Sustainable foundation solutions for industrial structures under earthquake conditions – theory to practice

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ABSTRACT

The choice of suitable foundation system below an industrial structure is primarily governed by underlying soil strata, liquefaction potential considering the intensity of an envisioned earthquake and the importance or service life of the structure itself. This paper presents five different case studies which particularly highlights the engineering judgement being adopted for proving sustainable foundation solution to various industrial structures. First four cases demonstrate the efficient use of combined pile-raft foundation as a suitable foundation choice. The proposal advocated the use of performance based design approach where the individual capacities of the foundation components were assessed and utilised leading to considerable cost saving. The numerical analysis using various computation techniques helped in understanding the pre-failure response of the soils supporting these foundations and obtaining the structure's performance. The last case study describes the cost-efficient innovative practice of foundation system in combination with ground improvement techniques for Petroleum-Oil-Lubricant (POL) terminal facilities that are proposed to be constructed in originally liquefiable soil at Motihari, India.

Keywords: Piled-raft foundation, in-situ investigation, finite element method, case study, liquefaction, earthquake.

1 INTRODUCTION

Industrial structures play an important role in overall strategic and economic development. These include power plants facilities, refineries, airports, storage shelters and other places of national and international importance. The foundation of industrial structures such as oil tank foundation, reactor building of a nuclear power plant, industrial facilities storing flammable liquids or hazardous minerals and others may fail during an earthquake event due to excessive differential settlement and rotation of foundation with development of an unacceptable drift ratio for the supporting structures. Circular tank foundation encounters elephant's foot buckling failure modes in case of excessive foundation settlement. This may be due to an increase in the vertical compression load of the tank, sloshing phenomenon and reduction in the strength of the underlying soil due to liquefaction. Huge loss of lives and properties were reported due to uncontrolled fire broke after the failure of storage tanks during 1964 Niigata (M_w=7.6), 1964 Alaska (M_w=9.2), 1971 San Fernando (M_w=6.6) and 1999 Kocaeli (M_w=7.6) Earthquake. Although nuclear power plant is typically designed considering an earthquake hazard for the return period of 10,000 years, an unacceptable differential settlement may cause cracking in the containment structures storing the radioactive materials. The leakage of radioactive water after 2011 Tohoku earthquake in Japan posed huge threat in the region and raised serious concern to the countries lying in the seismically active zones. With the enormous extent of damages observed over the years, it is extremely important to include the influence of seismic forces in the analysis and design of industrial structures to safeguard them from the anticipated seismic hazards.

Recent trend in the foundation design poses a principal question on how to provide a sustainable design solution by minimizing the deformation of adjacent structures and the structure in question by adopting a performance-based approach. In case of encountering soft soil strata, a conventional approach of using pile foundation is still dominant in the engineering practice due to the imposition made by the codes and regulation in many countries. The recent development in the practice of performance-based design approach by using computer technology followed by few instrumented field observations from the sites proved that the capacity based design approach is too conservative mainly because of its inability to incorporate the intermediate capacity of the foundations and to understand the pre-failure response of the supporting soils. The question of how many piles are required to support the weight of the building is now turned to how many piles are actually needed to reduce the settlement to an acceptable limit. The approach where the capacity of both raft and piles are accounted to fulfil the static and seismic demand of the structures has received attention throughout the world and is named as Combined Pile-Raft Foundation (CPRF) or piled raft foundation. This foundation system has recently been used below several high-rise buildings and its evidence can be found out in the literature (Poulos and Davids 2005; Yamashita et al., 2011; Katzenbach et al., 2016). The design of CPRF proposed in the seismically active region requires calculation of the dynamics loads acting at the foundation level and the estimation of the natural frequency of the superstructure. Different methodologies are available to analyse the building foundation system subjected to combined (static + seismic) loading conditions. The most popular is converting the earthquake load to an equivalent static lateral load and applying it to foundation components called pseudo-static approach. One such example of the use of pseudo-static approach for piled raft foundation was reported by Kumar et al. (2016). However, the pseudo-static approach does not consider the earthquake characteristics, namely, frequency and duration of earthquake motion and natural frequency of soil media supporting the foundation. This often plays a significant role in dictating the overall behaviour of the foundation system. The structure can be subjected to very high seismic force if the natural frequency of the soil-structure system comes in the near zone of the earthquake exciting frequency called attainment of resonance condition (Roy et al. 2018). At the design stage, the likelihood of such an undesirable condition can be identified by performing the site-specific response analysis considering dynamic soil properties, thereby adopting the appropriate foundation system to safeguard the superstructure.

This paper presents five different case studies of the application of sustainable foundation solutions to industrial structures. Out of five, first four cases demonstrate the efficient use of CPRF in various important structures, namely, raw material storage building in Vietnam, an oil tank in Iraq, industrial plant building in Uttarakhand, India and reactor building of a Nuclear Power Plant (NPP) in Haryana, India. The last case study illustrates the cost-efficient innovative practice of foundation system in combination with ground improvement techniques for oil tank and (Liquefied Petroleum Gas) LPG storage mounded vessels that are proposed to be constructed in originally liquefiable soil in Northern Part of India.

2 CASE STUDIES

2.1 A raw material storage building in Vietnam

The CPRF system (81 m x 55.5 m raft supporting 581 piles of length 20 m and diameter 0.4 m) was adopted as a foundation system for raw material storage building of NPK fertilizer plant. The building is located in Phu My fertilizer plant in Ba Rai Vung Tau Province of Vietnam. The storage facility consisted of 12 compartments, 6 on each side, of width 10.75 m and height 7 m. The foundation proposal takes into consideration of the presence of soft clay to medium dense sand strata at the site and different loading intensity scenario varying from 24 kPa to 75 kPa. The feasibility of using un-piled raft foundation was first assessed which gave an unacceptable value of differential settlement (120 mm) leading to the choice of an alternative foundation system. The possibility of using CPRF was then assessed in a view to provide cost-effective foundation solution. The pile-raft and raft-pile interaction factors were obtained using the available equations in the literature. The load sharing by piles in CPRF (α_{CPRF}) were then obtained by using the available closed-form solutions and obtained as 0.8. Thereafter, the foundation system comprising raft, piles along with the material being stored was modelled by using finite element based computer program PLAXIS3D to check the serviceability of the foundation system, as shown in Figure 1. The detailed modelling and design are reported in Kumar et al. (2017).

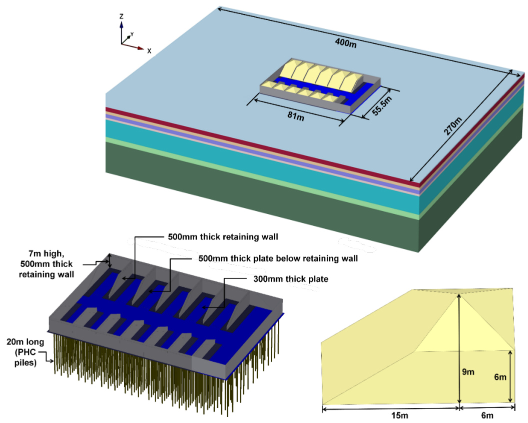

Fig. 1. 3-D developed model of storage building in Vietnam using PLAXIS3D (Kumar et al., 2017)

The developed model was subjected to different loading scenarios ranging from the all compartments full of materials to the compartments full only on one side. The settlement, differential settlement, axial loads in piles and contact pressure in raft were obtained and assessed about the feasibility of CPRF. A maximum vertical average settlement of 30 mm was observed under the vertical pressure of 75 kPa when all the compartments full case was simulated. The obtained value of settlement were within the permissible settlement limit for CPRF reported by Yamashita et al. (2011) that is 30 mm to 50 mm. The obtained axial load for all the cases indicated the minimum values of axial load at the centre of the foundation to a maximum value at the edges of the foundation footprint. The primary reason for this behavior is the development of negative pile-raft interaction where the presence of raft reduces the mobilization of axial resistance in the piles at the lower settlement range at the center of the foundation. This is unlike the case of the mobilization of resistance at the edges. The load sharing by raft ranged from 23% to 30% which was in accordance with results reported by Cooke et al. (1981) (23%), Russo and Viggiani (1995) (20%). The complete analysis and design of foundation system involving complex interaction along with the modelling of storage structure and material being stored provides confidence in the response of the building throughout its intended service life.

2.2 Oil Tank Foundation for the Kafza site in Iraq

A sustainable foundation solution comprising of 24.1 m diameter raft of thickness 1.5 m connected to 89 piles of length 26 m and diameter 0.8 m was proposed for oil tank of diameter 23.15 m and height 15 m constructed in the seismically active region of Kafza, Iraq. Standard penetration tests were carried out at the site which revealed fine to medium dense sand fill up to 2 m. The recorded SPT-N_{60} was 48. Below this, a very soft clay layer exists up to 20 m depth having SPT-N_{60} value ranging from 2-7 as shown in Figure 2. A very stiff clay layer of thickness 4 m with SPT-N_{60} value of 49 followed by dense to very dense sand layer to a greater depth were recorded.

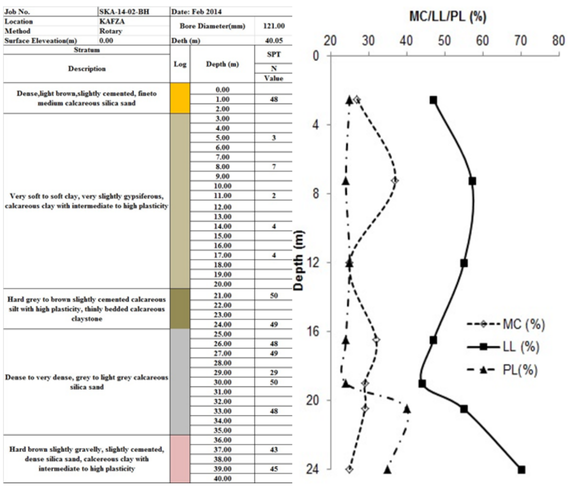

Fig. 2. Typical soil profile, penetration resistance and consistency limit for the Kafza site in Iraq

The in-situ condition of the construction of oil tank foundation was simulated in FLAC3D v4.0 (Itasca, 2009) where soil was modelled by Mohr-Coulomb constitutive relation and piles by structural elements and raft by shell elements. The piles were modelled to carry the load through skin resistance and end bearing. Interface element were defined between shell element of raft and soil. Upon validation of the numerical model by field pile load test data, the entire foundation system was simulated and the response of the foundation system was obtained under static loading condition. The vertical loads were converted to vertical pressure and applied at the raft level. The horizontal loads were applied at the centre of gravity of the raft. The load sharing by pile component was obtained and represented by CPRF coefficient (α_{CPRF}) i.e. ratio of load carried by piles to the total load acting on the foundation system. The obtained value of α_{CPRF}, 55800/62000, was 0.90 which was in accordance with the percentage of load shared by pile that was reported by Yamashita et al. (2011) for 47 storey residential tower in Japan (91% load sharing by piles). The static analysis results indicated that the axial forces in piles and settlements of the entire foundation system were within the permissible limits.

The foundation system was then analysed under seismic loading condition based on the developed synthetic time-history for the return period of 2475 years relating to Maximum Credible earthquake. The time-history was developed based on the analysis result of Probabilistic Seismic Hazard Assessment (PSHA) as per the procedure laid out in Desai and Choudhury (2015). The developed acceleration time history (Peak Horizontal acceleration- 0.15g, duration 54 sec) was applied to the base of the soil model. The free-field boundary condition was assigned to model the infinite medium in FLAC in order to prevent incident wave reflections. Figure 3 illustrates the application of free-field boundary condition to the developed soil-foundation model. More details about the modelling procedure and analysis steps can be found in Kumar and Choudhury (2016). To capture the stiffness degradation of soil under seismic loading condition, the Finn and Byrne model was assigned to the soil which is able to capture the volumetric straining during seismic excitation.

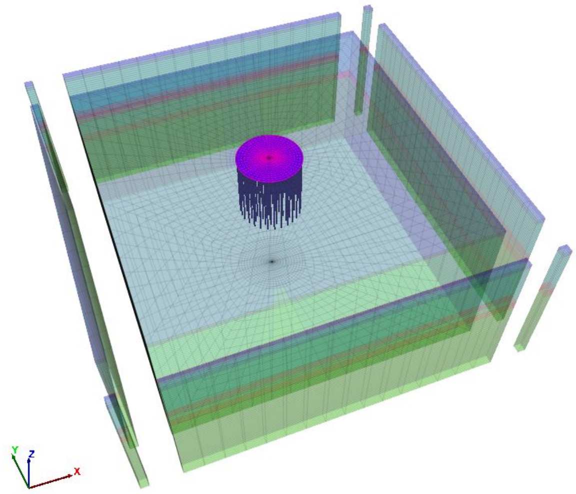

Fig. 3. Three-dimensional view of foundation model in free field condition at the Kafza site in Iraq

The values of excess pore-water pressure ratio, r_u were obtained at different depth viz. 5 m, 10 m, and 15 m below the normal ground level. The value of r_u at the depth of 5 m were obtained as 0.85 at the end of 54 second which indicated the reduction in the shear strength of soil mass due to an increase in the pore-water pressure. The value close to one indicates near full cyclic mobility of the soil. The value of r_u at depths of 10 m and 15 m were obtained as 0.65 and 0.55 which further reduces with an increase in the soil depth. This was due to the increase in the confining stress of the soil with an increase in the soil depth. The axial loads in the piles were observed to increase substantially (697 kN during static case to 1690 kN under seismic loading) due to the stiffness degradation of the soil mass. The bending moment and shear forces in piles were also observed to increase during the combined static and seismic loads. However, they were within the permissible limits. The rotation in foundation was obtained as 5.81×10^{-4} which was lower than the permissible limit for oil storage tanks indicating a satisfactory performance during the earthquake loading. The current study signifies the importance of modelling the combined static and seismic condition by developing a three dimensional numerical model for obtaining the overall response a foundation system that can capture the static and dynamic soil-structure interaction.

2.3 Remedial foundation solution for an industrial building in Uttarakhand, India

This study provides the details of the remedial measures and corrective solutions suggested for constructed group piles foundation supporting industrial buildings in an industrial zone of Uttarakhand, India. The industrial structure includes the drier building area, chiller office area and tank farm area. To obtain the ground information, SPT, seismic cross hole test and CPT were carried out at site. The structures were designed as per relevant Indian Standard codal guidelines and then constructed. The pile integrity testing carried out at the site pointed serious construction malpractice in terms of short concreting in piles, unavailability of the reinforcement at the pile cut-off level and even the absence of a pile at just below the footprint of one tank. The geotechnical design of each of the structural members were altered considering the structural demand and re-analysis were carried using finite element based computer program PLAXIS3D to check the stress resultant in the foundations and settlement obtained.

A typical borelog data of the site shown in Figure 4 indicated the variation of SPT values from 10–50 having soil type as clayey silt to sandy slit layers with few interbedded layers of sands. Considering the seismic safety of the site, the liquefaction potential for the site was calculated by simplified procedure, as described in NCEER Summary report (Youd and Idriss, 2001) and it was found that the soil up to 4.5 m depth is prone to liquefaction for an earthquake of magnitude 6.7 and peak ground acceleration 0.24g for seismic zone IV of IS 1893-Part 1(2016).

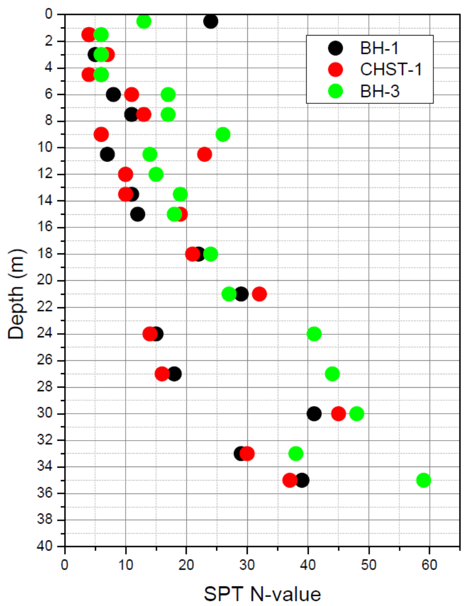

Fig. 4. Variation of SPT values along soil depth at Uttarakhand site in India

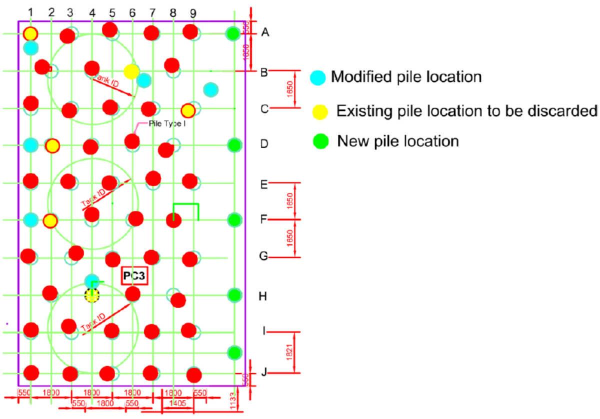

Fig. 5. Schematic representation of piles with modified pile locations, exclusion of defective piles and addition of new piles at Uttarakhand site in India

This paper explains the analysis of one of the industrial building i.e. tank farm area comprising a pile cap of length 21.5 m and width 8.3 m that connects 46 piles of length 18 m and diameter 0.6 m. The proposal takes into consideration of in-situ site-specific soil profile, expected earthquake hazard, liquefaction assessment and loading scenario. Figure 5 illustrates the original design drawing showing the exact pile location, it also shows the location of the misaligned piles and new pile locations cater to the existing structural demand. A total of 11 new piles of same diameter and length were added and 6 defective piles were discarded. The foundation system was subjected to static and seismic loading conditions. Figure 6 illustrates the vertical settlement contour indicating lesser differential and average settlement as identified for group pile by IS 2911 (Part 1): 2013. This study indicates the importance of engineering judgement to requalify the geotechnical structure with minimal changes in the proposed design.

Upon completion in re-design and considering the past experience of poor quality of construction, pile integrity testing was recommended on all the piles constructed to ensure the safety of superstructure. Brief description of these two tests is given in the following section.

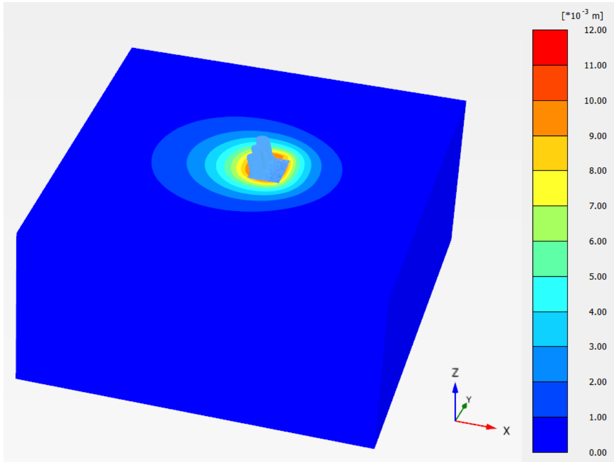

Fig. 6. Vertical settlement (in m) contour under combined static and earthquake loading conditions obtained using PLAXIS3D for Uttarakhand site in India.

2.3.1 Pile Integrity Test (*PIT*)

In the present study, PITs were performed on 165 cast-in-situ reinforced concrete (M30) bored piles of diameter 0.6 m, length 18 m . All the field tests were conducted in accordance with ASTM D5882 (2016). Wave speed (*c*) was varied from 3400m/sec to 4200m/sec. The testing program revealed a total number of 34 piles were identified as defective piles as they got early toe response. It is imperative to note that the pile integrity up to the toe response is found satisfactory, it is possible to utilize these piles as pile shaft of 12 m long or higher shall offer considerable pile resistance. Therefore, the allowable vertical capacity of all 34 defective piles was calculated according to IS 2911 (Part 1): 2010 for the corresponding available lengths of these piles. Thereafter, comparing these capacities with the loads on respective piles imposed by the superstructure and computed by using finite element based computer program PLAXIS3D. With this comparison, it was found that 5 piles are getting marginally overloaded. Hence, it is essential to perform additional High Strain Dynamic Test (HSDT) on these 5 overloaded piles to check for further safety concern of building, if any, and additional 2 piles as confirmatory. So, altogether 7 numbers of HSDT was further recommended.

2.3.2 High Strain Dynamic Test (*HSDT*)

The main aim of HSDT is to evaluate pile static capacity, structural integrity, and measure the settlement by applying an axial impact force with a pile driving hammer or dropping a large weight at the top of the deep foundation unit. The response in terms of both force and velocity at pile top is derived from strain and acceleration data which is recorded with the help of two strain transducers and two accelerometers. More details about the test and analysis can be found in Chatterjee et al. (2015a and b; 2019). HSDT was performed on seven numbers of the cast in situ bored piles which were identified as marginally overloaded piles by examining PIT reports. All the tests were conducted in accordance with ASTM D4945 (2017). The instrumentation of HSDT which were used in the present study is shown in the following Figure 7. The loading details, hammer drop and other relevant data used in the site are summarised in Table 1. Since the stiffness factor (characteristic length) of all tested piles are calculated as T ≈ 2.02 m which is less than the one-fifth of their respective pile lengths, hence, all piles are classified as long flexible piles (Das, 2004; Phanikanth and Choudhury, 2014).

Fig. 7. Instrument brought to perform HSDT test at the Uttarakhand site in India

The capacities of all tested piles are estimated from PDA and CAPWAP analyses are furnished in Tables 2 and 3. Table 2 dictates the maximum pile capacity (R_u), net settlement (S_n) and total settlement (S_t) as observed in PDA. In same way, Table 3 illustrates pile capacity (R_u), skin friction (R_s), toe resistance (R_b), maximum compressive stress (σ_c), maximum tensile stress (σ_t), maximum settlement at pile top (S_t), maximum energy transferred to the pile (E_{max}) and pile integrity factor (*IF*) as recorded in CAPWAP analysis.

Table 1 Loading details on the piles

Pile no.	Hammer weight (kN)	Height of fall (m)	Blow number*	Duration of travel [t_o] (ms)
P1		1.3	3	9.63
P2		0.5	1	9.63
P3		1.5	3	9.25
P4	24.52	0.5	1	9.95
P5		0.8	1	9.25
P6		0.5	4	9.25
P7		1.3	3	9.23

*Used for CAPWAP analysis

Table 2 Summary of PDA analyses results

Pile no.	R_u (kN)	S_n (mm)	S_t (mm)
P1	2110	2.2	4
P2	2000	1	2
P3	1950	3	10
P4	1470	0.5	3
P5	1620	1	2
P6	1490	2	5
P7	1610	2	9

Table 3 Summary of CAPWAP analyses results

Pile no.	R_u, R_s, R_b (kN)	σ_c, σ_t (MPa)	S_t (mm)	E_{max} (kJ)	IF^* (%)
P1	1663,1332,331	22.2, 9.65	11	18.31	81% from 17 m to pile toe
P2	2188,1781,407	11.7, 3.30	5	4.98	70% between 14 m to 15 m
P3	988, 727,261	21.9, 6.32	12	20.70	11% to 61% from 13 m to pile toe
P4	1402,1175,227	10.2, 5.26	3.5	4.10	42% to 77% from 13 m to pile toe
P5	1313,1117,196	15.0, 3.36	7.1	6.38	46% to 57% from 14 m to pile toe
P6	1111,809,302	15.5, 3.65	4.8	7.06	29% to 85% from 12 m to pile toe
P7	1081,867,214	19.3, 6.44	11.6	16.83	15% to 82% from 14 m to pile toe

*Below test level

Once the capacity of piles is derived, then the next step was to compare these capacities with loads acting on these respective piles and compute the factor safety which is given in Table 4. It can be seen that piles offered sufficient resistance and the achieved factor of safety is much higher than that prescribed by Indian Standard IS 2911 (Part 1/Section 4): 2010. Hence, these piles are considered as good piles and included in the pile foundation system with recommendations; (1) if any pile requires building up to raise the pile up to pile cut off level shall be carried out under strict supervision of responsible site engineer, (2) In case, reinforcement need to be welded to ensure sufficient development length embedded in the pile cap, same shall be carried out as per drawing and specifications under strict supervision of site engineer.

Table 4 Estimation of factor of safety

Pile no.	Loads* (kN)	PDA Capacity, R_u (kN)	FoS	CAPWAP Capacity, R_u (kN)	FoS
P1	427	2110	4.9	1663	3.9
P2	495	2000	4.0	2188	4.4
P3	404	1950	4.8	988	2.4
P4	427	1470	3.4	1402	3.3
P5	494	1620	3.3	1313	2.6
P6	312	1490	4.8	1111	3.6
P7	396	1610	4.1	1081	2.7

*Load on piles is estimated by performing the pile group analysis using FEM based computer program PLAXIS3D for governing load case.

2.4 Foundation system for a reactor building of Nuclear Power plant in India

The potential future Nuclear Power Plant (NPP) is going to be set up at the North-western part of India in the state of Haryana, which is 200 km far from the state capital Chandigarh. According to the seismic zonation map of India, IS 1893 (Part 1): 2016, this area falls under the Seismic zone III, where earthquakes of magnitude up to 6 (Intensity of VIII) may be expected. The seismic control zone of a NPP area is considered as 300 km radius circle from the plant boundary (AERB, 1990; Desai and Choudhury, 2014a, b, c and d). The seismic activity of this control region is explored by Rao and Choudhury (2018) and stated that this region has witnessed six major earthquakes of magnitude greater than or equal to 6 in the last two centuries. One of the most devastating earthquakes of the last century, that is, 1905 Kangra earthquake (M_w 8) occurred in this zone, which caused the collapse of buildings over a large extent and resulted in an economic loss of 2.9 million rupees (Ambraseys and Bilham, 2000; Jain, 2016). Considering the seismicity and importance of the facility, the complete soil-structure interaction analysis is performed using finite difference based computer program FLAC3D v6.0 (Itasca, 2017).

The major highlight of this facility is that unlike the previous cases, this is the first instance when a NPP is going to be built on the entire soil stratum, rather than a rocky site in India. Due to this reason, the designing and detailing of a NPP needs to be done, not only for the execution of this particular power plant but also to recommend a generalized design methodology which can be followed in future for NPPs proposed to be constructed in similar ground conditions. Combined Pile-Raft Foundation (CPRF) has been decided as the foundation system to be used for the NPP. A number of iterations are performed for finalizing the layout of the raft and the number of piles along with their alignments. The whole NPP is subdivided into two major parts i.e. Reactor Building (RB) and Reactor Auxiliary Building (RAB). As soil-structure interaction (SSI) plays a pivotal role in governing the behaviour of the foundation system, detailed SSI analysis of the same is performed under both static and lateral loading conditions. In the first iteration, a total 576 number of piles are used, of which 272 piles of larger diameter and length were allocated below the raft in RB area and the remaining 304 piles of smaller diameter and length were assigned below the RAB region. The raft possesses a greater thickness in the RB region with an inclined edge along its periphery, to cater the effect of lateral load. This configuration evaluates a vertical pile load sharing of around 95% which indicates a raft-enhanced pile group as delineated by O'Brien et al. (2012). However, this alignment exhibits that the piles on the peripheral portion fail due to excessive shear force under lateral loading condition. To avert this undesirable scenario, in the subsequent design iteration, ground improvement has been done on the top 6 m soil layer around the peripheral region of the raft. The beneficial influence of ground improvement portrays a reduction in pile load sharing to 79% which indicates an increased amount of load sharing by raft in CPRF. However, the peripheral piles in the RAB region still experience higher shear force under lateral loading, which necessitates further alteration in the alignment of the CPRF system. In the next iteration, the area of the raft below the RB region is increased and shear key is introduced around the peripheral region of the raft in this region. This launch of shear key helped in withstanding lateral load more efficiently than previously. Also, the number of piles has been reduced to 271 where all the piles are designed to be clustered in the RB area which eliminates the possible occurrence of shear failure in the peripheral piles. Figure 8 portrays the isometric view of the numerical model of a proposed NPP along with its foundation system, generated in FLAC3D. This case study illustrates the significance of strategically located piles in CPRF and the advantageous inclusion of shear key in withstanding lateral loading along with the beneficial outcome of ground improvement for the satisfactory performance of CPRF as the foundation system below a complex structure like a NPP.

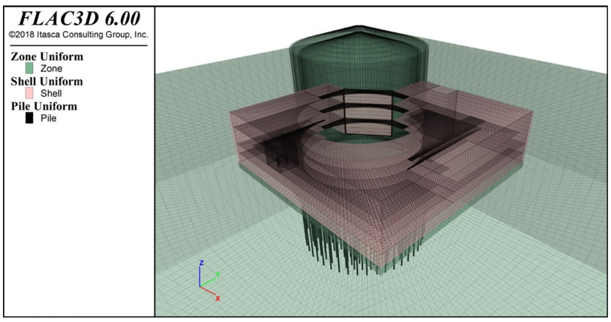

Fig. 8. Representative isometric view of a proposed foundation system for NPP with CPRF in India

2.5 Foundation system for Petroleum-Oil-Lubricant (POL) terminal at Motihari, India

The site of an on-going POL terminal project is located at Motihari in the Indian state of Bihar (as shown in Figure 9), which falls in the Indo-Gangetic plains and is classified as seismic zone IV as per IS 1893 (Part 1): 2016. In the recent 2015 Nepal earthquake, this region has witnessed severe damage to manmade structures. The site investigation revealed that the subsoil consists of top 3 m to 5 m fine grained clayey material underlain by loose to medium dense sand up to 30 m with ground water table close to ground surface and hence it has potential to liquefy in an event of an earthquake.

To eschew the economic and environmental loss to the proposed POL terminal in future earthquake events, the liquefaction analysis was carried out considering peak ground acceleration of 0.24g and a moment magnitude (M_w) of 7.6 (M_w is calculated from the intensity of an earthquake as per MSK scale for zone IV in IS 1893 (Part 1): 2016 and Das et al. 2019). The data obtained from 34 SPTs and 42 CPTs spread over an entire area were used to determine the liquefaction potential of the soil strata. The liquefaction assessment of the site has been carried out as per the procedure mentioned in IS 1893 (Part 1): 2016 for granular soils and criteria proposed by Boulanger and Idriss (2006; 2007) was used for clay like fine grained soils. The minimum target factor of safety of 1.1 is adopted for liquefaction mitigation.

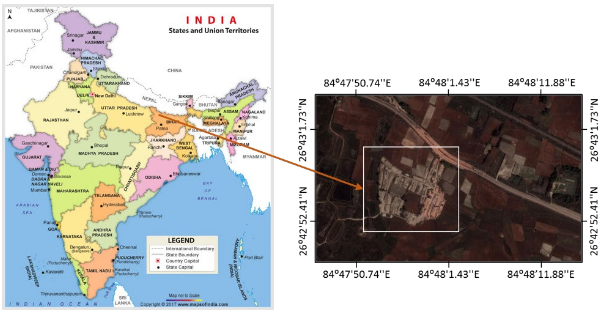

Fig. 9. Location of POL terminal site at Motihari, India

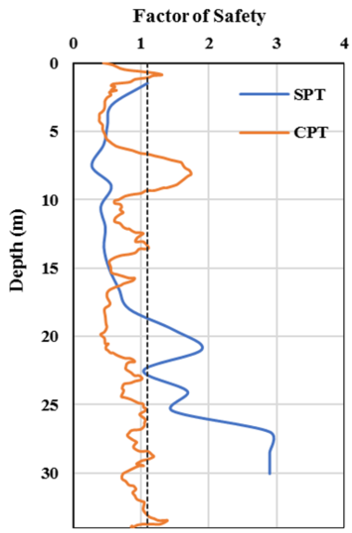

Fig. 10. Variation in safety factors of liquefaction with depth for Motihari site in India

The depths of liquefaction based on data of SPTs and CPTs were found out to be 23 m and 33 m, respectively. From Figure 10, it can be seen that soil is critically susceptible to liquefaction across the depth. Proposed POL terminal contains oil tanks, underground storage facility, mounded storage vessels (MSV), administrative control unit, fire pump house, LPG bottling plant and other buildings. The feasibility study of foundation system of oil tank and mounded storage vessels (MSV) were carried out by using finite element based computer program PLAXIS3D v2017.01. Figures 11 and 12 show the typical three-dimensional model of oil tank foundation and MSV foundation system, respectively. Soil materials are modelled using Mohr-Coulomb (MC) model, structural elements like concrete retaining wall, MS tanks, etc. are modelled using conventional Linear-Elastic (LE) constitutive model. Oil storage tank having diameter 28 m and height 20 m is rested on ring foundation. There are total three mounded vessels each having length of 66.66 m and diameter of 7.26 m. The clear distance between two vessels is 2 m. Height of mound is 10.41 m and it is supported by retaining wall from all the sides. The retaining wall is of height 10.8 m and varying thickness of 0.3 m at top to 1.5 m at bottom. Prior to construction of vessels and mound, the filling of 1.5 m is carried out to raise the ground level followed by the 1.5 m of engineered filling. The analyses were carried out under the effect of self-weight and hydrostatic loading. In case of oil tank, effect of ground remediation using cement grouting up to a depth of 9 m has been investigated and settlement was compared with the case of existing unimproved soil.

Under the most critical load combination, total maximum settlement just below the bottom of plate of tank in unimproved soil is found out to be 532 mm and in case of improved ground is 471 mm. Net settlement (after deducting the settlement during construction process) at centre and edge of foundation in unimproved is 383 mm and 337 mm, respectively, whereas in improved ground net settlement at centre and edge of foundation is 331 mm and 289 mm, respectively. It can be clearly seen that cement grouting technique is not adequate to control the settlement within allowable limit of 150 mm. The performance of MSV foundation system is evaluated by serviceability approach (settlement criteria). In this approach, maximum and differential settlement under various loading considerations are calculated. All loading combinations and results obtained under respective loading conditions are listed in Table 5. Figure 13 illustrates the deformation contours in MSV under loading case 1 (all vessels are empty). It can be seen that the proposed MSV system experiences a large total settlement in almost all loading scenarios.

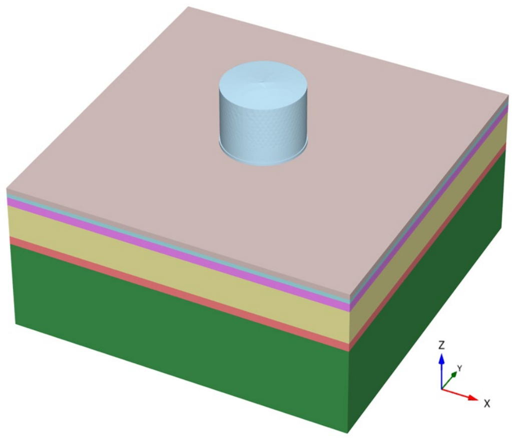

Fig. 11. Three-dimensional view of developed numerical model of oil tank foundation system in PLAXIS3D for Motihari, India

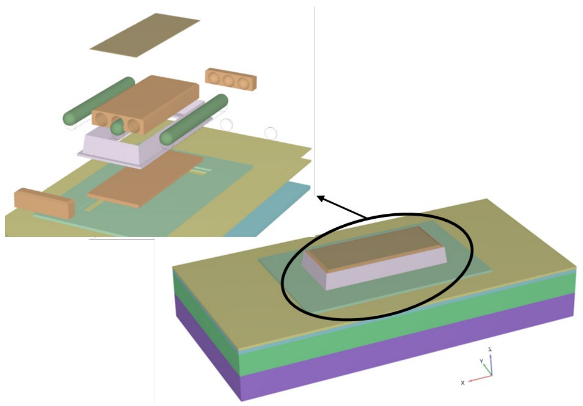

Fig. 12. Three-dimensional view of developed numerical model of MSV foundation in PLAXIS3D for Motihari, India

The permissible post hydrotest total settlement is 100 mm and differential settlement is 1 in 200. Numerical analysis results revealed that the obtained settlement under various loading considerations are more than the permissible limit, and hence unsafe. Therefore, it was concluded that a robust solution is needed to safeguard the foundation system of various facilities of entire POL terminal.

Table 5 Settlement in MSV foundation system in hydrotesting at various loading conditions for Motihari site in India

Loading case	Vertical settlement (in mm)		Differential settlement (in mm)
1 – All vessels are empty	Total in MSV	206	0
	At sand bed level	198	
	At Final ground level	185	
2 – 1st vessel is full, 2nd and 3rd are empty	Total in MSV	231	31
	At sand bed level	203	
	At Final ground level	222	
3 – 2nd vessel full, 1st and 3rd are empty	Total in MSV	231	30
	At sand bed level	226	
	At Final ground level	209	
4 – 3rd vessel is full, 1st and 2nd are empty	Total in MSV	230	31
	At sand bed level	221	
	At Final ground level	203	
All vessels are full	Total in MSV	258	Vessel 1 = 32
	At sand bed level	255	Vessel 2 = 25
	At Final ground level	233	Vessel 3 = 32

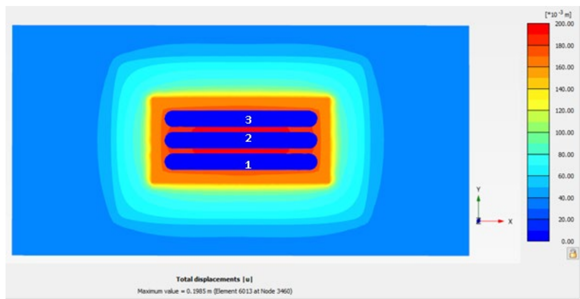

Fig. 13. Deformation contour of MSV in loading case 1 at sand bed level along with vessels for Motihari site in India

Based on the review of liquefaction assessment of subsoil profile, loading conditions of superstructures and outcomes of numerical investigation, the major geotechnical challenges in designing foundation system were liquefaction mitigation and controlling post construction long term settlement within allowable limit. Hence it is decided to execute ground remediation work. Ground improvement technique using combination of vibro compaction and vibro stone column is employed to mitigate the liquefaction potential, to reduce the excessive settlement and to enhance the bearing capacity of ground supporting oil tanks, industrial sheds and underground facilities. Optimum grid spacing of 2.75 m in triangular pattern for both stone columns and vibro compaction was obtained from field trials. Based on the confirmatory soil investigation data and technical analysis, the maximum depth of treatment will be in the range of 24 m to 30 m below working platform level. Depth of 7 m for vibro stone columns is considered based on silty clay layer depth and transition layer from silty clay to sand. The treatment scheme is designed considering minimum area replacement ratio (ARR) of 20% for vibro stone columns.

3 CONCLUSIONS

This paper briefly presents five (5) case studies highlighting the sustainable design approach for the foundations of various industrial structures considering the soil variability, seismic hazards, loading scenarios and the importance of proposed structure. The analyses were carried out by using finite element based and finite difference based computer programs simulating the in-situ soil condition, earthquake hazard and different loading scenarios to be experienced by the structures during its service life. The proposed foundation system for raw material storage building in Vietnam shared 30% of its total load to raft component and remaining are taken by underlying piles. The foundation for an oil tank in Iraq was developed considering the site specific earthquake hazard and soil types. The remedial measures and the corrective solution applied in a constructed pile foundation for proposed storage tanks in Uttarakhand, India. For complicate structures like NPP, the importance of using CPRF with strategically places piles allowing with the beneficial inclusion of suitable ground improvement technique is exhibited. The study presented a corrective engineering solution to the foundation of the proposed tank farm area by adding piles based on the engineering judgement. The decision of the adopted foundation system for oil storage terminal lies on the soil types and envisioned seismic hazard. The liquefaction review hinted a liquefaction susceptible zone up to 30 m. The performance of foundation system of oil tanks and mounded storage vessels was investigated using numerical analysis and observed unacceptable settlements leading to recommendation of vibro compaction coupled with vibro stone columns techniques to reduce the settlement to an acceptable level.

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