40 m from grund level were conducted (Fig. 1) to identify the stratigraphic features of the area. When the occurrence of coarse materials inside the fill did not allow C.P.T., static penetration tests (S.P.T.) were carried out to a depth of about 6 m (Fig. 2).

The stratigraphy of the foundation soils is characterized by the following layers:

- from ground level, (+16-+18 m a.s.l.) to a depth of 4-6 m to +11.5-+13: waste material with highly variable origins and characteristics. These materials were prevalently coarse and permeable; gravel and sands in mixed with formed of silt and mud of low permeability, with varying mechanical characteristics. The C.P.T. indicated a cone resistance q_c varying from 1 to 10 MPa, and static tests a number of N_{SPT} blows varying from 1 to 25.
- below +11.5-+13, for a few centimetres to 1.5 m: a silt and peat layer, resulting from the organic deposits of the previous ground level due to flooding in past ages, with plasticity from moderate to high (w_L%=106-177 and I_p=51-112), medium consistency (pocket penetrometer=30-80 kPa and torvane=30-40 kPa), cone resistance q_c on average 1 MPa and N_{SPT} 3. Edometric consolidation tests measured a vertical consolidation coefficient between 8.9×10^{-9} and 7.6×10^{-9} m^2/s.
- from +12-+15 to +1--1: a medium-fine sandy layer with gravel, with high mechanical characteristics, quite dense, with cone resistance q_c above 10 MPa and N_{SPT} values of 20-30. The permeable layer is the site of the deep water-table situated at +14-+14.5 m a.s.l.
- from +1--1 to approximately -20 (the final investigation depth): prevalence of incoherent layers of mixed sand and dense gravel with q_c higher than 10 MPa and N_{SPT} > 50, and a thickness of 2-4 m intercalated with thin clay, silt and peat layers between 1-2 m. Laboratory tests showed medium plasticity (w_L%=26-60 and I_p=6-40).

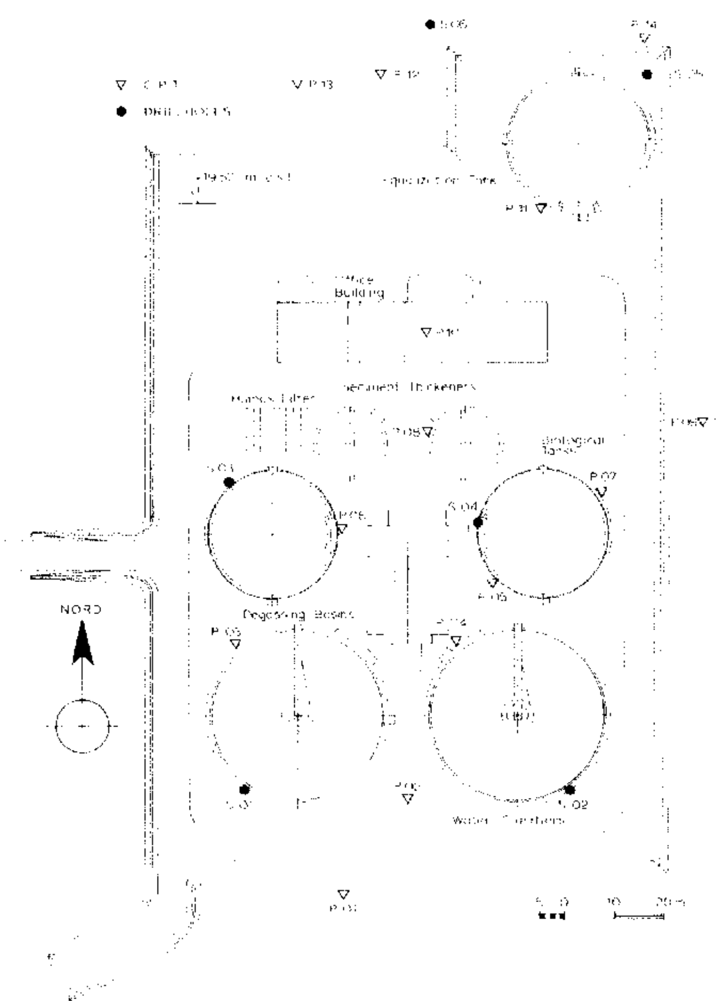


Fig. 1 Sketch of structures and investigation plan.

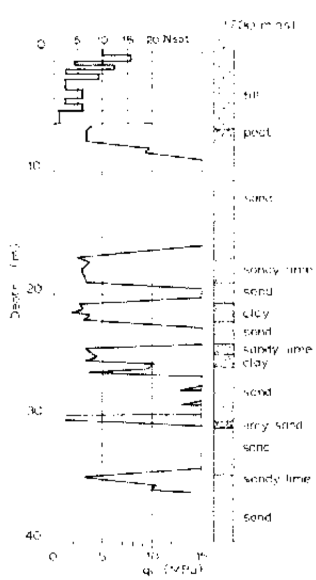


Fig. 2 Soil profile.

2. PRELOADING EMBANKMENTS AND FOUNDATIONS

The design, considering 2 and 3 points on the edge-base foundation, assigned differential settlements and distortions of less than 1/300 and 1/500 to the steel tanks and concrete structures.

Using an FEM program, schematizing the ground as a multilayer with variable geotechnical characteristics, the stress induced by the leveling fill from +16-+18 m to the foundation line at +19.50 m and by the planned tanks and works was evaluated. The elastic and consolidation settlements of the various works were then calculated with the one-dimensional consolidation theory of Terzaghi (Table 1). The time estimated for 90% consolidation was 3 months. The finished level of the plant, as stated above, was established at +19.50 m for hydraulic safety reasons, owing to the possibility of flooding from the nearby lakes, with the consequent need to create a embankment on the existing ground level at +16-+18 m a.s.l.

Table 1 Calculated settlements (mm)

	Fill (m)	Settlem.	Oper. load	edge	center
Biological tanks	+17 to +19.5	402 mm	170 MPa	126	241
Equalization tanks	+18 to +19.5	303 mm	180 MPa	101	195
Water clarifiers	+18 to +19.5	322 mm	40 MPa	39	77

The preloading embankments from +19.50 m was designed in relation to the dimensions and loads transmitted by the works and the soil bearing capacity (Table 1): 8 m (+27.5) for the metal tanks and 4 m (+23.5) for the office building and basins (Fig. 3). Embankment side slope was 2:3, so as to guarantee an adequate safety margin with regard to local and construction failures.

The leveling fill (about 43000 m^3) and preloads (about 50100 m^3) used soil from quarries (sands) and gravelly and sandy tout-venant, which was then re-used to create level foundations for the tanks and other works.

Secondary compression of the silt and peat layer was not taken into account when predicting of the extent and course of settlements. Preloading criteria involved reduction of major settlements under works once they were operational, so that the effects of possible secondary settlement in the peat layer were considerably reduced.

Twenty-one plate bench marks were emplaced after work involving the leveling fill but before that of the preload embankments, in order to check the progress of displacements over time (Table 2). The settling meters were read weekly during construction and 3 months after embankment emplacement (Fig. 4).

At the end of 3 months, total settlements measured were of the order of 300-500 mm for 4-m preloads and 500-600 mm for those of 8 m.

For the foundation plan of the tanks and basins, starting from +19.50 m, the ground fill was removed and replaced with tout-venant for 2 m at the edge of the tanks (in which concrete kerbs were planned to support the metal structures) and for 1-1.5 m at the flexible foundations of the bottom of the tanks. For the other works, it was sufficient to remove 0.5-1 m of ground fill.

Table 2 Measured displacements over time during preloads from +19.50 m (mm)

	t = 00 d.		t = 20 d.		t = 45 d.		t =90 d.		t =180 d.	
	ed.	cen.	ed.	cen.	ed.	cen.	ed.	cen.	ed.	cen.
Biol. tank	12	145	14	175	16	190	16	200	17	210
Equal tank	11	140	15	180	15	190	15	190	15	190
Water clarif.	65	90	95	130	10	135	10	140	10	140

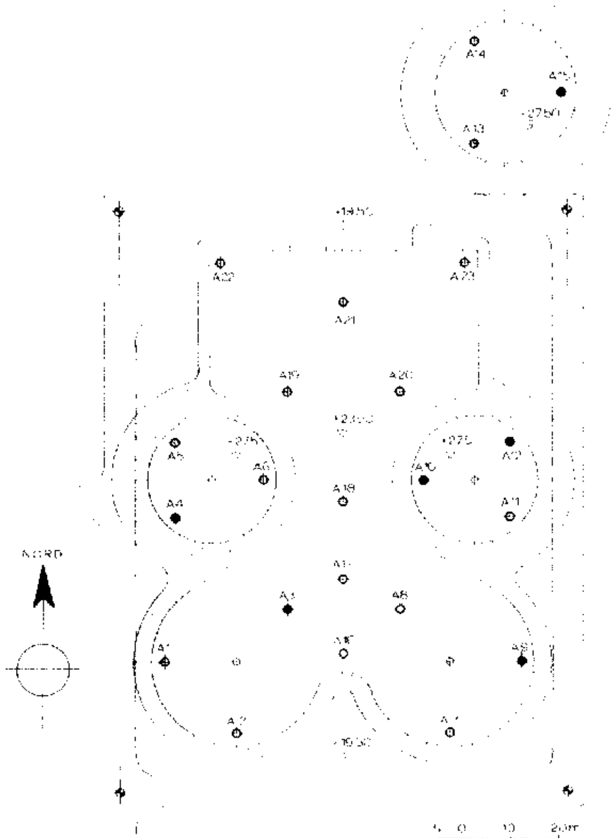


Fig. 3 Locations of preload embankments and plate bench marks.

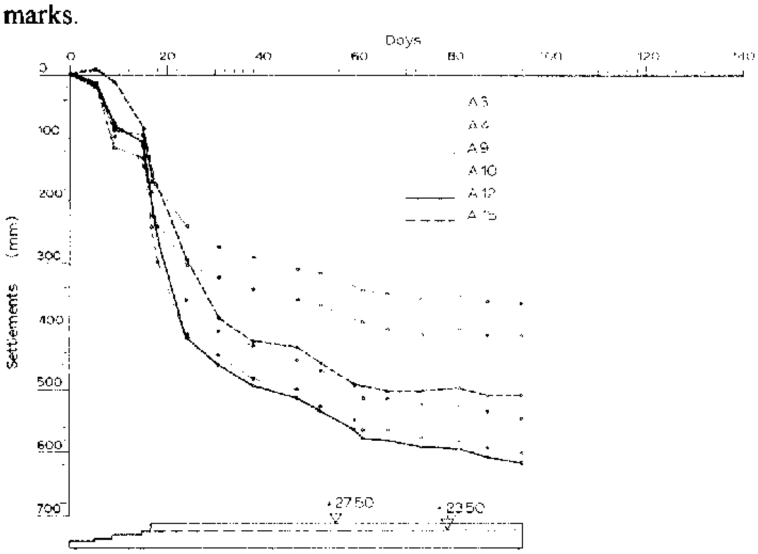


Fig. 4 Observed displacements as measured by plate bench marks.

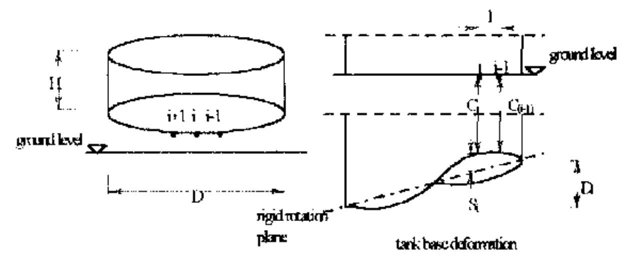
3. HYDRAULIC TESTING

After the construction of the tanks and basins, hydraulic testing for each single work, lasting about 1 month, was carried out with a water load equal to that of operation.

Settlement measurements were carried out with leveling of datum points along tank and basin perimeters (Fig. 5).

The trend of the settlement-time graph for all works demonstrated elastic settlements, for example for the biological tank (Fig. 6). The final measurements with empty works revealed the extent of elastic settlements. Elastic recovery in relation to total settlements showed how the works and connections to the plant network were not greatly stressed in the load-unload cycle of the structures.

Distortions were defined and compared with the corresponding admissible limits.



C_i total settlem. S_i out-of-plane settlem. D_i/D rigid rotation

$D_{2c}=(C_i-C_{i-1})/l$ distortion on basis of 2 consecutive points in relation to total settlements

$D_{3s}=[S_i-0{,}5x(S_{i-1}-S_{i+1})]/l$ distortion on basis of 3 consecutive points in relation to total settlements

Max. rigid rotation D_i/D	Max. out-of-plane settlem (D.<50m) S_i	Max. distor. 2 consec. points D_{2c}	Max. distor. 3 consec. points D_{3s}	Max. cen.-ed. distor.	Max. diff. settlem. diamet. op. points D_i
1/200	40 mm	1/300	1/500	1/75	500 mm

(Langeveld 1974; Belloni et al. 1975; Sullivan & Nowicki 1975; Greenwood 1975; D'Orazio & Duncan 1982.)

When identifying rigid rotations and distortions along three consecutive points, it was necessary to determine out-of-plane settlements S_i on the basis of total settlements measured, so that Malik's equations were used.

The results of these equations were graphed and compared with the base deformations of every tank and basin at the end of each phase.

Analysis of distortions did demonstrate that all tanks had undergone settlements and distortions along their edges lower than the indicated limits. For the concrete basins too (for which the admissible limits could be taken as indicative, as they had considerable deformability without static problems), the comparison parameters were reasonable.

In conclusion therefore, the tanks of all types had fairly regular movements contained within admissible limits, thus demonstrating that they could guarantee excellent behavior in operation from both static and functional viewpoints.

Adm. limits			40	500	$5e^{-03}$	$3e^{-03}$	$2e^{-03}$
Water level (m)	$C_{i\,min}$ (mm)	$C_{i\,max}$ (mm)	S_i (mm)	D_i (mm)	D_i/D	D_{2c}	D_{3s}
Equaliz. t.							
6	13.0	15.1	1.1	0.6	$2e^{-04}$	$1e^{-04}$	$1e^{-04}$
12	29.0	32.6	2.1	2.0	$6e^{-04}$	$2e^{-04}$	$2e^{-04}$
15	40.5	45.4	2.8	2.5	$8e^{-04}$	$3e^{-04}$	$3e^{-04}$
Bilog. t. W							
5	10.8	13.1	0.4	2.2	$3e^{-04}$	$1e^{-04}$	$5e^{-05}$
10	23.6	31.3	1.4	7.7	$8e^{-04}$	$3e^{-04}$	$1e^{-04}$
15	34.0	50.0	1.0	16.0	$6e^{-04}$	$6e^{-04}$	$1e^{-04}$
Water clarif.							
4	5.4	10.5	1.6	3.8	$1e^{-04}$	$2e^{-04}$	$1e^{-04}$

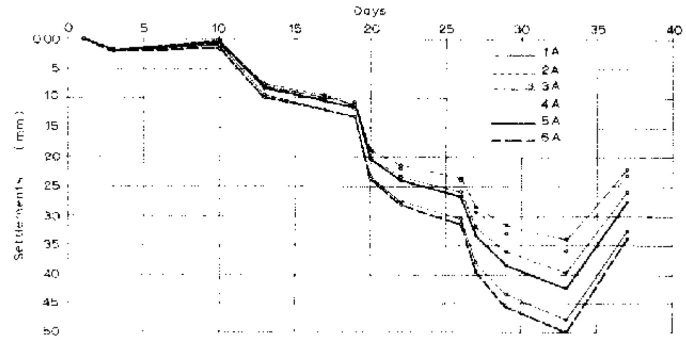


Fig. 6 Time-settlement graph of biological tank.

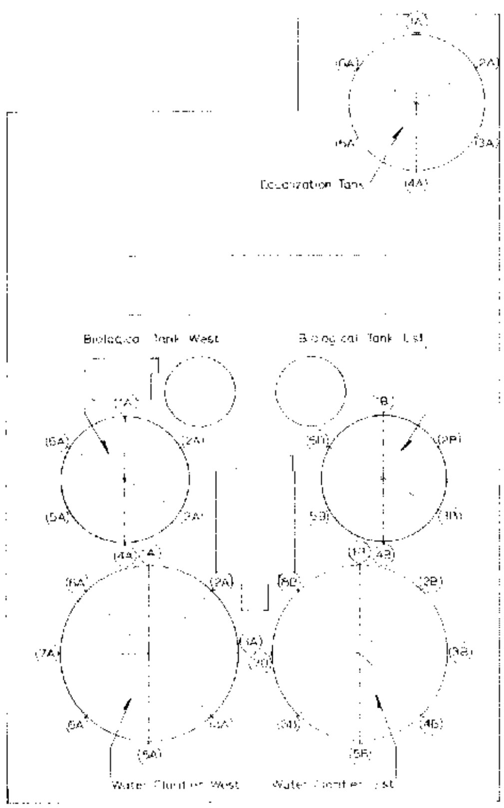


Fig. 5 Location plan of leveling datum points.

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