ON THE PROBABLE CAUSE OF THE FAILURE OF KANDLA PORT AND CUSTOMS OFFICE TOWER DURING THE 2001 BHUJ EARTHQUAKE

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ABSTRACT :

This paper presents a case study on the 22m high six-floor Port and Customs Office tower in Kandla port that tilted during the 2001 Bhuj earthquake (India). The tower building was founded on 32 cast-in-situ concrete piles. As the piles were closely spaced, the pile caps were connected together and were acting like a foundation mat. The geotechnical study of the soil at building site shows that the 12m sandy soil layer overlaid by 10m of clayey crust would have been liquefied during the earthquake. Based on the post-earthquake damage investigation of the building, it is calculated that the tip of the pile settled by about 0.45m. Conventional axial load transfer analysis and lateral spreading analysis, considering the degraded strength of soil during earthquake, could not predict the actual failure pattern of the building during earthquake. It has therefore been considered that the foundation mat would also have shared the load of the superstructure. A detailed analysis of the foundation, considering mat-pile-soil interaction, has been carried out whose results are able to describe the tilting of building up to certain extent. This study suggests that the piles passing through non-liquefiable laterally spreading crust and terminating in liquefiable deposit is not a good practice. However, the use of the foundation mats reduces the risk involved in the building from sudden collapse. Settlement of these foundations reduces the axial load acting on the pile as it shares a significant amount of load while the whole building tries to sink into the soil.

KEYWORDS: Earthquake, Pile foundation, Liquefaction, Lateral spreading, Settlement, Mat-pile-soil interaction

1. INTRODUCTION

The Bhuj earthquake that struck the Kutch area in Gujarat at 8.46am (IST) on January 26, 2001, with a magnitude (M_w) of 7.7 was one of the major earthquakes in India that caused extensive damage to the built environment. Along with other cities of Gujarat, Kandla also experienced significant damage. Kandla, located at the mouth of little Rann of Kutch on the south eastern coast, is about 50km from the epicenter of the 2001 Bhuj earthquake. Many pile-supported buildings, warehouses and cargo berths in the Kandla area were damaged during the earthquake.

The present study analyses the failure of a 22m high six-floor building called the Port and Customs Office Tower (which will be referred as “Building” in rest of the paper) located in the Kandla port area very close to the waterfront. This pile-supported building leaned about 30cm at its top and separated from its adjacent building. Figure 1 shows the location map of the building in Kandla port precinct along with the tower building. A thorough geotechnical study of the site has been carried out. The foundation system is analyzed considering the soil-pile interaction, effect of foundation mat and the nonlinear behaviour of the soil. The analysis demonstrated the importance of the foundation mat in the pile supported buildings in seismically liquefiable area.

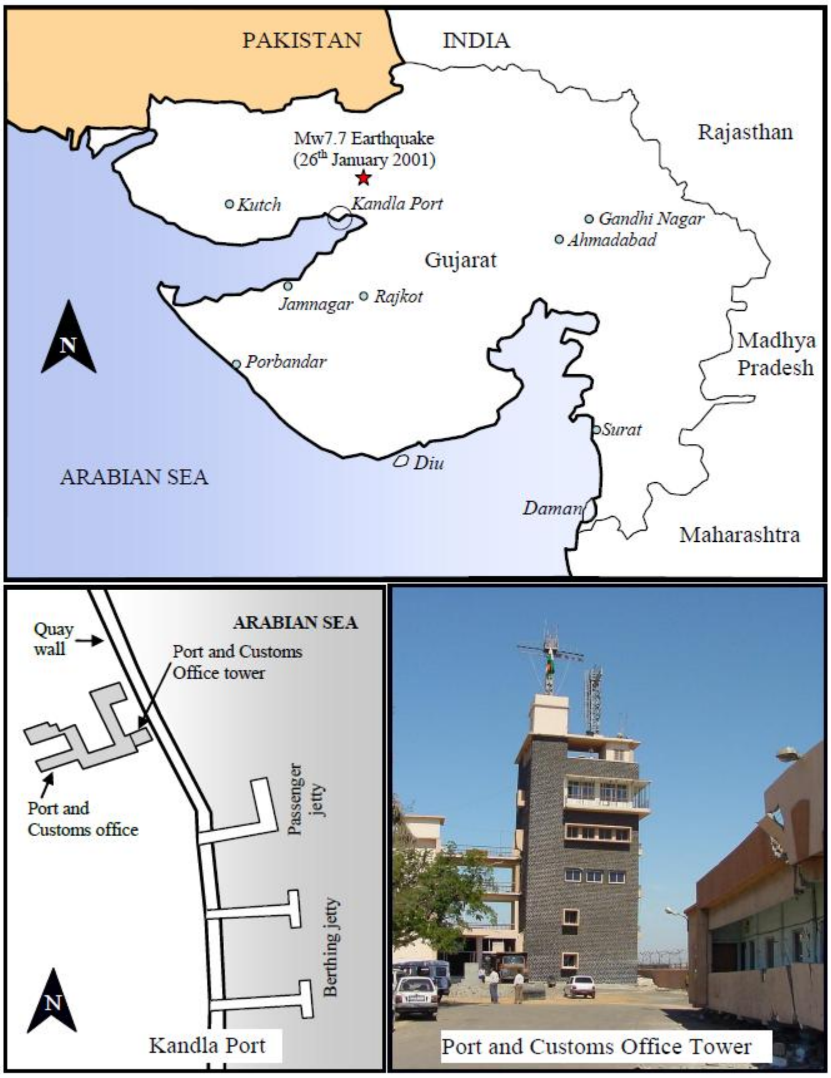


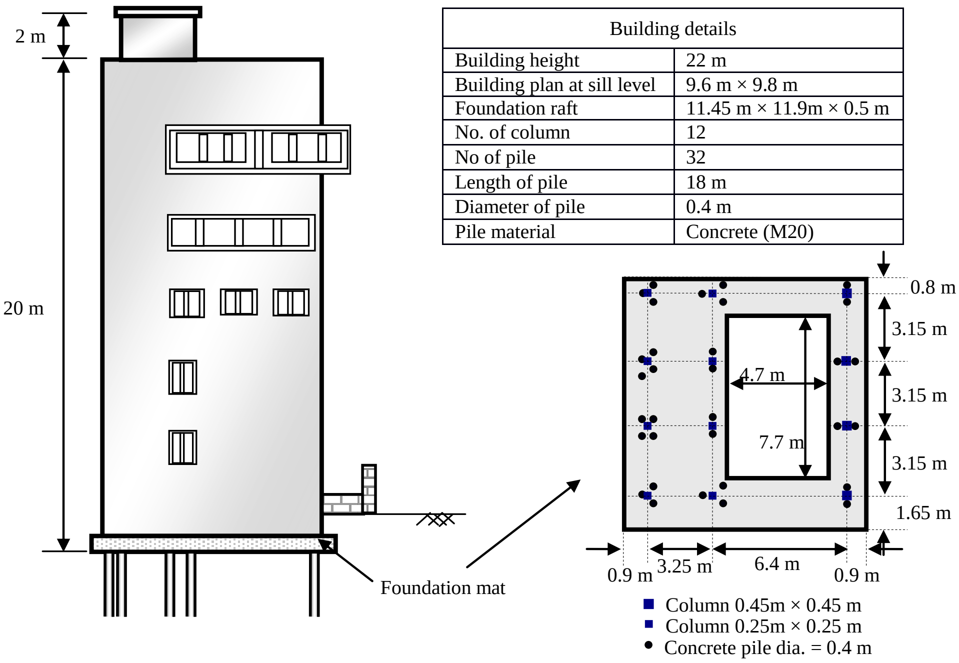
Figure 1: Location map of Kandla Port and Customs Tower

2. BUILDING DETAILS

The building was founded on 32 short cast-in-place concrete piles. Each pile was 18m long and 0.4m in diameter. The piles were passing through 10m of clayey crust and then terminated in a sandy soil layer. The loads from superstructure are transferred to the piles through the foundation mat, which eventually acts like a pile cap. Major structural details of the building are shown in Figure 2. The service load of the building is estimated to be 10749kN. The details of the calculations can be found in Dash et al (2008). Assuming equal sharing of the vertical loading, the static axial load per pile is about 336kN.

3. SITE DESCRIPTION

The Port of Kandla is built on natural ground comprising of recent unconsolidated deposits of interbedded clays, silts and sands. The water table is about 1.2 - 3.0m below the ground level. Figure 3 shows the borehole profile (taken from EERI, 2002) of the natural ground near to the building. Post earthquake observations indicated that the ground in the vicinity of the tower settled by about 30cm (one foot), resulting in the settlement of the floating mat floor of the building. There were evidences of extensive liquefaction with ejection of sand through ground cracks in the vicinity of the building. Post earthquake observations also reveal lateral spreading in many places in the Kandla port area.



Building details	
Building height	22 m
Building plan at sill level	9.6 m × 9.8 m
Foundation raft	11.45 m × 11.9m × 0.5 m
No. of column	12
No of pile	32
Length of pile	18 m
Diameter of pile	0.4 m
Pile material	Concrete (M20)

Figure 2: Major details of the Kandla Port and Customs Office tower

4.SEISMIC RESPONSE OF SOIL AT BUILDING SITE

As the soil profile comprises of unconsolidated deposits of interbedded clays, silts and sands, it is quite evident that the clayey soils will exhibit stiffness degradation and sandy soils will undergo liquefaction during strong earthquakes. The seismic response of the soil at the building site has been analyzed for the following three conditions.

4.1 Liquefaction potential of sandy soil and cyclic failure potential of clayey soil

For the soil at building site, the potential for liquefaction of sandy soils has been evaluated based on the method recommended by Idriss and Boulanger (2004). Further, cyclic failure in clays has been evaluated based on the new procedure proposed by Boulanger and Idriss (2005). The results of these analyses give factor of safety (F.O.S) against liquefaction potential and cyclic failures with depth as presented in Figure 3. It is evident from the figure that most part of the clay layer except the top 2m undergoes cyclic failure that may result in ground deformation and cracking. Furthermore, the entire sandy stratum between 10m to 22m is likely to have experienced liquefaction which may result in ground settlement and flow failure.

4.2 Post liquefaction settlement and lateral spreading of ground

The amount of seismic settlement of the soil deposit after liquefaction at the building site is calculated using two different methods; a) Method-1as suggested by Tokimatsu and Seed (1987), and b) Method-2 as suggested by Ishihara and Yoshimine (1992). The analyses predict a post-liquefaction settlement of 0.31m and 0.37m (Dash et al. 2008) respectively, which matches reasonably with the observed ground settlement of 0.3m. The amount of lateral spreading of the ground at the building site is estimated using the simplified semi-empirical probabilistic method proposed by Bray and Travasarou (2007). The expected amount of lateral spreading is estimated for the ground slope of 1˚ and 5˚ to be about 0.2m – 0.9m (Dash et al. 2008) that fairly matches with the post earthquake observations at the site. Hence, this gives confidence on the soil profile that has been chosen for the study.

4.4 Strength degradation of soil at site during earthquake

It is well understood that while subjected to strong seismic shaking, saturated clayey soil loses its strength due to cyclic mobility and saturated sandy soil loses its strength due to the increase in pore water pressure. For clayey soil, the strength degradation factor (β) is calculated by using the computer program SHAKE. The PGA at site is considered to be 0.33g. The degraded strength for liquefied sandy soil is taken as 10% of the strength of non-liquefied sand as suggested by AIJ (2001) code. The strength degradation factors for the soil at the site for different depths are presented in Figure 3.

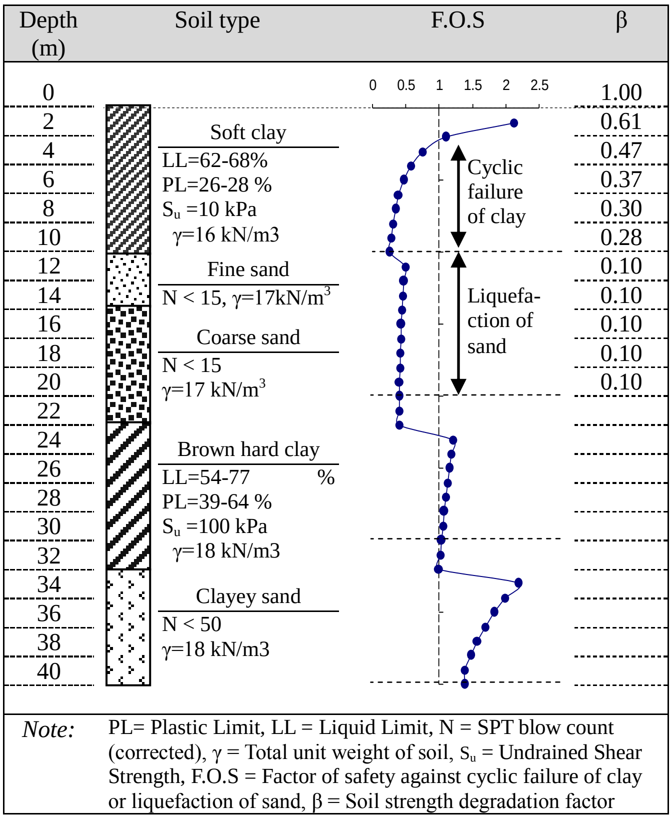


Figure 3: Soil profile at site, liquefaction potential and cyclic mobility analysis results

5. SEISMIC RESPONSE OF BUILDING DURING EARTHQUAKE

Based on the surface measurement, the pre and post earthquake configuration of the building is schematically drawn in Figure 4. From the soil-pile configuration, with an assumption that there was no structural failure of the piles, it can be inferred that the pile tip could have settled about 45cm (30cm ground settlement + 15cm building settlement) from its original position. As there were very little damage to the superstructure and all the damages were in the foundation, a numerical model has been developed using SAP (CSI 2005) to study the seismic response of the foundation during earthquakes. The modeling details are described below.

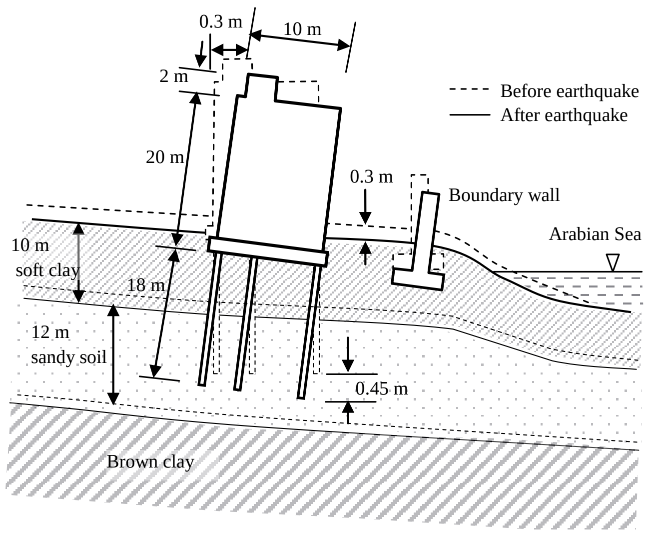


Figure 4 Schematic showing the tilting of the building assuming no structural failure in it.

5.1 Numerical model

5.1.1. Modelling of the building foundation

The cast in place concrete piles of the building are modelled as frame elements with specified axial and lateral stiffness. The foundation mat is modelled as 0.5m thick concrete shell element with a cut-out of 4.7m × 7.7m at one side (see Figure 2). In the present analysis, the foundation mat and the piles are treated as linear elastic concrete elements, as the nonlinear behaviour of the material is not expected. The superstructure load is estimated considering the effective tributary area for each column. However, the nonlinearity of the soil is incorporated in the analysis, which will be described in the following sections.

5.1.2 Modelling of soil

The soil interacting with the foundation of the building is modelled by four types of Winkler springs such as: a) axial (t-z springs), b) lateral (p-y springs), c) end bearing (q-z springs), and d) shallow foundation bearing (Q-u springs) springs (see Figure 5). The load-deflection curves for the soil springs at pile-soil interaction, i.e. first three types of springs, are estimated based on API guidelines for service condition (non-seismic). However, for seismic condition, the maximum strength of the soil springs for service condition is reduced by multiplying the strength degradation factor, β, (Figure 3). For shallow foundation bearing springs, the initial stiffness and ultimate load bearing capacity are calculated using the following two equations.

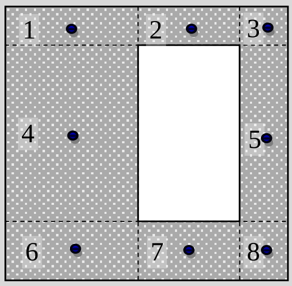
$$\text{Initial stiffness} = k = \frac{Gb}{(1-\mu)}\left[3.1\left(\frac{a}{b}\right)^{0.75} + 1.6\right] \quad \text{and}$$

$$\text{ultimate load carrying capacity} = Q = S_u N_c \left[1 + 0.3\left(\frac{a}{b}\right)\right],$$

where, a, b are the width and length of the raft, G is the Shear Modulus of soil, μ is the Poisson's ratio, S_u is the undrained shear strength of the soil, and N_c is the bearing capacity factor for two layered soil (NAVFAC 1982). The degraded soil strength factor (β) at 0.5m depth from the ground is about 0.8 (Figure 3). Hence, the strength of soil under the mat in seismic condition is taken as 80% that of service condition. The strength of soil at various depths before and during the earthquake (i.e., service and seismic conditions) is presented in Table 1.

Table 1: Properties of soil springs for pile-soil and mat-soil interaction

	Soil-Pile interaction						Soil-Foundation mat interaction		
Depth	[a]Maximum axial soil resistance (kN)		[m]aximum lateral soil resistance (kN)		[m]aximum end bearing capacity of soil (kN)		Mat no.	[d]Maximum load bearing capacity (kN)	
	Service	Seismic	Service	Seismic	Service	Seismic		Service	Seismic
0	0	0	0	0	-	-	1	881.23	704.98
1	2.6	1.98	21.6	16.42	-	-	2	842.53	674.02
2	6.4	3.90	33.8	20.62	-	-	3	231.66	185.32
3	8.3	4.48	46.1	24.89	-	-	4	2471.31	1977.05
4	9.81	4.61	58.3	27.40	-	-	5	815.76	652.61
5	11.1	4.66	73.4	30.83	-	-	6	1042.95	834.36
6	12.3	4.55	91.8	33.97	-	-	7	999.28	799.42
7	13.4	4.42	110.2	36.37	-	-	8	291.06	232.85
8	14.4	4.32	128.5	38.55	-	-			
9	15.3	4.44	146.8	42.57	-	-			
10	16.2	4.54	165.2	46.26	-	-			
11	30.9	3.09	593.1	59.31	-	-			
12	47.2	4.72	614.4	61.44	-	-			
13	51.3	5.13	651.7	65.17	-	-			
14	55.4	5.54	660.7	66.07	-	-			
15	59.5	5.95	690.5	69.05	-	-			
16	63.6	6.36	717.2	71.72	-	-			
17	67.7	6.77	740.6	74.06	-	-			
18	71.8	7.18	761.2	76.12	-	-			
18	-	-	-	-	633.34	63.33			



Foundation mat

5.2 Conventional Settlement and lateral spreading analysis

Settlement analysis of piles during earthquakes involves three major components, such as: a) axial compression of pile, b) slip between pile-soil interface, and c) settlement of the soil mass as a whole. The axial compression of pile is generally very less as compared to other two components in seismic condition. For the present case, the maximum frictional force that the soil can carry during earthquake through soil-pile skin friction per pile is 86.64kN (sum of the forces in column 3 of Table 1), and the maximum end bearing force is 63.33kN. The axial load on each pile is 336kN which is more than the total soil resistance from skin friction and end bearing (152.97kN). Hence, this analysis suggests that the pile will lose its equilibrium during the earthquake and will punch into the soil. The pile response is also checked for expected lateral spreading force. The piles are embedded in a non-liquefiable clayey crust and terminated in a liquefying soil layer (Figure 4). Hence, it is a case of hanging pile. During lateral spreading, the top non-liquefied clayey crust will move laterally carrying the building along with it as a rigid mass. Hence this analysis only gives the bending moment in the lower 8.5m pile due to differential soil-pile movement.

The above two analyses, however, does not predict the actual behaviour (i.e., tilting of the building) that has been observed in the field after the earthquake. Hence a detailed numerical analysis is carried out to study the combined effect of mat-pile soil interaction subjected to axial and lateral load together.

5.3 Detailed analysis including mat-pile-soil interaction

Figure 5 details the mat-pile-soil interaction model of the foundation system. The column loads are applied at the foundation mat level at their locations. The analysis is carried out by applying the superstructure load as well as the lateral spreading simultaneously to investigate the combined response of settlement and lateral spreading. The superstructure is assumed to be rigid and the dynamic effects have been ignored. The bottom 8.5m of the piles were ending in the liquefied zone having very less or no confinement during liquefaction and hence behaves like hanging piles. As discussed in section 5.2, these piles are subjected to negative relative ground displacement in the bottom 8.5m zone. In the present study, a pushover type of analysis is carried out applying the relative ground movement to the piles along with the axial load coming from superstructure. The analysis is carried out for both service condition (i.e., response before earthquake) and seismic condition (i.e., response during earthquake), and the results are discussed in the following section.

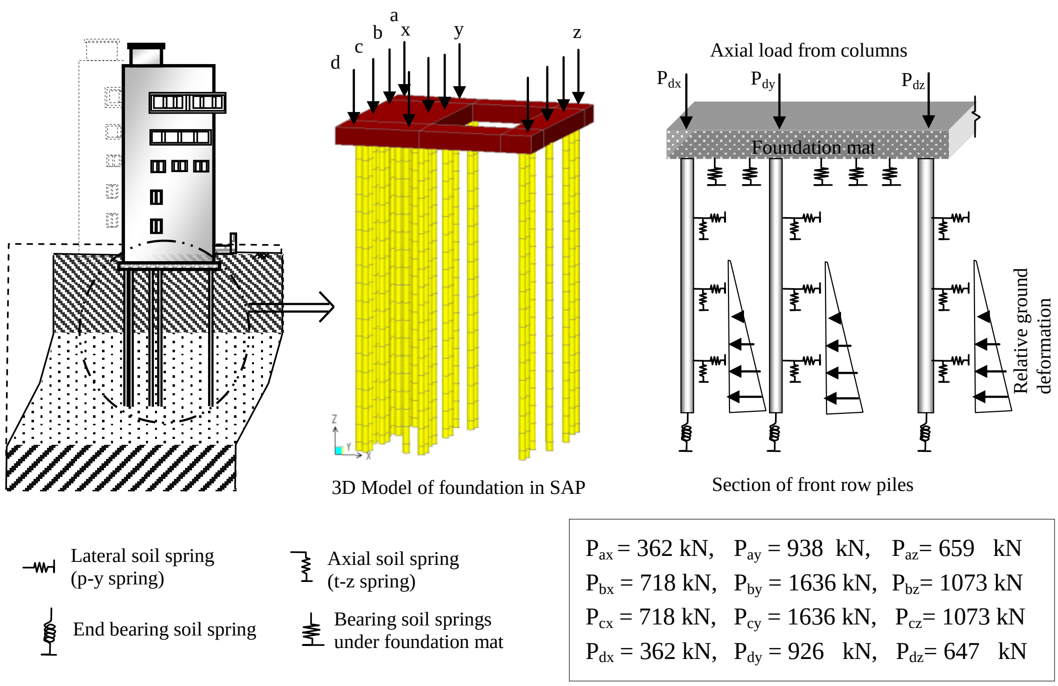
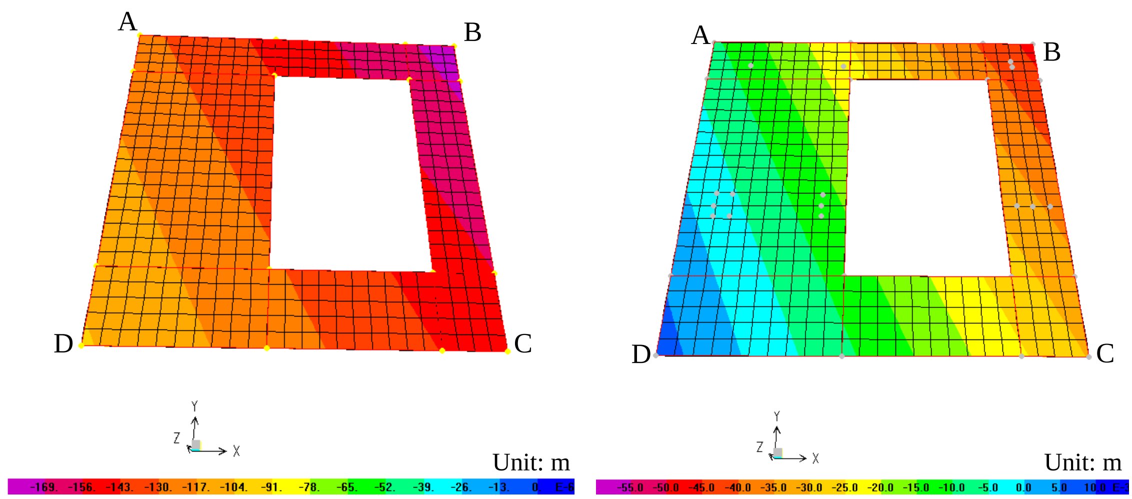


Figure 5: Detailed model description of the foundation system including mat-pile-soil interaction

6. RESULTS AND DISCUSSION

The analysis for service condition of the building shows that the maximum settlement of the building at service condition is about 0.2cm, which is within the permissible limits as suggested in most of the codes of practice. Figure 6(a) shows the deflection profile of the foundation mat for the service load. The analysis shows that the distribution of mass and stiffness of the structural system is such that the building tilts towards the sea side with more inclination towards the north end.

Figure 6(b) shows the pattern of foundation mat deflection obtained from the analysis for seismic condition. The figure shows the deflection contours of the foundation mat in the direction of gravity. The analysis shows that the foundation raft settled about 4.7cm at B. The mass distribution of the superstructure would hint that the building would tilt away from the creek (i.e. towards the eccentric building mass or towards the left in Figure 4) as the centre of mass (CM) is on the left of the geometric centre of the building. However, the cut-out in the foundation mat shifts the centre of resistance (CR) even left of the centre of mass causing the building to lean towards the Kandla creek i.e. towards right in Figure 4.



Service Condition				
	A	B	C	D
Vertical settlement (cm)	0.12	0.2	0.14	0.07

Seismic Condition				
	A	B	C	D
Vertical settlement (cm)	0.91	4.7	3.0	0.7

Figure 6: Deflection profile of foundation mat at: (a) service condition and (b) seismic condition

Axial load-deflection analysis for a single pile as described earlier has shown that under seismic condition the pile side friction and end bearing is not sufficient to prevent settlement of the building. However, this analysis is an improvement over the earlier analysis as it can be expected that the foundation raft will inevitably share a part of the superstructure load. Though the pile friction and end bearing was not enough to resist settlement, the foundation mat transferred a significant amount of superstructure load and was able to maintain equilibrium of the structure. This hence, prevented further sinking and/or tilting of the building. The detailed analysis also supports the post-earthquake field observation of the building that the north side of the building tilted more than the south side. The pattern of tilting of the building from the analysis matches fairly with that of the field observation. The total pile settlement predicted by the analytical study is 35cm (30cm is ground settlement + 5cm building settlement). In contrast, the observed settlement of pile tip is about 45cm which seems quite reasonable, if viewed in isolation.

7. CONCLUSION

The failure of Port and Customs Tower in Kandla provides a case study for the possible interaction between effects of lateral spreading, liquefaction induced settlement and foundation mat. In particular, the following conclusions can be drawn:

1. Piles passing through a deep non-liquefied crust and resting on liquefied soil can suffer excessive settlement and tilting rendering it unusable or expensive to rehabilitate following the earthquake. This should be avoided in practice.
2. Use of a large foundation mat or a large pile cap has a number of advantages such as: (a) reduction of the risk of sudden and/or catastrophic collapse as it is difficult for the large raft to punch through the soil even if the top soil is liquefied; (b) restraint to the settlement of the foundation.

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