

GEOTECHNICAL DESIGN CRITERIA FOR FOUNDATIONS OF LARGE DIAMETER TANKS

Gopal Ranjan

College of Engineering Roorkee, Roorkee

Ravi Sundaram
Sanjay Gupta

Cengrs Geotechnica Pvt.Ltd.,
B - 3 / 87, Safdarjung Enclave, New Delhi.

SYNOPSIS : Large diameter steel tanks are commonly used to store large volumes of crude oil, petroleum products etc. The paper reviews the geotechnical design criteria for the foundation system for such tanks. Where tanks are planned to be installed in weak sub-soils, this forms the basis for assessment of the need for ground improvement. The concepts for the analysis based on the bearing capacity and settlement criteria are discussed in detail together with the stability of the tank during earthquakes. A need for standardizing hydrotest procedures is emphasized.

A case study illustrating foundation strengthening by provision of granular piles to improve loose subsoil to support a 22 m diameter tank is presented. The field control methods for successful ground improvement are also discussed.

INTRODUCTION

The liquid storage tank is unique with respect to the nature of the load which primarily is a uniformly distributed load on a circular area. The foundations of these tanks are proportioned based on allowable bearing pressure on the subsoil satisfying the shear failure and settlement criterion.

Where the tanks are required to be located on weak subsoils - loose sand, recent alluvium, soft clay, marine clay, etc. - the foundations need to be designed carefully to ensure safety. If need be, the subsoil conditions should be improved so that the settlement is within permissible limits.

DESIGN CRITERIA FOR TANK FOUNDATION

The two important considerations to estimate the allowable soil bearing pressure for foundation design are :

- a. Shear failure criterion
- b. Settlement criterion

These two criteria are independent of each other and both must simultaneously be satisfied. Firstly, it should be ensured that the foundation system has an adequate safety factor against shear failure. Second, the total and differential settlements should be within the permissible limits. The differential settlement should not cause any overstressing or opening up the welded connections.

Shear Failure Criterion

Two modes of foundation instability with respect to shear need to be considered. These are -

- a. Base shear failure
- b. Edge shear failure

Base shear failure involves a deep seated rotational movement of the tank with soil mass as a rigid body. For soft clays subjected to undrained failure, the slip surface is usually circular. For this mode of failure to occur, the thickness of the weak layer should be comparable to the tank diameter. Heaving of the surrounding ground accompanied by tilt of the tank would be a clear indication of this mode of failure (Fig. 1).

Localized failure near the edge of the tank may occur due to the presence of relatively thin weak layer or a localized weak pocket. This mode of failure causes distortion of the flexible bottom and consequent rupture of the welds. (Fig. 1).

The basic premise of these two modes of failure is that in the former case, the tank fails as a single unit while in the latter case only a limited portion of the tank base comes into play. It has been reported that the latter case is more critical than the former one because it may also lead to tank base distortion, thereby culminating into loss of contents.

Additional modes of failure, particularly in variable deposits with thin / localized soft soils (Datye, 1986) include :

- (i) Squeezing of soft layer below the tanks : This mode of failure involves squeezing of clay layer sandwiched between two stiff layers. Failure is accompanied by punching of the tank through top stiff layers. Punching causes vertical cracks near

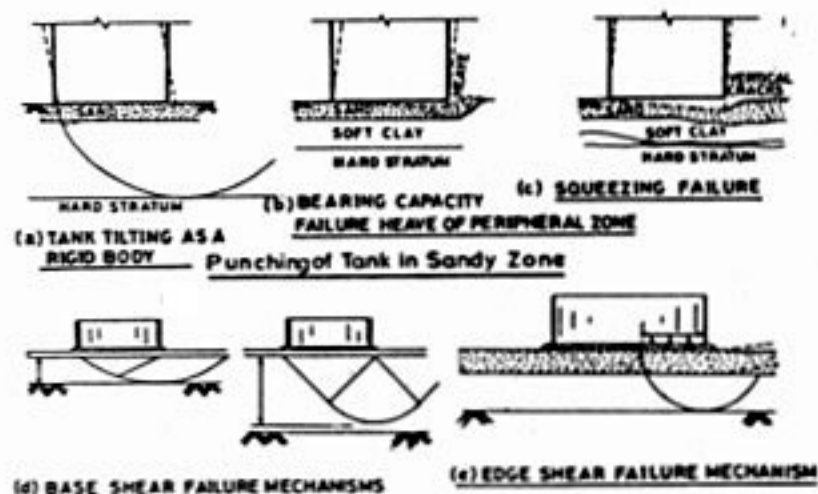


Fig.1 : Bearing Capacity Failure Modes

the tank periphery and will be accompanied by settlement. There would be no heave (Fig. 1).

- (ii) Pockets of loose soil may get punched. This may cause abrupt changes in settlements resulting in tank failure. It is possible to prevent damage by this mode of foundation deformation by stage loading of the tank and re-leveling / grouting.

Settlement Criterion

Settlement can cause the tank to fail structurally and collapse even if the factor of safety against shear failure is high. Hunt (1986) discussed the effects of settlement on a steel storage tank (Fig. 2). In general, the settlement are of two types -

- a. Uniform settlement
- b. Differential settlement.

Flat bottom tanks are flexible structures and generally can tolerate substantial total and differential settlement if they are structurally sound. Total settlement of 150 to 500 mm are relatively common.

A storage tank consists of four main structural elements : shell, bottom plate, connection of shell to bottom plate, and roof. The settlement pattern influences each of these components. The differential settlement may result in failures such as distortion of shell resulting into the floating roof malfunctioning or rupture the shell or bottom plate or shell - bottom plate connection.

Uniform, total settlements of a tank are usually not of major concern except for effects on connecting piping. Differential settlement along the wall of a tank is of critical concern. This can cause distortion and buckling of the shell and have the secondary effect of causing ovalization of the shell and binding of floating roofs.

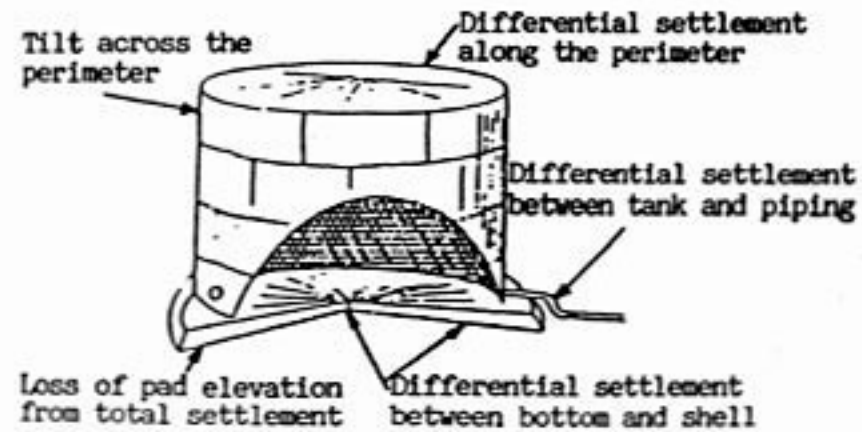


Fig.2 : Effects of Settlement on a Steel Storage Tank (After Hunt, 1986)

Uniform Settlement

Foundation loads can produce basically three types of settlement. These are :

- a. Immediate or elastic settlement
- b. Primary consolidation settlement
- c. Secondary compression settlement

Differential Settlement

Differential settlement acts as the governing parameter because it can lead to the rupture of the tanks, culminating in excessive soil spillage. It may even lead to malfunctioning of floating roof of the tank.

COMPONENT	ASPECT OF PERFORMANCE	MODE OF FAILURE	CRITERION	DEFINITIONS & COMMENTS
SHELL	<p>Plane of average tilt Planar Tilt $Z_1 = \bar{\rho}_1 + 2(\delta/D)\cos(x/R + \beta)$ Where : $\bar{\rho}$ = average shell settlement β = orientation of plane of average tilt δ/D = angle of average planar tilt. Non-Planar Settlement : $S_1 = \rho_1 - Z_1$ $\Delta S_1 = S_1 - 0.5(S_{i+1} + S_{i-1})$, $\ell = \pi D/n$</p>	PLANAR TILT I. Overtopping of Shell II. Loss of roof Seal NON-PLANAR SETTLEMENT III. Binding of roof seal IV. Overstress of shell	I. $\delta \leq 2 \Delta h_4$ II. $\delta \leq 2 \sqrt{\Delta R_{tot} D}$ III. $\Delta S \leq \frac{\ell^2}{HD} \Delta R_{tot}$ IV. $\Delta S \leq 11 \frac{\ell^2 \sigma_y}{HE}$	Δh_4 = freeboard H, D = tank dimensions ΔR_{tot} controlled by : 1. Tolerance of roof seal 2. Buckling of wind girder 3. Distortion of cone roof. σ_y = yield strength of shell E = Young's modulus of elasticity.
BOTTOM PLATE	<p>Non-Planar Settlement: D, d, W</p>	V. Rupture from dish-shaped settlement. VI. Rupture from localized depressions. a. remote from shell b. adjacent to Shell	V. $W \leq \left[W_e^2 + \frac{0.37 \sigma_f D^2}{FS \cdot E} \right]^{0.5}$ VI. a. $S \leq d \left[\frac{0.28 \sigma_f}{FS \cdot E} \right]^{0.5}$ b. $S \leq d \left[\frac{2.25 \sigma_f D}{0.75 FS EH} \right]^{0.5}$ d and S in meters	σ_f = ultimate strength of bottom plate $FS \leq 4$ localized yield possible $FS \leq 2$ severe overstress and yield possible E = Young's modulus of elasticity W_e = initial camber d, \bar{d} = dimensions of local depression, m For local depressions adjacent to shell, if $d < D/4$ and $\bar{d} \geq 2d$, use criterion VI.b. Otherwise use criterion VI.a.
SHELL BOTTOM PLATE CONNECTION	<p>Non-Planar Settlement Distorted Weld Deformed tank wall Deformed annular rings</p>	VII. Rupture of connection as shell bridges over soft spot.	VII. Surveillance & maintenance to prevent separation of shell & foundation.	

Fig. 3 : Recommend Criteria for Differential Settlement of Welded Steel Tanks (After Marr et al, 1982)

Differential settlement originates due to the following causes :

- (i) non - homogeneous geometry/compressibility of the soil deposit
- (ii) non - uniform stress distribution

Planar tilt of the shell reduces the freeboard and places additional stress on the shell. Non - planar tilt may radially distort the shell (ovalization) or overstress the shell causing the floating roof to malfunction. Non-planar settlement of the bottom plate may cause a dish shaped settlement contour (dishing) or localized depressions. Non - planar settlement of the shell bottom plate connection could result in the rupture of the weld. The criteria recommended by Marral (1982) for differential settlement of tanks is presented in Fig.3.

STABILITY UNDER DYNAMIC LOADING

On the basis of the observations of numerous earthquakes around the world, damage to tanks can be placed in the following five general categories (Cambra, 1982).

- a. *Buckling near the base* : Lateral forces produce overturning moments and hence longitudinal compressive tank wall stresses that may result in buckling near the base. The buckling manifests itself as either an outwards bulge, termed as 'elephant's foot buckling' or as 'diamond shape buckling'.
- b. *Buckling of roof and upper tank walls* : Waves generated by lateral motion may result in buckling of roofs and upper tank walls as well as failure of internal roof support column.
- c. *Damage to piping and other appurtenances* : Movement of tank during seismic activity may result in damage to piping and other appurtenances.
- d. *Soil failures* : The types of soil failure identified include instability of bearing caused by liquefaction, wash out due to ruptured piping, and shear failure resulting from high bearing loads at the base plate annulus.
- e. *Uplifting* : Large uplifting displacements of the tank shell impose large stresses in the base plate annulus near the tank wall. This may result in extensive tearing of the tank bottom plate and ultimate loss of tank contents.

TANK PERFORMANCE

Before the tank is filled with oil / petroleum products, it should be hydrotested. The hydrotest is a performance study of the tank and thus plays a significant role in ensuring its safety. Also it allows major part of the settlement that occurs during the test so that the settlement during the working life of the tank is restricted.

If the rate of loading during the hydrotest is high, the possibility of failure of the tank cannot be ruled out. The rapid loading of the subsoil results in building up of high excess pore water pressure in the subsoil, particularly in clay deposits. This cumulative build up of pore pressure under undrained condition of loading eventually triggers ground movement leading to failure.

The estimate of time required for hydrotest must take into account the fact that the development of excess pore water pressure and its dissipation is a function of subsoil characteristics and magnitude and

spatial distribution of induced stress. Unfortunately, except for a few hydrotest data available (Rao 1993, Ranjan 1998), there is lack of data available on monitoring during the hydrotest. Further, the time for hydro-testing in India is totally arbitrary, sometimes depending upon absurd factors such as the availability of water supply, convenience of testing personnel etc.

A scientific approach to the problem would be to permit a relatively high rate of loading say of the order of about 1 m height of water per day upto a point at which excess pore water pressures and ground deformations are within safe limits. Beyond this point, the rate of loading should be suitably reduced. Further, stage loading followed by a pause at every stage is also useful since it not only reduces excess pore pressure but also leads to higher factor of safety against shear failure. Much higher rates of loading may however be adopted in case of tanks on free draining soils.

CASE STUDY

A case study discussing the load testing and performance evaluation of granular piles installed at a project site near the bank of River Yamuna at Delhi is presented here. At this site, three 22 m dia, 10 m high steel tanks were planned for storage of water for fire fighting purpose.

The net applied pressure on the subsoil due to the load on the tank was about 10.5 T/m^2 . On the periphery of the tank, due to stress concentration and hoop stresses, the pressure is about 12.5 T/m^2 . Analysis of the soil boring data indicated that the settlement under the applied bearing pressure may exceed 100 mm (the tolerable settlement as per the project specifications).

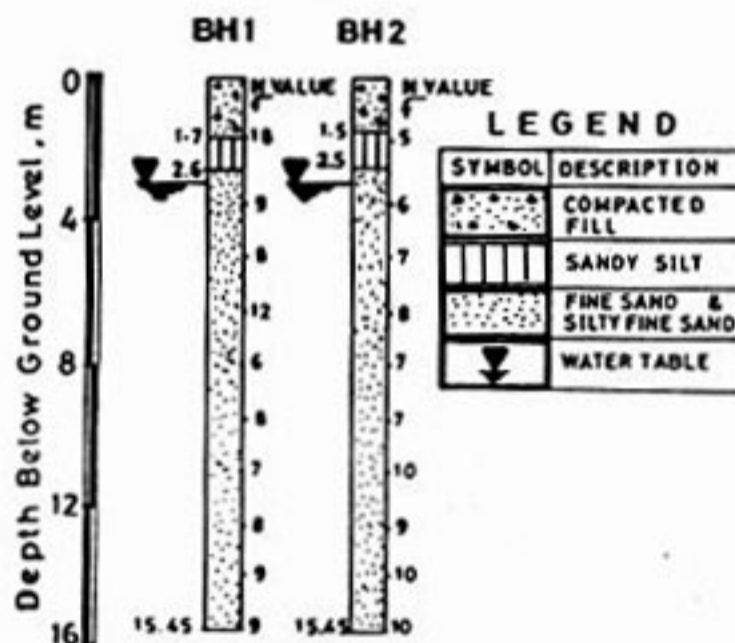


Fig. 4 : Site Stratigraphy

To improve the density of the loose subsoil and to reduce the total settlement, it was decided to install 400 mm diameter 12 m long rammed granular piles to improve the soil so as to ensure a safe soil bearing pressure of 12 T/m^2 . Trial granular piles were installed at spacings of 1.2, 1.5 and 1.8 m. Boring was done by DMC method. A surface casing of about 2 m length was used. A thin (5 percent) bentonite slurry was circulated to maintain the borehole stable. The

stone aggregate (40 mm, 20 mm and 10 mm mixed, graded) and coarse sand were placed in 1 m thick layer.

Ramming was done on the gravel using a 600 kg hammer falling through a height of 3 to 3.5 m. The set criterion developed from field observations was as follows (Sanjay Gupta and Ravi Sundaram, 1996):

- (1) Apply minimum 25 blows on the gravel layer.
- (2) Record penetration of the gravel for every 5 blows
- (3) If the set (lowering of level of gravel) is more than 2 cm for 5 blows, ram the gravel further by giving 5 more blows and check the set obtained.
- (4) If the set for 5 blows is less than 2 cm, the next charge of sand and gravel may be poured.

Fig. 5 presents a comparison of dynamic cone penetration tests before and after improvement. The extent of improvement achieved indicates the compaction of the sand that has been taken place between the granular piles.

Fig. 6 presents results of load test on group of granular piles for the different spacings selected. These results suggest that for centre to centre spacing of 1.8 m between the granular piles, the improvement achieved is not significant. Further, the 1.5 m spacing between the granular piles appears to yield optimum compaction. The reduction of the spacing to 1.2 m does not yield any significant advantage.

The final design and installation was done maintaining a 1.6 m spacing in the central portion of the tank and 1.4 m spacing in the periphery. Two rows within the tank area and two rows outside the tank area had the reduced spacing of 1.4 m. A triangular grid pattern built up on a hexagonal layout was used.

A hydrotest was conducted on one of the tanks. The tank shell settled by about 20 mm under a 10 m water height, thereby indicating successful ground improvement.

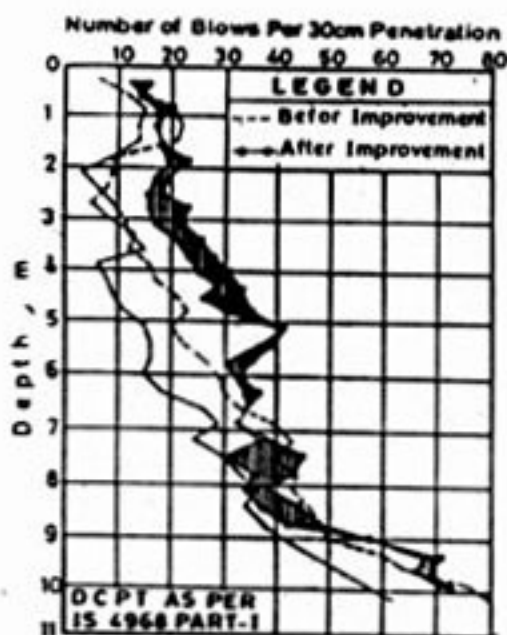


Fig. 5 : Dynamic Cone Penetration Test Before and After Improvement

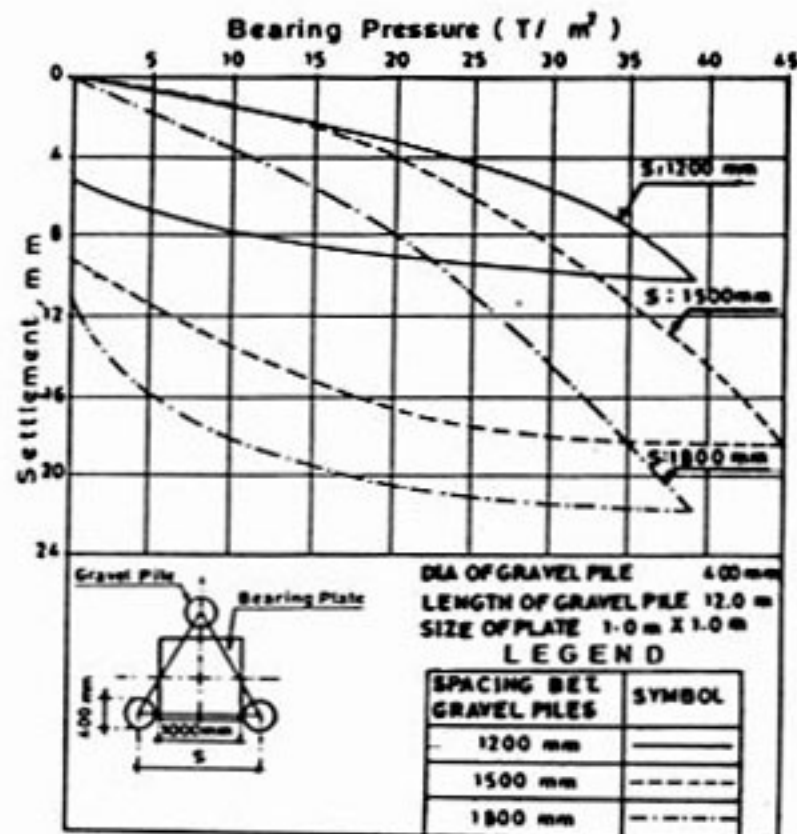


Fig. 6 : Load Settlement Behaviour of Improved Ground

CLOSURE

The allowable bearing pressure at the base of large diameter welded steel tanks should be decided based on the shear failure and settlement criteria. While these tanks can withstand large total settlements, differential settlements can be critical. If required, ground improvement may be done to support tanks on weak substrata. There is an urgent need to standardize the procedure for hydrotesting of tanks.

REFERENCES

- Cambra, F.J. (1982) - "Earthquake Response Considerations of Broad Liquid Storage Tanks", Report No. WCB/EERC-82/25 Earthquake Engineering Research Centre, College of Engineering, University of California, Berkeley, California, Nov. 1982.
- Datye, K.R. (1986) "What Foundation Systems are Appropriate for Large Steel Oil Storage Tanks Resting in Weak Soils?", Proceedings, Indian Geotechnical Conference IGC - 1986, Vol.1, pp 159 - 170.
- Hunt, R.E. (1986), "Geotechnical Engineering Analysis and Evaluation", McGraw Hill Book Co. New York.
- Marr, W. Ramos, J.A., and Lambe, T.W. (1982) - "Criteria for Settlement of Tanks", Journal Geotechnical Engineering Division, ASCE, Vol. 108, No. GT 8, pp. 1017-1037.
- Sanjay Gupta and Ravi Sundaram (1996), "Gravel Piles: Construction and Field Testing", Proceedings, Seminar on Piles, Indian Geotechnical Society, Delhi Chapter, pp 72-81.