Aug 11th - Aug 16th

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6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

PERFORMANCE OF LARGE STORAGE TANK IN BHUJ EARTHQUAKE

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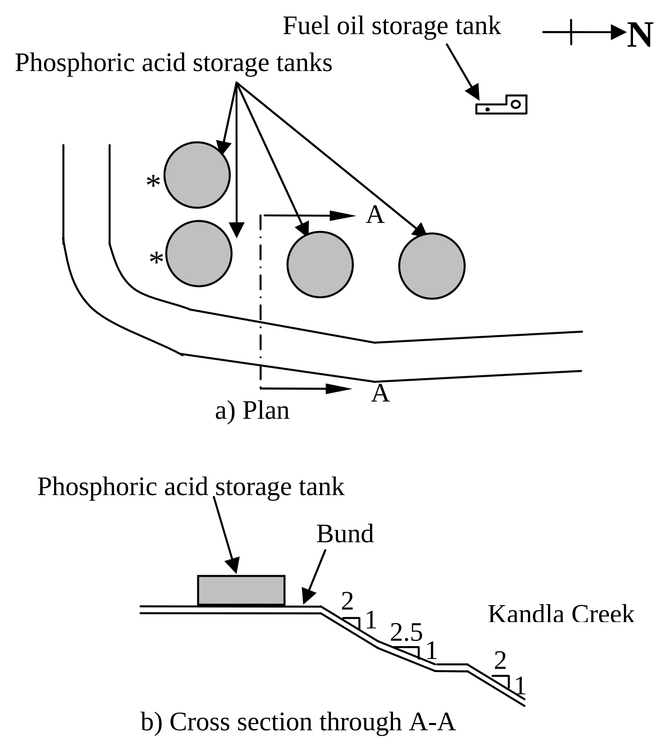
ABSTRACT

The 2001 Bhuj earthquake of magnitude 7.7 caused a widespread damage in the state of Gujarat, India. This paper presents a case study of phosphoric acid storage tank weighing 100,000 kN and measuring 30 m in diameter built for a fertilizer plant in Kandla, Gujarat. The post earthquake performance assessment was carried out by exhuming the nearby piles and non-destructive testing of piles. The storage tanks supported on piles, installed in a ground treated with stone columns, showed no failure and have performed well during the earthquake. The design philosophy used to resist axial and lateral loads is explained.

PROJECT BACKGROUND

A large fertilizer plant was built by the Indian Farmers Fertilizer Cooperative Ltd. (IFFCO) at Kandla in the western state of Gujarat, India. The phase I was built from 1970-1972. It included the NPK plant (main plant), phosphoric acid tanks, ammonia tanks, water tanks, storage heaps, steel conveyor galleries, workshops, pipe racks and other service structures. Except for the main plant, all other structures were founded on stone column/sand drain treated ground. For the major heaps (product), piles were used with due consideration to negative drag. Phase II was built from 1976 to 1980, to double the plant capacity. For Phase I, pseudo static analysis was carried out, whereas for phase II, dynamic analysis was also carried out for soil-pile-superstructure interaction.

The project included various critical structures such as ammonia, main NPK plant supporting heavy machines, phosphoric acid storage tanks, and adjoining RCC conveyer galleries. The site profile near the phosphoric acid storage tanks is shown in Fig. 1. The 2001 Bhuj earthquake prompted the IFFCO authorities to undertake a detailed study of the possible damage to foundation and rehabilitation measures required to ensure the plant safety in the future. The first author was responsible for the original design of foundation system used for all the structures within the IFFCO plant when it was built and therefore was entrusted the task of post earthquake analysis of its performance. This paper deals with the soil profile, design philosophy adopted for foundation and its post earthquake performance through specific reference to the large phosphoric acid storage tank.



* Tank exclusively on stone columns

Fig. 1 Site lay out of IFFCO plant

SOIL INVESTIGATION

The IFFCO site is situated on a low-lying area gradually sloping towards the Kandla creek. The ground where the large storage tanks and other critical structures would be built was submerged under Kandla creek with a mean tidal level of 2 m. The mobilization of drill rigs for the soil investigation in shallow water was not possible. Therefore, the submerged areas were investigated using a lightweight dynamic probe having the same geometry of the SPT sampling tube. The probing, using a hammer weighing 18 kg along with the disturbed sampling was carried out using small boats. The disturbed samples were used to estimate the soil index properties. A more elaborate soil investigation including SPT, vane shear test, and undisturbed sampling was carried out where the drill rigs could be mobilized. The site specific correlations thus developed were used for structures located in areas where a detailed investigation could not be carried out. This decision was based on the experience from the previous soil explorations in the Kandla creek where the correlations have been found to be valid owing to the consistent geomorphology of the creek.

SOIL PROFILE

A typical soil profile based on visual classification and sieve analysis carried out in site laboratory is shown in Fig. 2. The top 4 m consist of well compacted fill of silty clayey sand (SM - SC). The fill is underlain by 10 m of sandy silt. The silt is followed by 6 m of clay and silt mixed with fine sand. The field vane shear tests conducted on silty and clayey sand layers showed the shear strength values in the range of 5 to 70 kPa. Although the clay fraction in these layers (between 1.5 to 15 m depth) is less, the soil is plastic due to high activity of clay. At few locations, the liquidity index of more than one was observed. These layers are susceptible to cyclic mobility due to the presence of silt. This behavior was confirmed during the pile driving when the piles showed a penetration of up to 4-5 m under a single stroke of the hammer. In addition, the vane shear test results show that soil has low shear strength. Therefore, for design parameters, the interpretation of soil characteristics was done by examining vane shear tests, index properties and liquidity index in particular. The clay and silt layer is followed by 2 m of cemented sand and hard clay with SPT values higher than 50. The hard clay was chosen as the bearing layer for supporting the storage tank. The bearing capacity was estimated by conducting a static cone penetration test on top of hard clay. A special tool was fabricated since the hard layer could not be reached by conventional static cone penetration (SCPT) tools. The ultimate bearing capacity was estimated to be 20000 kPa using static cone penetration test and the cavity expansion theory proposed by Vesic (1972).

DESIGN ISSUES

The phosphoric acid storage tank with design vertical load of 150,000 kN had to be built with stringent performance criterion. Apart from the large vertical load, the foundation had to be designed to resist the lateral load due to soil movement in the event of earthquake.

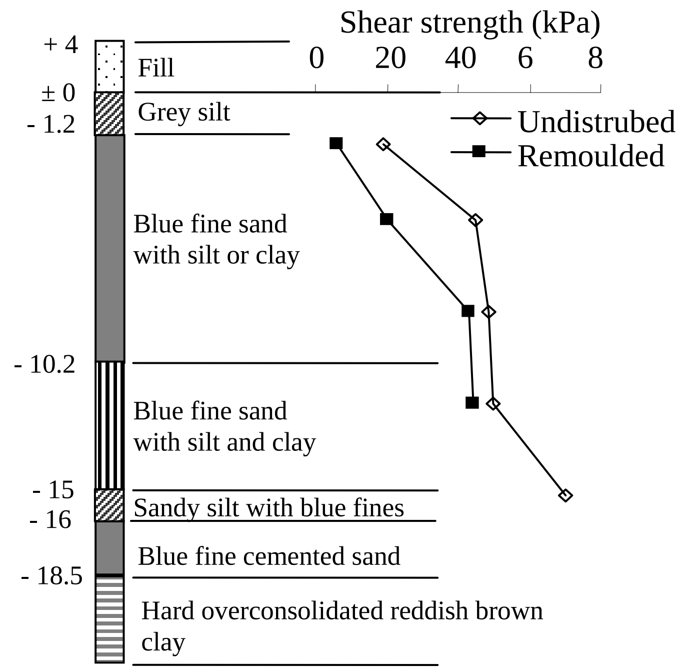


Fig. 2 Soil profile at IFFCO plant

During earthquakes, low density fine sand with little silt content can liquefy but clay even of low shear strength does not liquefy but is subject to cyclic mobility and may thus result in very large displacement. In the present case, the soft soil between 1.5 to 16 m depth was likely to develop cyclic mobility. In such event, the foundation would be subjected to large lateral displacement. In such situations, it is important to predict the maximum ground motion anticipated based on the soil profile (Kanai, 1966, Idriss and Seed, 1968). Tajimi (1977) reported the maximum ground displacement for different soil conditions as given in Table 1.

Based on Table 1 a maximum ground displacement of 150 mm that corresponds to the 10 m thick soil of soft consistency was anticipated for the present case. However it is not clear whether the anticipated displacement values correspond to short-term event (during earthquake) or to the long-term residual deformations. For the design purpose, it was assumed that the load corresponding to lateral soil displacement would remain for design life. This would result in the large lateral load on the foundation in the event of earthquake.

Table 1. Anticipated maximum ground displacement and soil conditions (after Tajimi, 1977)

<table>
<tr><th colspan="3">Soil Conditions</th><th rowspan="2">H (m)</th><th rowspan="2">U_g (cm)</th></tr>
<tr><th>Soil consistency</th><th>Soil type</th><th>SPT value</th></tr>
<tr><td rowspan="3">Firm</td><td rowspan="3">Sandy
Clayey</td><td rowspan="3">30>N>20
20>N>5</td><td>10</td><td>1</td></tr>
<tr><td>20</td><td>1.5</td></tr>
<tr><td>30</td><td>2</td></tr>
<tr><td rowspan="3">Intermediate</td><td rowspan="3">Sandy
Clayey</td><td rowspan="3">20>N>5
5>N>2</td><td>10</td><td>3</td></tr>
<tr><td>20</td><td>7</td></tr>
<tr><td>30</td><td>10</td></tr>
<tr><td rowspan="3">Soft</td><td rowspan="3">Sandy
Clayey</td><td rowspan="3">5>N
1>N</td><td>10</td><td>15</td></tr>
<tr><td>20</td><td>25</td></tr>
<tr><td>30</td><td>35</td></tr>
</table>

Where
H = thickness of soil layer,

N = penetration blow count,

U_g = maximum ground displacement.

Therefore, the foundation had to satisfy the following,

a) settlement criterion under the vertical load of structure plus the dragload on account of fill
b) resistance to large lateral forces resulting from soil movement in the event of earthquake.

A raker pile system is generally used when the structure is subjected to large lateral loads. A raker pile system installed in a thick soft soil would be subjected to large interactive forces when the ground shifts due to earthquake of large intensity. However, due to uncertainty involved in anticipated ground movement the raker pile option was dropped during the Phase II of the project. A foundation system was devised to take the vertical loads and minimize the lateral loads on account of soil movement as explained in the following sections. The vertical load from the storage tank was transferred to the hard clay stratum through a system of short columns and raft supported on driven precast concrete piles as shown in Fig. 3. The large diameter bored cast in situ piles have brittle structural response due to lack of flexibility and ductility when subjected to horizontal movement. The precast concrete piles of 400 mm size were adopted to carry the axial load.

The precast piles were designed as flexible vertical members to withstand the large lateral deformations that may occur during earthquake.

Axial Capacity

The design axial capacity of piles was achieved by driving the piles into hard clay stratum. After the required set was achieved the driving was continued till piles were penetrated further one pile diameter using 'Oslo' type shoe pin inserted at the pile tip as shown in Fig. 4. The piles were driven to impart an estimated compressive force of 2450 kN (250 tons) into hard clay. The hard clay was preconsolidated under this compressive force. The increase in the soil strength as a result of preconsolidation was hypothesized based on the following,

a) The hard clay would undergo hydraulic fracturing under the high stresses induced by the pile driving and result in the cavity expansion around the pile tip. This would result in to a network of cracks in the radial and other directions. The excess pore water pressure would dissipate within few days due to formation of cracks and soil would regain its shear strength.

b) The clay stratum, which experiences a large compressive force (estimated to be 2450 kN) during pile driving, is likely to undergo a very small settlement under the design load of 780 kN for which each pile was designed.

The long-term pile settlement was restricted as the pile was preloaded by overdriving beyond the design capacity. This hypothesis was verified by the settlements observed in pile load tests as shown in Fig. 5. The settlement of 5 mm was measured at 1300 kN (which is 1.5 times the design load). The settlement increased to 6 mm at the end of 48 hours from the time of the peak load application, which was, much less than the allowable settlement of 12 mm. A substantial component of the recorded settlement can be attributed to the elastic compression of the pile material. Therefore, it is evident that the low magnitude of the settlement is a result of preconsolidation of the hard clay stratum. Also the observed pile settlement in load test carried out at the end of 7 and 30 days were practically same.

The piles were provided with spiral reinforcement in pile head and tip to withstand high driving stresses. The confining effect of spiral reinforcement also prevents cracks if pile undergoes a combination of axial and flexural stresses.

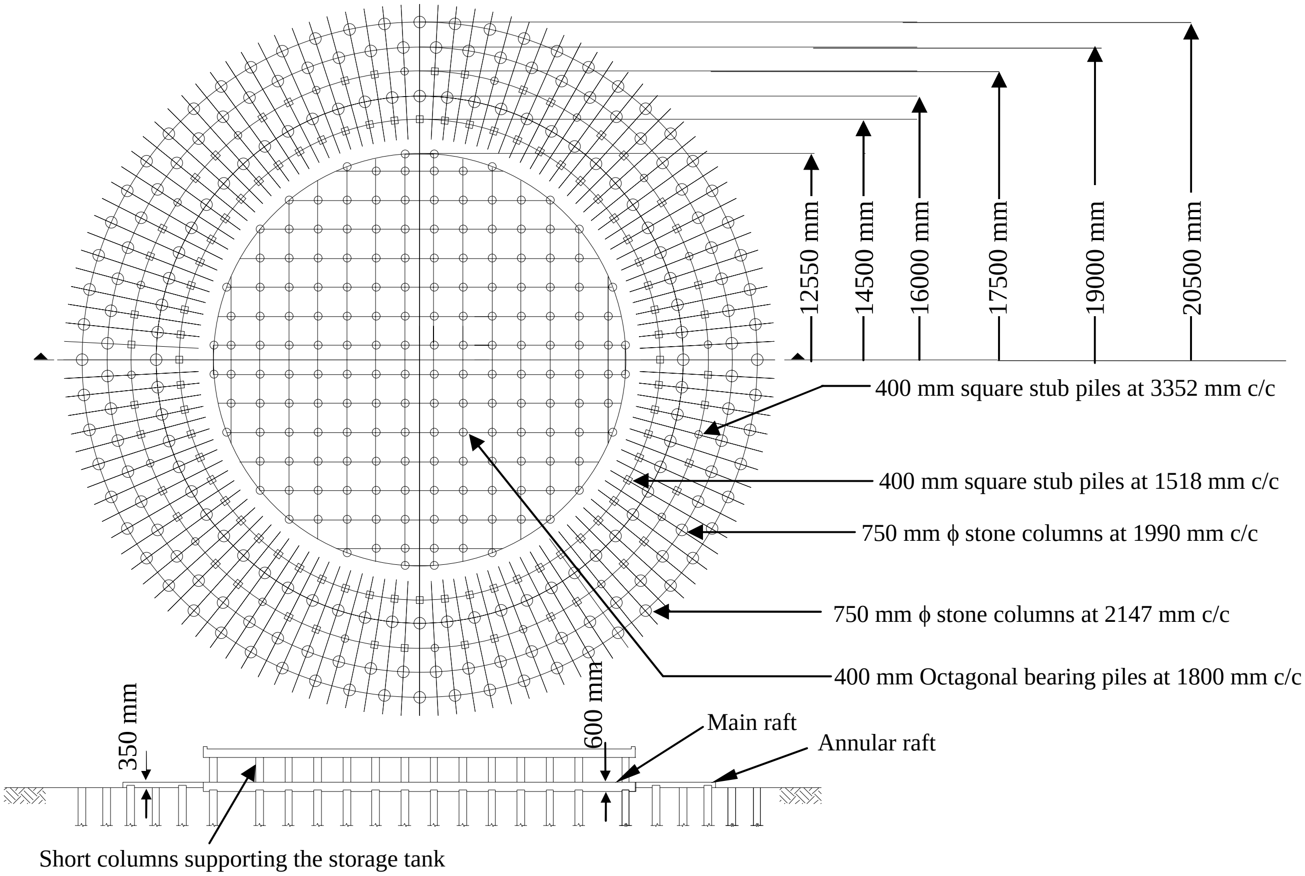


Fig. 3 Foundation system for phosphoric acid storage tank

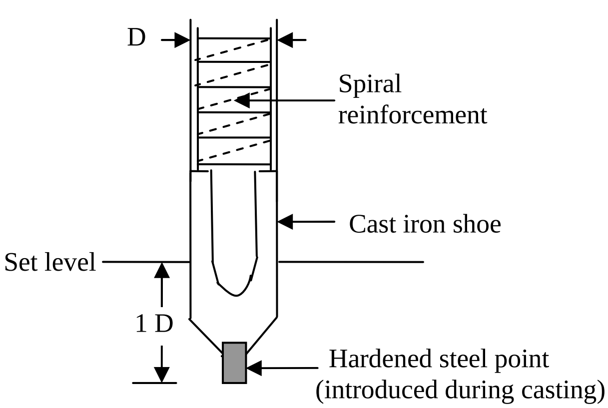


Fig.4 Details of pile toe

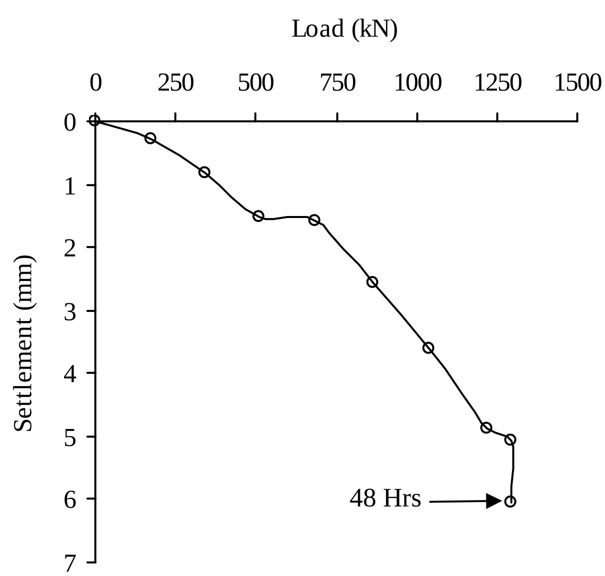


Fig. 5. Results of pile load test

Lateral Capacity

The lateral capacity was achieved using three types of foundation systems as described in the following section.

Piles Installed in Improved Ground. If the foundation is designed to provide ductility, a large amount of energy can be absorbed during the period of a very strong earthquake (Penzien, 1970). Energy absorption or work capacity is a more meaningful measure of earthquake capacity of most structures than the strength alone (Blume, 1960, and Housner, 1956). If there is no ductility and if there is no alternate stress path, the structure collapses under continued earthquake demands of similar or greater amount. On the other hand, ductility and reserve inelastic energy capacity and /or alternate or stronger stress paths may be available and may save the structure (Blume, 1970). The large diameter piles cannot provide such ductility when the surrounding clay medium moves during the earthquake. In such situations a small diameter piles would have an advantage over the large diameter piles as far as flexural stresses are concerned. In this view, the system of following components was adopted for foundation and as shown in Fig.3,

a) 400 mm octagonal precast concrete piles to support the storage tank
b) an annular raft supported on 400 mm square stub piles (of short length) to resist the lateral loads due to ground movement
c) isolation of annular raft from the piles supporting the storage tank.
d) ground improvement of the soil using 750 mm diameter stone columns to restrict the soil movement surrounding the piles supporting the main storage tank, during the event of an earthquake

The stone columns improved the surrounding ground, which would also act as drainage channels for quick dissipation of excess pore pressures, which may develop during the earthquake. The stone columns also result in the better interaction between the laterally displaced soil and piles supporting the annular raft. Stone columns being a stiff element would prevent collapse of soil structure liable to liquefy. The stone columns also contribute to the shear strength of the soil mass and thereby reduce the displacement of the top layer, relative to the bearing stratum. The improved soil in the annulus enhances the lateral capacity of the stub piles. The annular raft supported on short piles was designed to take the most of the lateral load, which may result from the soil displacement. The annular raft and the main raft supporting the storage tank were at the same elevation but separated by a flexible bituminous material to prevent any vertical load transfer by shear to the main raft. The storage tank was placed above short columns fixed on the main raft. A hinged connection was used between the top of main piles and the raft as a result of which, no bending moment would be created at the pile top. As a consequence, the pile was behaving with hinges both at top and at bottom and was free to move so that any relative movement between pile and raft would not result in stresses in pile.

Piles connected to Diaphragm Wall. The diaphragm walls were used in places where a wall of sufficient face area could be constructed to mobilize the required passive resistance. The pile caps were connected to the diaphragm walls, which were designed to transfer the base shear to the surrounding soil through passive resistance. In this case, the displacement required to mobilize the passive resistance was much less than the anticipated ground movement of 150 mm.

Raker Piles. Raker piles were used for some structures (which were away from the creek) to take the lateral loads arising from the ground movement during the first phase of the plant. The raker piles were designed for interaction forces resulting from pseudo static analysis of superstructure and foundation. No specific estimate of interaction forces between soil and pile was done. However, the subsequent assessment of piles with respect to lateral forces during phase II showed that if the piles were to be subjected compressive loads higher than the base capacity, it would result in yielding of pile into the bearing layer without causing axial stresses larger than the design structural strength of the pile. The piles were designed for a large margin in respect to the structural strength compared to the ultimate bearing capacity. The piles experiencing the tensile forces (as a result of horizontal movement) were not critical for design. However due to uncertainties involved in this method of analysis and lesser reserve capacity with respect to lateral resistance, the option of raker piles was dropped during phase II. In the phase II, vertical piles installed in improved ground were used extensively.

POST EARTHQUAKE PERFORMANCE

The settlements of all critical structures were monitored over the period of 25 years as a part of routine plant maintenance, and no significant relative settlement was observed. However, there was a concern regarding the safety of foundation and the superstructures after the Bhuj, 2001 earthquake. The post earthquake performance check was carried out which included the exhuming of 4 m of top soil around piles in the area close to the phosphoric acid storage tanks and non-destructive testing of piles. The main load bearing piles of phosphoric acid storage tanks were not accessible for the inspection. However, the piles supporting the phosphoric acid storage tank were installed in the improved ground and there were no signs of cyclic mobility observed around these tanks after the earthquake. Hence, it was evident that the piles would not have suffered any damage during the earthquake. Therefore, the post earthquake inspection was carried out on piles where the diaphragm wall and raker piles were used to resist the lateral loads.

In all six piles were exposed by excavating the top soil since the piles are subjected to the maximum flexural stresses near pile head when there is any substantial lateral soil movement. The visual inspection of piles showed no sign of cracks and corrosion over 20-25 years of operation in saline environment. In addition to visual inspection, integrity tests were carried out on these piles to check the damage at deeper level. All piles were found to have acceptable integrity. The pile concrete was found to be of satisfactory to excellent quality. The raker piles have performed well, although the piles were designed based on pseudo static analysis with an element of uncertainty regarding anticipated ground movement and the reserve capacity of pile.

CONCLUSIONS

Long flexible piles driven in hard clay performed well even in earthquake of magnitude 7.7. The soil improved with stone columns provided better interaction between displaced soil and piles supporting the annular raft. A system of annular raft supported on short piles driven in an improved ground was successful in taking the substantial amount of lateral load, keeping the main piles supporting the tank free from undue shear stresses. The entire foundation system was found to be intact in the post earthquake investigation at the site. The foundation system using piles installed in the improved ground and the piles connected to diaphragm wall have an edge over the raker pile system as a substantial lateral resistance can be mobilized at a relatively small horizontal deformation and consequently gives a reserve capacity to the foundation.

ACKNOWLEDGEMENT

The major contribution made by Late Dr. S. D. Bondre, Structural Consultant and partner of the first author, in development of design concepts used in foundation system is gratefully acknowledged. The literature relevant to this paper was provided by Mr. Jay M. Joshi a post-graduate student working on this topic and guided by Prof. Mhaiskar, Sardar Patel College of Engineering, Mumbai.

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