Visakhapatnam Chapter

Proceedings of Indian Geotechnical Conference 2020
December 17-19, 2020, Andhra University, Visakhapatnam

A Case study of Liquefaction-induced Damage to a Port Building Supported on Pile Foundation

Monirul Mallick[1], Kalyan Kumar Mandal[2] and Ramendu Bikas Sahu[3]

[1]PhD Research Scholar, Department of Civil Engineering, Jadavpur University, Kolkata-700032
E-mail: monirulmallick64@gmail.com
[2]Associate Professor, Department of Civil Engineering, Jadavpur University, Kolkata-700032
E-mail: kkma_ju@yahoo.co.in
[3]Professor, Department of Civil Engineering, Jadavpur University, Kolkata-700032
E-mail: sramendu@gmail.com

Abstract. Pile foundations are often used to support structures in potentially liquefiable soil. However, the recurrence of failure or damages of structures resting on pile foundation in liquefiable areas during strong seismic event is still noticed. The failure of pile foundation in liquefiable soils poses a great concern to the geotechnical earthquake engineers. The damages of pile foundation in liquefiable soil may be due to structural failure, geotechnical failure or combination thereof. In order to study behaviour of pile foundation in liquefiable soil a well-documented case study on liquefaction-induced damages of Kandla port building (Gujarat) during 2001 Bhuj earthquake has been analysed and presented in this paper. The 22.0 m high six-storied RCC frame building supported on combined pile-raft foundation was tilted towards sea side after the earthquake. The soil at the site consisted of soft clay followed by fine to medium dense sand and hard clay. The liquefaction of intermediate sandy layer, ground settlement and lateral spreading were noticed in the consequences of this event. Nonlinear effective stress-based ground response analysis of the port site has been carried out. The foundation system is analysed by using beam on nonlinear Winkler foundation (BNWF) model to understand the probable failure mechanism of the port building. The open source finite element-based code, OpenSees is used to conduct all the analysis. The results of the present analysis are compared with the post-earthquake observations as well as the analyses reported in the literature. It is seen that the current results are matching well with the field observations at port building site.

Keywords: Port Structure, Nonlinear, BNWF, Opensees, Liquefaction, lateral spreading

1 Introduction

Pile foundations are extensively used to support the high-rise structures when the top soil is weak and results bearing capacity and settlement problems. Soil liquefaction is a great concern for design of pile foundation in seismically active areas. Indian standard seismic design code IS 1893-2002, Part-I [1] provides various guidelines mainly for seismic design of superstructures. However, this code does not provide detailed guidelines for seismic design of pile foundation in liquefiable soil. So, it is a challenging job for geotechnical earthquake engineers to ensure safe and economical design of pile foundation and pile-supported high-rise structures in liquefiable soil. A significant number of damages and/or collapses of pile foundations and pile-supported structures are observed in liquefiable soils during past major earthquakes. The lateral spreading was reported the main factor causing bending failure of piles [2-7]. However, pile foundations are also vulnerable due to bending-buckling interaction in liquefiable soil [8-10].

Port structures are more vulnerable to seismic damages when built in seismically active area like Gujarat in India [11]. The foundation of Port Structures is often constructed on reclaimed land which are potentially liquefiable. During the 2001 Bhuj earthquake the liquefaction of intermediate sandy layer, ground settlement, lateral spreading and resulted damages of Kandla port building have been reported [12]. The damages of pile foundation in liquefiable soil during seismic event may be due to structural failure, geotechnical failure or combination thereof depending up on the relative thickness and position of liquefiable soil layer. The non-liquefied crust overlying on liquefiable soil layer is mainly responsible for bending or shear failure of pile foundation. If top non-liquefiable crust is absent, the pile may loss the lateral support and prone to buckling failure under large axial load. However, if the top and bottom non-liquefiable crust is not present, pile may fail due to excessive settlement [12].

Liquefaction-induced damage to building supported on Pile foundation during earthquake is presented in this study using a reported case study on damages to the Kandla Port building during the 2001 Bhuj earthquake in India [12]. The effective stress-based ground response analysis (GRA) of the port site has been carried out using nonlinear finite element program Cyclic1D [13] considering nonlinearity of soil. The foundation system has been analyzed by using beam on nonlinear Winkler foundation (BNWF) model to study the lateral spreading of the port building site and the results obtained are found to be in line with the post-earthquake observations.

2 Kandla Port Building: Liquefaction-induced Damage during Bhuj Earthquake

2.1 The Earthquake

The 2001 Bhuj earthquake (Mw=7.7) was the most devastating seismic hazard causing tremendous damages of lives and properties in urban area of India. It struck the Kutch area of Gujarat state of India on January26, 2001. The epicentre of the earthquake was situated at 23.419ºN, 70.232ºE located at a distance of 20 km North East of Bhuj in Gujrat. The maximum bedrock level acceleration (MBRA) was 0.106g. The acceleration time-history of the earthquake is shown in Fig. 1(a).

2.2 The structural details of the Kandla Port building

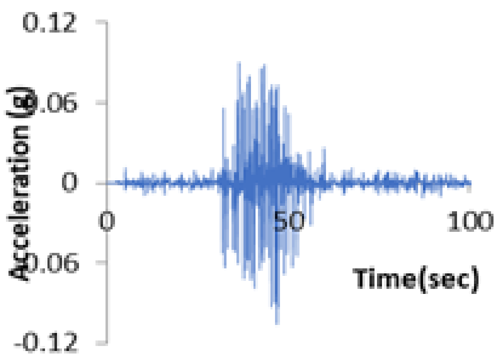
Kandla Port is located in the Kandla Creek and is 50 km from the epicentre of the earthquake. The Kandla Port tower, a 22.0 m high six-storied building supported on combined pile-raft foundation is located proximate to waterfront. The building was supported by 12 numbers of column (0.45x0.45 m and 0.25x0.25 m) and 32 numbers RCC piles of diameter 0.4 m and length 18 m. The 0.5 m thick foundation mat was provided as a rigid pile cap.

2.3 The Geotechnical properties of the port site

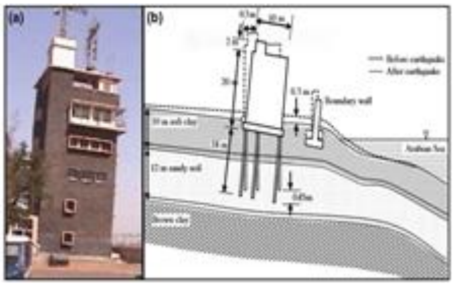
The Kandla port is built on inherent ground consisting of recent unconsolidated layer of clay, silt and sand. The ground slope is about 1.5-2.0 % towards seaside. The ground water table (GWT) is located at 1.2-3.0 m below ground surface. The soil of the site composes of 10 m deep soft clay underlain by 12 m deep fine to medium dense sand and 10 m deep hard clay [12]. The top clay layer having water content 42-47 % is highly plastic in nature. The SPT-N values of the upper fine sand layer is below 15, whereas the deep coarse sand layer is below 50. The fines content of sandy soils is in the range of 1-32%. The N-value below 15 and fines content 1-32% below GWT of intermediate sandy layer are prone to liquefaction under strong to moderate earthquake vibration.

2.4 Post-Earthquake observations

The top of the considered pile-supported building was tilted about 0.30 m towards sea side. The ground adjacent to the building was settled about 0.3 m. Ejaculation of sand through ground cracking was observed near the building site which indicates the widespread liquefaction. A successive pattern of lateral spreading was noticed after earthquake. The maximum magnitude of lateral spreading reported was 0.80 to 1.0 m [12]. Very little damage of superstructure was noticed and significant damages observed in the foundation. Fig. 1(b) shows the tilted building, schematic drawing of the building before and after earthquake respectively [12].



(a)



(b)

Fig. 1. a Acceleration time-history of Bhuj earthquake and **b** Tilting of Kandla Port Building along with building configuration before and after the 2001 Bhuj earthquake [12]

3 Nonlinear Ground Response Analysis of the Port Building Site

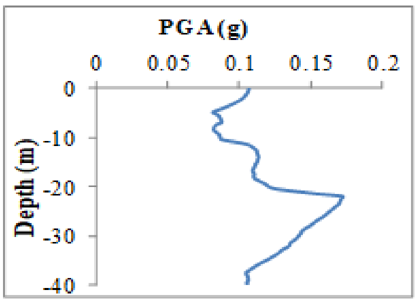
1D nonlinear GRA has been carried out for predicting the layered soil response of Kandla port building site using 2001 Bhuj earthquake input ground motion at bedrock level. The present study utilizes the nonlinear methods of analyses through computer program Cyclic1D [13]. Cyclic1D has been developed for 1D wave propagation analysis using pressure-dependent and pressure-independent soil constitutive models. The constitutive model of soil in Cyclic1D has the capability to narrate the development and dissipation of pore water pressure. The liquefaction model within Cyclic1D is built under multi-yield-surface plasticity framework. The soil models available in Cyclic1D have also been implemented in OpenSees [14], a software framework for developing applications to simulate the performance of structural and geotechnical systems subjected to earthquakes. Incremental plasticity model is used to simulate the nonlinearity of soil. The finite elements are assigned for saturated soil strata under formulation of fully-coupled fluid-soil system. In time domain based nonlinear analysis the dynamic equation of motion is solved at every time step with the help of Newmark time integration method by specifying two user defined coefficients Beta (β) and Gama (γ).
A finite element model in the present study is defined in Cyclic1D by specifying the total height of soil profile of 40m.The multi-layered soil profile layer has been divided into total 80 numbers of elements, each of 0.50m thick after convergence analysis. Predefined material models of clayey and sandy soils are chosen to define the soil profile of the site as shown in Table 1. A rigid bedrock base is considered for soil layer. Location of GWT is assumed at 1.50 m below ground surface. Mass and stiffness proportional Rayleigh-type damping (5%) has been considered. In current study, average acceleration method (γ=0.50, and β=0.25) has been used in Cyclic1D.

Table 1. Parameter values for the soil materials models of Kandla Port soil [13]

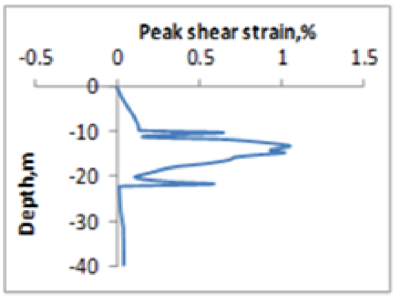
Depth (m)	SPT-N value	Material model	Unit weight (kN/m$^{3)}$	V_s (m/sec)	Poisson's ratio	Co-eff. of permeability (m/s)	Φ (degree)	C_u (kN/m^2)
0.0-10.0	5	Cohesive soft	13.00	100.	0.4	1.0E-09	-	18
10.0-22.0	14	Medium, sand permeability	19.00	205.	0.4	6.6E-05	31.5	-
22.0-32.0	35	Cohesive stiff	18.00	300.	0.4	1.0E-09	-	75
32.0-40.0	50	Medium-dense, sand permeability	20.00	225.	0.4	6.6E-05	35	-

3.1 Assessment of liquefaction potential

Figs. 2(a) and (b) show the profile of PGA and peak shear strain. The PGA at surface level for Bhuj earthquake motion is 0.107g against MBRA 0.106g**.** It is seen that surface acceleration is almost same with respect to MBRA for the Kandla port site. The soil strata at 10.0 m to 22.0 m depth consisting of fine to medium dense sand undergoes large strain. The peak strain value obtained is 1.05% at 13.25m depth from surface. Amplification of PGA reduces significantly at that strata due to higher shear strain. Higher strain value indicates the liquefaction susceptibility of the site. The profile of excess pore pressure (EPP) ratio is evaluated at each depth of Kandla Port site as shown in Fig. 3(a). The EPP ratio is almost 1 for soil layer of 10.0 to 22.0 m depth under the Bhuj earthquake. So, the intermediate fine to coarse sandy strata is prone to liquefaction.



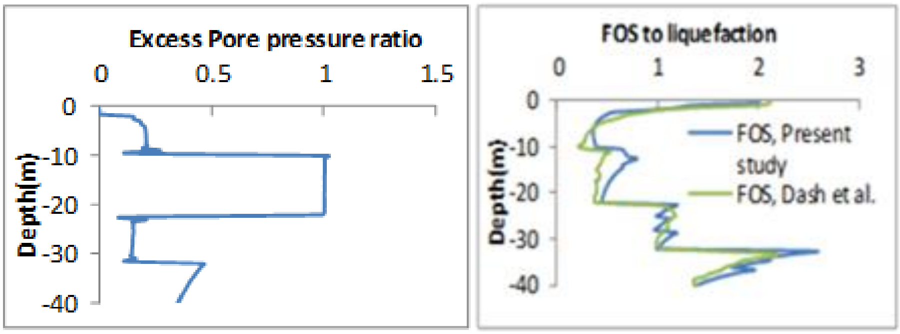
(a)



(b)

Fig. 2 Profile of **a** PGA and **b** peak shear strain

Simplified deterministic method [15-17] is used to assess the liquefaction susceptibility of the port site as shown in Fig 3(b). The factor of safety to liquefaction (FOS) which is ratio of cyclic resistance ratio (CRR) to cyclic stress ratio (CSR) is evaluated along depth. CRR is evaluated from corrected SPT-N value and fines content (FC) percentage. The PGA at surface and shear stress reduction co-efficient profile (R_d) obtained from nonlinear GRA has been utilized for evaluating the CRR. The FOS evaluated from the results obtained using Cycic1D software is compared with that obtained using SHAKE 2000 computer program [12], which uses equivalent linear analysis. Fig. 3(b) shows the comparison of FOS along depth using both the codes. It is found that the results of Cyclic1D are co-relates well with the results of SHAKE 2000. The slight deviation in results at intermediate depth may be reasonable due to different analysis procedures and soil model which are estimated based on soil descriptions by two authors.
It is clear from graph that the FOS is less than 1 for top clay (1.5-10.0 m) and intermediate sandy layer (10.0-22.0 m) under Bhuj motion. So, the upper clay stratum experiences ground deformation and cracking due to cyclic failure. Moreover, intermediate sand layer (10.0-22.0 m) suffers ground settlement and lateral spreading due to liquefaction. So, the results of GRA argues the ejaculation of liquefiable fine sand through ground cracking near the building site.

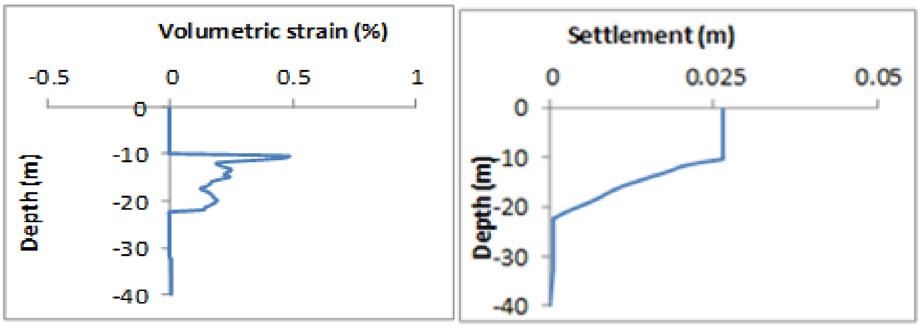


(a) (b)

Fig. 3. Profile of **a** Excess pore pressure ratio and **b** Factor of Safety for liquefaction potential (FOS)

3.2 Evaluation of post-liquefaction settlement

Post liquefaction settlement of saturated sand depends on a number of factors such as relative density, maximum volumetric strain and excess pore pressure. Figs. 4(a) and (b) show the profile of volumetric strain and settlement of the port site under Bhuj earthquake motion. The peak value of volumetric strain obtained is 0.49% at 10.25 m depth. The post-liquefaction settlement profile of port site is evaluated using Cyclic1D. The total post-liquefaction settlement calculated using present model, previous study [12] and field observation are illustrated in Table 2. The settlement values obtained from the present analysis (0.288 m) are matching well with the previous author as well as post-earthquake observed settlement of 0.3 m. Hence, the present study justifies the liquefaction phenomenon of the port site at Kandla.



(a) (b)

Fig.4. Profile of **a** volumetric strain and **b** post-liquefaction settlement

Table 2. Post-liquefaction settlement

Depth(m)	Settlement (m)	Settlement (m) [12]		Observed settlement (m) [12]
		Method-I	Method-II	
10-22	0.283	0.241	0.345	
32-40	0.005	0.070	0.028	
Total settlement	0.288	0.311	0.373	0.300

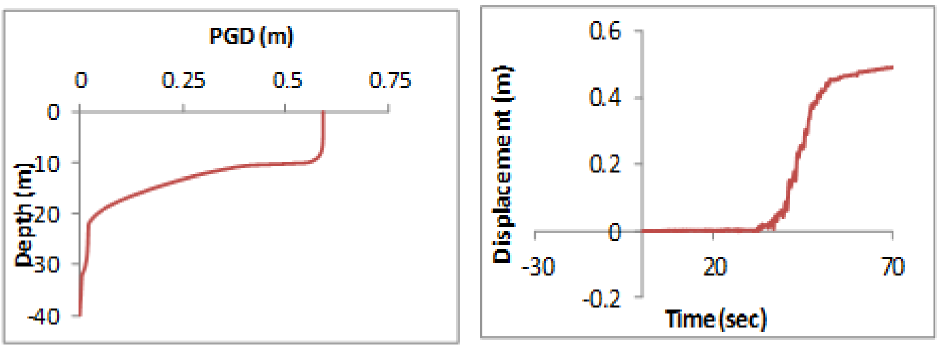
3.3 Lateral spreading of the site

Lateral spreading is generally defined as permanent lateral displacement of gently sloping ground due to earthquake-induced liquefaction. Various empirical and semi-empirical methods are available in the literature [18-21] for predicting the amount of lateral spreading. In this study, simplified semi-empirical relationship [20] is used to estimate the amount of lateral spreading for probability of exceedance of 16% and 84% respectively. The value of yield co-efficient of soil slope considered is 0.052 assuming 5% ground slope. Additionally, the following values have been considered for evaluation of lateral spreading: earthquake magnitude Mw=7.7, Average shear wave velocity=175 m/sec, Initial time period of ground Ts=0.23 sec, spectral acceleration at 1.5Ts= 0.44g. The amount of lateral spreading evaluated in the present study is presented in Table 3 along with the values calculated by Dash et al [12] and observed post-earthquake observation. The obtained value is comparable with the estimated value [12] and post-earthquake observed value.

Table 3. Liquefaction-induced lateral spreading

	Present study		Dash et al [12]		Observed value [12]
	16% probability of exceedance	84% probability of exceedance	16% probability of exceedance	84% probability of exceedance	
Lateral spreading (cm)	18.43	73.73	24	91	80-100

The peak ground displacement (PGD) is almost uniform for depth 10.0 m from ground surface as shown in Fig. 5(a). The PGD below depth 10.0 m decreasing linearly and becomes negligible at the bottom of liquefiable layer. Fig. 5(b) represents the displacement time-history of the site under Bhuj motion. It is noticed that soil shows residual displacement at the end of loading cycle indicating probability of earthquake-induced lateral spreading under strong motion. The PGD at surface of soil are 0.590 m.



(a) (b)

Fig.5. a) PGD profile and **b)** displacement time-history

4 Modelling and Analysis of Foundation System

The foundation of Kandla Port building consisted of 32 number 0.4m diameter and 18.0m long piles. The laterally loaded single pile is analysed using BNWF model subjected to a kinematic loading simulating lateral spreading as shown in Fig. 6. The open source finite element-based code, OpenSees [14] is used to conduct the analysis. The undermost 8.0 m of the pile was founded in liquefiable soil and top 10.0 m were embedded in non-liquefied soft clay layer as shown in Fig. 2(b). Pile and soil are simulated by displacement-based beam element and nonlinear spring element respectively. p-y curves based on API [22] procedures are used for simulating nonlinear behaviour of soil. The PGD profile as shown in Fig. 5(a) is employed to the soil end of the p-y springs to simulate lateral spreading [23]. The displacement profile is almost uniform for top 10.0 m soil layer. The same is varying linearly over liquefiable layer and is zero at the bottom layer.

The model is developed with three different sets of nodes: fixed spring nodes, slave spring nodes and pile nodes. The finite element mesh is generated using element length of 0.5 m. The three-dimensional spring nodes having three translational degrees of freedom are generated. Zero-length elements are used to define soil springs. The p-y springs oriented in lateral direction represent lateral resistance of soil-pile interface. The p-y springs are defined using the PySimple1 uniaxial materials. The input parameters for defining PySimple1 material are ultimate lateral resistance of soil (p_{ult}) and initial stiffness (k). p_{ult} values of clay and sand are calculated using procedure proposed by Matlock [24] and Brinch Hansen [25] respectively. Initial stiffness values (k) are computed as recommended by the API [22]. The modification of stiffness values for depth are done using procedure proposed by Boulanger et al [26].

The three-dimensional pile nodes are created with six degrees of freedom. Both translational and rotational degrees of freedom of pile nodes are considered. The base of the pile is considered as fixed. The pile nodes are connected with slave nodes of soil springs using equal degrees of freedom. Here the pile nodes are considered as master nodes. Both the nodes share equal degrees of freedom in lateral direction. The pile is modelled as displacement-based beam-column elements with elastic behaviour. The value of Young modulus and Poisson's ratio of pile used in the study are 30000 MPa and 0.3 respectively [12]. Pseudo-static analysis has been conducted applying incremental displacement.

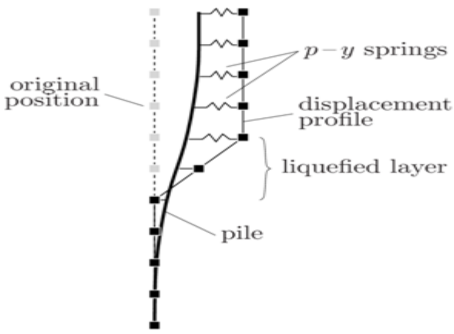
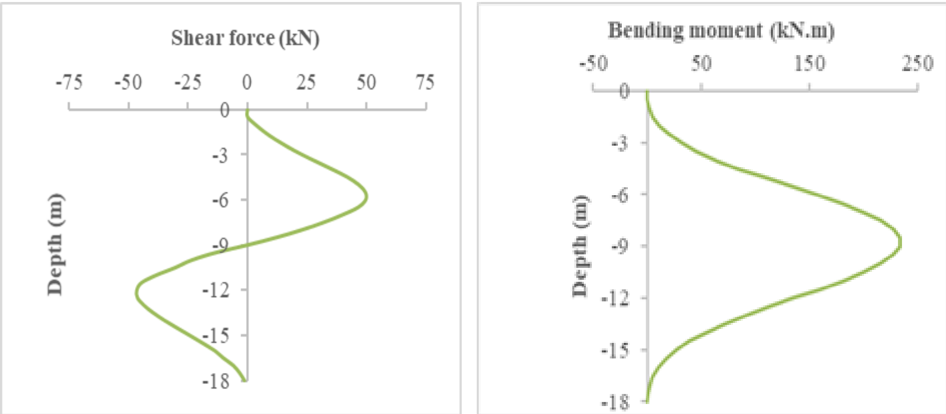


Fig. 6. BNWF Lateral spreading modelling approach adopted for soil-pile interaction [14]

The profile of shear force and bending moment obtained from the analysis are shown in Figs. 7(a) and (b) respectively. Table 4 summaries the results obtained from the present lateral spreading analysis of single pile. From Table 4, it is observed that the maximum bending moment (233.38 kN.m) exceeds the capacity of the pile in seismic condition. But shear force demand is less than shear capacity of pile. Hence, the pile is unsecured on account of kinematic bending failure. Formation of plastic hinge is anticipated at the interface between liquefiable and non-liquefiable crust as shown in Fig 7(b). However, no structural failure is expected. The pile supported Kandla Port building was tilted mainly due to liquefaction-induced ground failure.

Table 4. Bending moment and shear force of pile foundation

Bending moment (kN.m)		Shear force (kN)	
Demand (Present study)	Capacity [12]	Demand (Present study)	Capacity [12]
233.38	120-144	49.51	459.3-473.3



(a) **(b)**

Fig.7. **a**)shear force diagram and **b**)bending moment diagram of pile

5 Conclusions

In the present study, a case study of liquefaction-induced damage to Kandla Port Building (Gujarat) supported on pile foundation during 2001 Bhuj earthquake is presented. The 22.0 m high six-storied RCC frame building supported on combined pile-raft foundation was tilted towards sea side after this earthquake. The key conclusions from the present analysis are as follows:

1. The acceleration at the ground surface does not magnify significantly with respect to MBRA for the Kandla port site under Bhuj earthquake.
2. 12.0 m thick intermediate loose to medium dense saturated sand below soft clay strata is potentially liquefiable under 2001 Bhuj earthquake motion.

3. The post-earthquake observed ground settlement of 0.3 m is matching well with the evaluated post-liquefaction settlement of 0.288 m obtained from present study.
4. The field-observed lateral spreading of 0.8 to 1.0 m is also consistent with the present results of lateral spreading analysis.
5. There is no possibility of structural failure of the foundation of the Kandla port building under 2001 Bhuj earthquake. The foundation was failed mainly due to geotechnical failure (i.e., excessive settlement).
6. The pile foundation travel through non-liquefiable layer and terminated in liquefiable soil can experience undue settlement. This exercise can be avoided as far as possible in practical situation.

References

1. IS 1893-Part 1: Criteria for earthquake resistant design of structure. Bureau of Indian Standards, New Delhi, India (2002).
2. Ishihara, K.: Geotechnical aspects of the 1995 Kobe earthquake: Terzaghi Oration. In: Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, Germany (1997).
3. Tokimatsu, K., Oh-oka, H., Satake, K., Shamoto, Y. and Asaka, Y.: Effects of lateral ground movements on failure patterns of piles in the 1995 Hyogoken-Nambu earthquake. In: Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics III, pp. 1175-1181.ASCE geotechnical special publication, Reston (1998).
4. Berrill, J.B., Christensen, S.A., Keenan, R.P., Okada, W. and Pettinga, J.R.: Case studies of lateral spreading forces on a piled foundation. Geotechnique 51(6), 501-517 (2001).
5. Abdoun, T.H. and Dobry, R.: Evaluation of pile foundation response to lateral spreading. Soil dynamics and Earthquake Engineering 22(9-12), 1051-1058 (2002).
6. Finn, W.D.L. and Fujita, N.: Piles in liquefiable soils: Seismic analysis and design issues. Soil dynamics and Earthquake Engineering 22(9-12), 731-742 (2002).
7. Abdoun, T., Dobry, R., O'Rouke, T.D. and Goh, S.H.: Pile foundation response to lateral spreads: Centrifuge modelling. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 129(10), 869-878 (2003).
8. Bhattacharya, S.: Pile instability during earthquake liquefaction. Ph.D. Thesis, University of Cambridge, U.K (2003).
9. Dash, S.R., Bhattacharya, S. and Blakeborough, A.: Bending-buckling interaction as a failure mechanism of piles in liquefiable soils, Soil Dynamics and Earthquake Engineering 30(1-2), 32-39 (2010).
10. Knappett, J.A. and Madabhushi, S.P.G.: Effects of axial load and slope arrangement on pile group response in laterally spreading soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 138, 799-809 (2012).
11. Rajaram, C. and Kumar, R.P.: Vulnerability Assessment of Coastal Structure: A Study on Port Buildings, International Journal of Education and Applied Research, Vol. 4, Issue SPL-2 (2014).

12. Dash, S. R., Govindaraju, L. and Bhattacharya, S.: A case study of damages of the Kandla Port and Customs Office tower supported on a mat–pile foundation in liquefied soils under the 2001 Bhuj earthquake. Soil dynamics and Earthquake Engineering 29(2), 333–346 (2009).
13. Elgamal, A., Yang, Z. and Lu, J. Cyclic1D: A Computer Program for Seismic Ground Response. Report No. SSRP-06/05, Department of Structural Engineering, University of California, San Diego, La Jolla, CA (2006).
14. Mazzoni, S., McKenna, F., Scott,M.H., Fenves,G.L. et al.: OpenSees Command Language Manual, PEER, University of California, Berkeley (2006).
15. Seed, B. and Idriss, I.M.: Simplified procedure for evaluating soil liquefaction potential. Journal of Geotechnical Engg, Division, ASCE 97(9),1249-1273 (1971).
16. Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M.: The influence of SPT procedures in soil liquefaction resistance evaluations. J Geotech Eng 111(12):1425–1445 (1985).
17. Boulanger, R.W. and Idriss, I.M.: CPT and SPT based liquefaction triggering procedures. Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, University of California, Davis (2014).
18. Kramer, S.L. and Smith,M.W.: Modified Newmark model for seismic displacements of complaint slopes. Journal of Geotechnical and Geoenvironmental Engineering 123(7), 635-644 (1997).
19. Bray, J.D., Rathje, E.M., Augello,A.J. and Merry, S.M.: Simplified seismic design procedures for geosynthetic-lined, solid waste landfills. Geosynth Int 5(1-2), 203-235 (1998).
20. Bray, J.D. and Travasarou, T.: Simplified procedure for estimating earthquake-induced deviatoric slope displacements. Journal of Geotechnical and Geoenvironmental Engineering 133(4), 381-392 (2007).
21. Escudero, J.L.M.: Simplified Procedures for Estimating Earthquake-Induced Displacements. Ph.D. Thesis, University of California, Berkeley (2017).
22. American Petroleum Institute (API): *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms.* API Recommended Practice 2A(RP–2A), Washington, D.C., 17th edition (1987).
23. Brandenberg, S.J., Boulanger, R.W., Kutter, B.L., and Chang, D.: Static pushover analyses of pile groups in liquefied and laterally spreading ground in centrifuge tests. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 133(9), 1055–1066 (2007).
24. Matlock, H.: Correlations for design of laterally loaded piles in soft clay. In: Proceedings of the II Annual Offshore Technology Conference, pp. 577-594. Houston, Texas (1970).
25. Brinch Hansen, J.: The ultimate resistance of rigid piles against transversal forces. *Bulletin No. 12*, Geoteknisk Institute, Copenhagen, 5–9 (1961).
26. Boulanger, R.W., Kutter, B.L., Brandenberg, S.J., Singh, P., and Chang, D.: *Pile foundations in liquefied and laterally spreading ground during earthquakes: Centrifuge experiments and analyses*. Center for Geotechnical Modeling, University of California at Davis, Davis, CA. Rep. UCD/CGM-03/01 (2003).