Proceedings of the 7th World Congress on Civil, Structural, and Environmental Engineering (CSEE'22)
~~Lisbon, Portugal~~ Virtual Conference– April 10 – 12, 2022
Paper No. ICGRE 205
DOI: 10.11159/icgre22.205

Some Observations on Numerical Analysis of Lateral Response of Caisson Foundation

Sanshrit Singhai[1], Ramyasri Rachamadugu[2], Gyan Vikash[3]
[1] ss497@snu.edu.in
[2]rr783@snu.edu.in
[3]gyan.vikash@snu.edu.in
Shiv Nadar University, Dadri, UP 201314, India

***Abstract* -** The lateral response of a caisson is obtained by treating a three-dimensional (3D) caisson-soil system as a two-dimensional (2D) plane strain problem using the design procedures cited in codal provisions. Although numerical analysis with a 2D model requires less computational time than numerical analysis with a 3D model, the accuracy of the results obtained with 2D modeling must be checked. Therefore, the present study aims to verify the accuracy of modeling the 3D caisson problem as a 2D model. Moreover, the caisson-soil system is highly nonlinear with material and geometric nonlinearity. There exists no clear understanding of the role of material and geometric nonlinearities in the deformation mechanism of a laterally loaded caisson. Hence, this study develops a 2D and 3D finite element model to investigate the significance of both material and geometric nonlinearities in the lateral response of the caisson. This study models the material nonlinearity of soil using a multi yield surface plasticity model and assumes caisson as a linear elastic body. Geometric nonlinearity is modeled using zero thickness contact elements with elastic perfectly plastic constitutive models at the interface. The results of this study indicate that caissons with L/B ratios ranging from 1 to 3 should be modeled as a 3D model only for accurate results. This study demonstrates the predominant contribution of geometric nonlinearity over material nonlinearity during the lateral displacement of the caisson. This study highlights the importance of accurately modeling geometric nonlinearity due to interface and emphasizes the need to understand the interface mechanism in detail to effectively predict the response of the caisson-soil system.

***Keywords*:** Lateral response, Caisson-soil system, Interface nonlinearity, and Material Nonlinearity

1. Introduction

Caissons are massive deep foundations used for major highway and railway bridges [1]. Due to their massive structures, caissons are generally considered safe against lateral loading conditions. Caissons are preferred when stiff soil/bedrock is unavailable even for the large depths and often subject to heavy scouring and high magnitude of lateral loads [2]. Despite being the massive foundation, failures of structures rested on caisson have been reported in the literature [3-4]. These observations highlight the need to accurately estimate the lateral capacity of the caisson and understand the deformation mechanism that transpires during the lateral displacement of the caisson. The design procedures stated in codal provisions (IRC: 45-1972, IRC: 78-1983 and Japan road association codal provision) to predict the lateral capacity of caisson assumes a three-dimensional (3D) caisson-soil system as a two-dimensional (2D) plane strain problem [5-7]. Although numerical analysis with a 2D model requires less computational time than numerical analysis with a 3D model, it is critical to ensure that the reduction in computational time is not at the cost of compromising the accuracy of the results. Therefore, there exists a need to investigate the accuracy of treating the 3D problem as a 2D plane strain problem. There is no clear information available in the literature regarding the length (L) to width (B) ratios of caissons for which the 3D model of the caisson can be treated as a 2D model. Moreover, the caisson-soil system is highly nonlinear with material and geometric nonlinearity. Material nonlinearity is caused by the complex behavior of soil and materials of the caisson, while geometric nonlinearity is due to variations in the response of soil and caisson at the interface while transferring the load from structure to soil or vice versa [8]. Most of the numerical studies cited in the literature performed numerical studies assuming perfectly bonding conditions (i.e., the response of soil and structure is the same) at the interface [9-11]. The accuracy of predicting the lateral response of caisson by approximating geometric nonlinearity at interface needs to be explored. The numerical studies cited in the literature stated that the significance of geometric nonlinearity due to the interface is negligible for low magnitudes of seismic loading conditions [12]. However, there exists a need to observe the significance of geometric nonlinearity under high magnitudes of lateral loads. Moreover, none of the existing studies emphasized the role of material and geometric nonlinearities in the deformation mechanism of a laterally loaded caisson. Hence, the present study aims to examine the significance of both material and geometric nonlinearities during the lateral behavior of the caisson and to determine the optimum value of the *L/B* ratio of the caisson, for which the 3D model of the caisson-soil system can be modeled as a 2D model. For this purpose, this study develops finite element models of 3D and 2D caisson embedded in nonlinear soil. The caisson is modeled as a linear elastic body for 3D and 2D models considered in this study. A multi yield surface plasticity model is used to simulate the nonlinearity of the soil domain. This study assumes that the material nonlinearity is primarily caused by soil. As a result, in this case, material nonlinearity can be referred to as soil nonlinearity. Whereas geometric nonlinearity is caused mainly due to the interaction of the caisson and the soil at the interface. Hence this study refers to geometric nonlinearity as interface nonlinearity, and it is modeled using a zero-thickness contact element. This study compares the results of a numerical model that takes into account both soil and interface nonlinearities to the results of numerical studies that only address soil nonlinearity while ignoring interface nonlinearity. This is done to investigate the significance of interface nonlinearity during the lateral displacement of the caisson. Furthermore, the results of the numerical model with both soil and interface nonlinearities are compared to the results of the numerical model, which considers only interface nonlinearity and neglects the soil nonlinearity. This is done in order to observe the role of soil nonlinearity in the lateral response of the caisson. The numerical analysis performed in this study considers different *L*/*B* ratios of the 3D model of the caisson to determine the optimum *L/B* ratio for which the 3D model can be treated as a 2D model. The influence of soil type (medium clay and medium sand) on the lateral response of caissons is also investigated in this study.

2. Problem Statement

The dimensions of the existing caisson mentioned in the literature varied with width ranging from 5m to 13m, depth 17m to 65m, and scour depth observed to be varying from 5m to 36m [13]. This study considers a rectangular caisson of M35 grade concrete with a width of 6m, depth to be 36m, and magnitude of the scour depth (H) equal to 12m. The embedment depth of the caisson (D) is 18m. The value of D considered in this study satisfies the stability condition stated in IRC:78-2000 [14]. According to IRC:78-2000, the value of D must be greater than half of the scour depth. The lateral load(P) was applied at the well cap. The length (L) of the caisson is varied in terms of width (i.e., B,5B**,** and 7B). This study considers the embedment of the caisson in medium clay and medium sand.

3. Numerical Modeling of Caisson- Soil System

The numerical model of the caisson- soil system was developed using OpenSees version 2.5 solver and GiD version 14 software for pre and post-processing [15-16]. A detailed sensitivity analysis was performed to determine the optimum domain and element size. To perform sensitivity analysis, this study considered the caisson with L/B ratio equal to 1 subjected to 120MN. Sensitivity analysis was performed by taking into account various domain widths with the depth of the domain of 3B below the caisson. Different domain widths considered in the sensitivity analysis are 11B,21B,31B, and 41B. Sensitivity analysis was carried out with fixed boundary conditions at the bottom and rollers for the sides of the domain. This study assumes caisson as a linear elastic material. The young's modulus of the caisson was determined as stated in IS:456-2000[17]. According to IS:456-2000, the modulus of elasticity of the caisson is equal to the 5000 times square root of characteristic strength of M35 grade concrete. The Poisson ratio and density of the caisson in this study are 0.15 and 2500 kg/m3. The material property of medium sand was defined using pressure-dependent multi yield surface modeling and medium clay using pressure-independent multi yield surface modeling. The failure criteria for the medium clay were based on von Misses failure criteria, and Mohr–Columb failure criterion was considered for the medium sand. Table 1 consists of the values of material properties of nonlinear soils considered in this study [18]. Interface modeling was done using a zero thickness contact element with elastic perfectly plastic relationship to capture sliding and elastic perfectly gapped material for separation at the interface. The stiffness of the contact element to resist the sliding indicated as K_T. The zero thickness element with stiffness K_T assigned tangential to the interaction plane. The stiffness of the contact element to resist the penetration between soil and caisson at the interface is defined as K_N. The zero-thickness element with stiffness K_N assigned normal to the interaction plane. Ideally, penetration of soil and caisson at the interface should not be permitted. To maintain this condition, the value of K_N need to be very high (may tend to infinity). Hence, this study considers a permissible 0.02mm penetration at the interface. The stiffness of the contact element was determined using the penalty method to avoid over or underestimating the magnitude of separation and sliding at the interface. The results of the penalty method are listed in Table 2. Table 2 indicates that the convergence in the displacement of caisson had been initiated from the 4th iteration, and complete convergence was noticed for the 6th iteration. Hence, this study considers stiffness of contact element equal to 100 times the modulus of the surrounding soil (E_S). The magnitude of top displacement of caisson obtained from sensitivity analysis performed with different domain widths are shown in Table 3. Table 3 revealed no significant difference in the magnitude of displacement obtained from numerical analysis performed with the domain widths of 31B and 41B. As a result, 31B was chosen as the domain width for the numerical analysis in this study. Optimum domain size has been determined by performing sensitivity analysis. This study considered uniform mesh for caisson and nonuniform mesh for the soil domain. The width of the soil domain element at the interface is 5m. The width of the soil element gradually increases from 5m to 11.5m on reaching the boundary of the soil domain. The uniform mesh width of the caisson is 0.5m. Sensitivity analysis has been performed considering the constant width of the element and varied depth of the element. The different element depths considered in this study are 0.5m,1m,1.5m,2m, and 2.5m. Table 4 indicates the convergence in the top displacement of caisson for the mesh depth 2m and above. Hence, this study considered 2m as the element depth. Figure 1 depicts the 3D model obtained from the sensitivity analysis. The numerical scheme considered in this study was validated using experimental results stated by Anagnostopoulos and Georgiadis (1993) [19]. This study developed a numerical model of the experimental setup specified by Anagnostopoulos and Georgiadis for validation. Figure 2 shows the numerical model developed for validation with the numerical scheme considered in this study and performed a numerical analysis with the material and loading conditions considered by Anagnostopoulos and Georgiadis (1993). The analysis considered the foundation subjected to a horizontal load of 130 N to check the accuracy of the present numerical scheme in predicting the lateral response of the foundation. Table 5 compares the magnitude of the lateral displacement at the top of the caisson obtained from the numerical study to the experimental results stated by Anagnostopoulos and Georgiadis (1993). According to Table 5, the maximum difference between computed and observed displacement values is less than 10%. As a result, the numerical scheme presented in this study has been validated to accurately capture the interaction of the foundation-soil system under lateral loading conditions.

Table 1: Material properties of soil domain

Parameters	Medium clay	Medium sand
Density (kg/m^3)	1500	2000
Reference mean effective confining pressure(kPa)	80	100
Reference shear modulus (MPa)	60	75
Reference bulk modulus(MPa)	300	200
Undrained cohesion(kPa)	37	0
Angle of friction (^{0})	0	37
Shear strain at failure	0.1	0.1
Pressure dependent constant	0	0.5
Phase transformation angle (^{0})	0	27

Table 2: Iterations of Penalty method for stiffness of interface element

Iteration	Normal stiffness (K_N)	Tangential stiffness (K_T)	Top displacement (m)	
			Medium clay	Medium sand
1	E_s	$0.01E_s$	0.4235	0.42017
2	10 E_s	0.1 E_s	0.42114	0.42033
3	100 E_s	E_s	0.42041	0.42044
4	200 E_s	2 E_s	0.41957	0.41998
5	100 E_s	10 E_s	0.41950	0.41960
6	100 E_s	100 E_s	0.41949	0.41947

Table 3: Sensitivity analysis with different domain widths

Width of the domain	Top displacement (m)
11*B*	0.127
21*B*	0.218
31*B*	0.323
41*B*	0.324

Table 4: Sensitivity analysis with different element depths

Depth of the element (m)	Top displacement (m)
0.5	0.323
1	0.372
1.5	0.402
2	0.423
2.5	0.430

Table 5: Validation of numerical scheme considered in the study

Load (N)	Top displacement of foundation (mm)		
	Present numerical analysis	Anagnostopoulos and Georgiadis (1993)	% Error
20	3.642	3.419	6.541
40	7.004	6.837	2.442
60	11.763	12.977	9.353
80	18.269	19.744	7.473
100	33.023	34.884	5.335
120	45.254	42.907	5.470
140	50.496	46.953	7.544

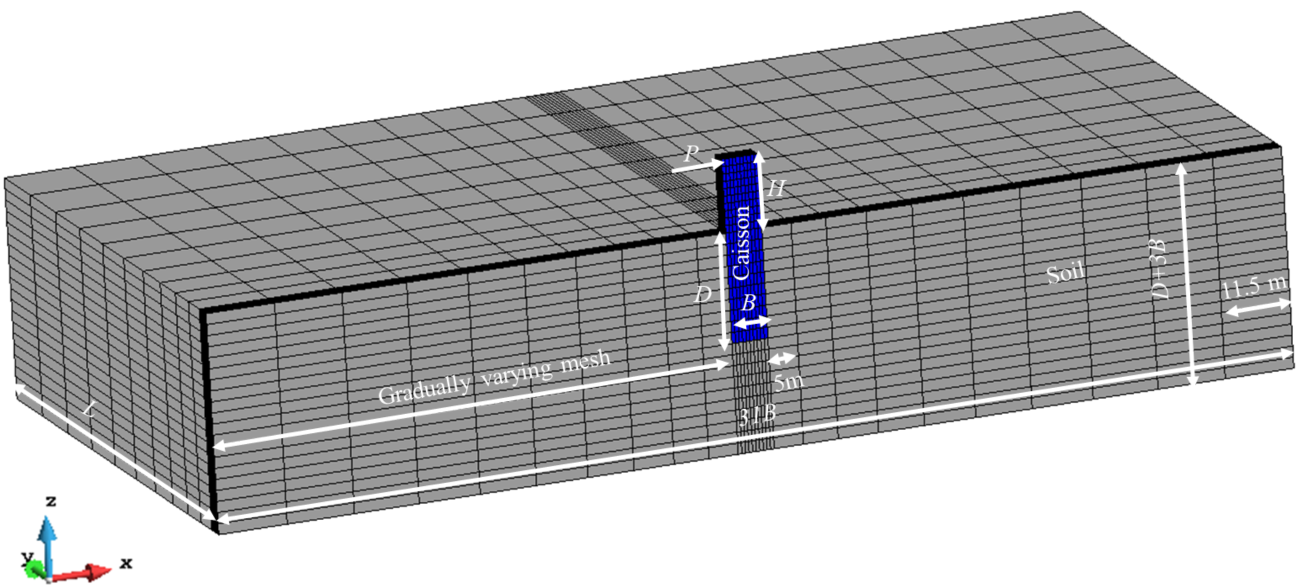

Fig. 1: Numerical finite element models of three dimensional (3D) caisson-soil system

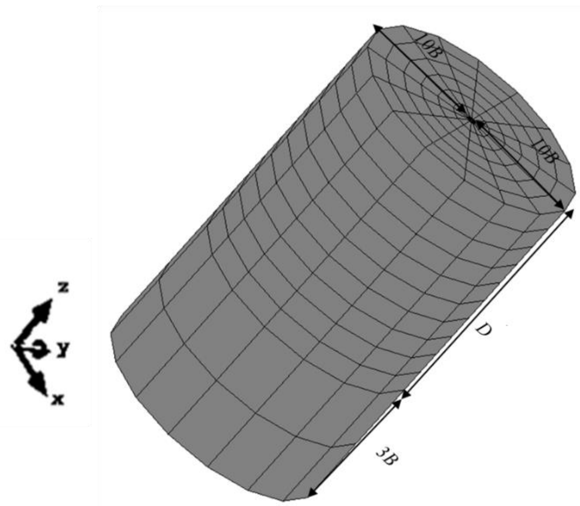

Fig. 2: Numerical model of the experimental setup of Anagnostopoulos and Georgiadis (1993)

4. Results and Discussion

The numerical analysis carried out in this study was by applying load incrementally until the ground level displacement (Ux) of the caisson reaches 0.15B (i.e., 0.9m for this study). A ground level displacement of not more than 0.15B is stated to be functionally acceptable [20]. Figure 3 presents the load-displacement curves of the 3D model of laterally loaded caissons with an L/B ratio equal to 1 embedded in medium clay and medium sand. Figure 3 compares the load-displacement curves obtained from the numerical analysis that takes into account both soil and interface nonlinearities with the load-displacement curves obtained from the numerical model that considers soil nonlinearity but ignores interface nonlinearity by assuming perfectly bonded conditions at the interface. It is clear from Figure 3 that the curves derived from the numerical analysis with soil nonlinearity and the perfectly bonding condition at the interface are linear. Whereas the load-displacement curve derived from the numerical analysis with soil and interface nonlinearities are nonlinear in the trend. It is worth noticing the nonlinear trend of the load-displacement curves due to interface nonlinearity. It is also interesting to observe from figure 3 that the load-displacement curves predicted with interface nonlinearity and perfectly bonding conditions coincide with small loads. However, the difference between the curves increases for a large magnitude of loads. This is mainly because of the incapability of numerical analysis with perfect bonding conditions at the interface to capture the loss of contact between soil and caisson for high loads. The relative separation and sliding due to loss of contact between soil and caisson at the interface for the high magnitude of loads have been overlooked in a numerical analysis obtained with the perfectly bonded condition due to its assumption of no relative movements between soil and foundation at the interface. Whereas numerical analysis with interface nonlinearity and its modeling captures the propagation of relative sliding and separation at the interface for high magnitude loads and large displacements. Hence, numerical analysis with perfect bonding conditions at the interface is valid only for the low magnitude of loads and small displacements. However, interface nonlinearity and its modeling in the numerical analysis cannot be neglected while predicting the lateral response of caisson subjected to high magnitude loads and large displacements. Figure 4 compares the load-displacement curves obtained from numerical analysis while accounting for soil and interface nonlinearities to the curves obtained from numerical analysis with soil linearity and interface Nonlinearity for 3D caisson embedded in medium clay and medium sand. Figure 4 indicates no significant differences between the lateral load-displacement curves obtained considering soil linear or nonlinear material. Figure 4 also presents that the load-displacement curves obtained from the numerical models considering with and without soil nonlinearity are nonlinear in trend. This indicates that the relative sliding and separation mechanism at the interface governs the lateral displacement of the caisson, irrespective of soil nonlinearity. Hence the observation obtained from this study highlight that the contribution of interface nonlinearity is more predominant than material nonlinearity during the lateral displacement of the caisson. Figure 5 compares the load-displacement curves obtained from the 2D model and the 3D model for caissons embedded in medium clay and medium sand with different L/B ratios (1,5 and 7) to determine the optimum L/B ratio for which the 3D model can be treated as a 2D model. The lateral load-displacement curves plotted in Figure 5 are obtained from numerical models considering both material and interface nonlinearities in the analysis. Figure 5 shows that the caissons with L/B ratio 1 are stiffer than L/B ratios 5 and 7. Figure 5 also indicates the value to L/B ratios increases, the load-displacement curves of 3D caissons converge towards the load-displacement curve obtained from 2D. Hence, observation from Figure 5 depicts modeling caisson with L/B ratio 1 as 2D will result in underestimating the lateral capacity of the caisson- soil system. The findings of this study from Figure 5 illustrates caisson-soil systems with large L/B ratios can be modeled as 2D models. However, large L/B ratios are only applicable for retaining walls, and foundations usually have L/B ratios ranging from 1 to 3. Therefore, this study suggests that model caissons with L/B ratios 1 to 3 should always be modeled as 3D finite element models.

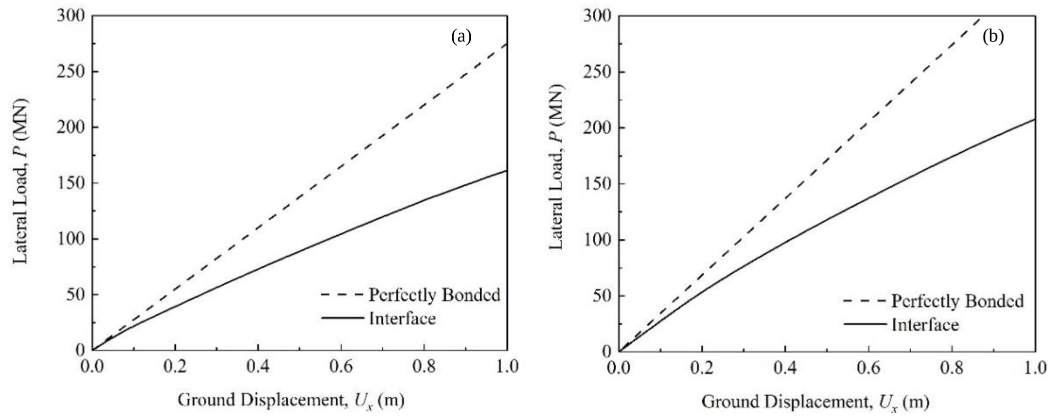

Fig. 3: Load- displacement curves to observe the significance of interface nonlinearity (a) medium clay (b) medium sand

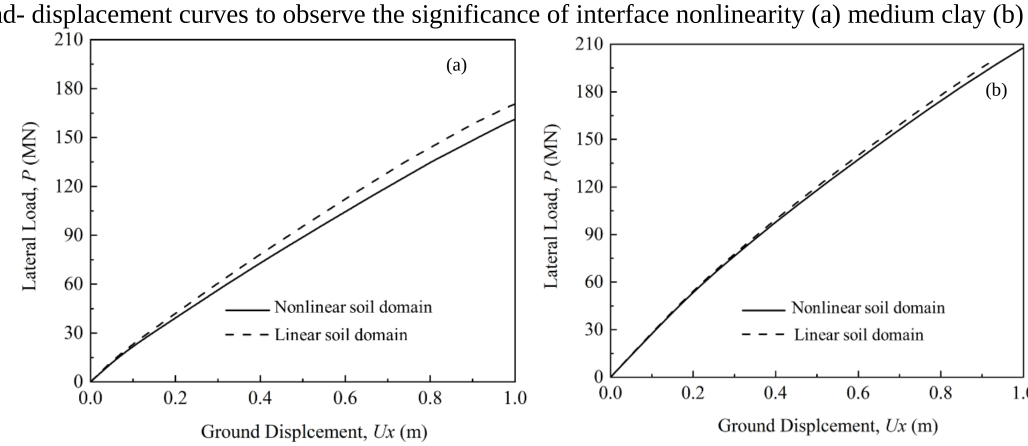

Fig. 4: Load- displacement curves to observe the significance of soil nonlinearity (a) medium clay (b) medium sand

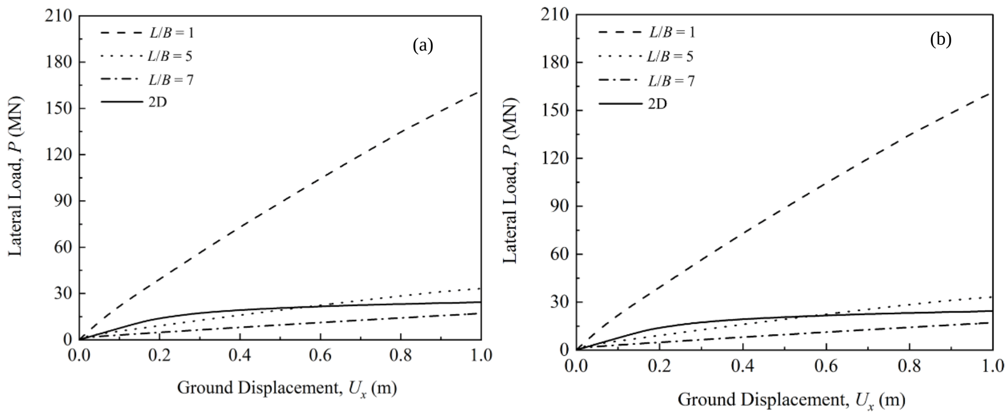

Fig. 5: Load-displacement curves to determine optimum *L*/*B* ratio (a) medium clay (b) medium sand

5. Conclusion

This paper provides insight to the designer for developing a numerical model to accurately estimate the lateral response of caisson embedded in nonlinear soils. This study develops a finite element model of 3D and 2D of a laterally loaded rectangular caisson embedded in medium clay and medium sand. This study models the material nonlinearity by modeling soil nonlinearity using a multi yield surface plasticity model and assumes caisson as a linear elastic body. The modeling of geometric nonlinearity is done using zero thickness contact elements with elastic perfectly plastic constitutive models at the interface. The results of this study showed that caissons with high L/B ratios can be modeled as 2D; however, caissons with L/B ratios ranging from 1 to 3 should be modeled as a 3D model only for accurate results. It is also observed that the results obtained from the numerical model with perfect bonding conditions at the interface are only accurate to predict the lateral displacements of caisson for small magnitudes of load and displacements. Whereas the numerical model with interface nonlinearity should not be overlooked while estimating the lateral response of caisson for large loads and displacement. The results obtained from this study demonstrate the predominant contribution of geometric nonlinearity over material nonlinearity during the lateral displacement of the caisson. Hence, this study highlights the importance of accurately modeling geometric nonlinearity due to interface and emphasizes the need to understand the interface mechanism in detail to effectively predict the response of the caisson-soil system. The future scope of this study is to check the accuracy of modeling nonlinear interaction between soil and caisson at the interface using simple elastic perfectly plastic constitutive models in numerical analysis. Further, this study aspires to understand the interaction between the soil and caisson at the interface using shear tests or discrete element method and develop a nonlinear constitutive model that can be easily implemented in numerical analysis to accurately capture the interaction at the interface while predicting the response of caisson.

Acknowledgments

The authors of this paper are highly thankful to Shiv Nadar University for providing facilities to conduct this research.

References

[1] S.K.Jain," Earthquake safety in India: achievements, challenges, and opportunities," Bulletin of Earthquake Engineering, vol.14, No.5, pp.1337-1436,2016.

[2] S.Ponnuswamy, *Bridge engineering*. Tata McGraw-Hill Education, 2008.

[3] K.Ishihara," Terzaghi oration: Geotechnical aspects of the 1995 Kobe earthquake", in *Proceedings of the ICSMFE International Conference on Soil Mechanics and Foundation Engineering-International Society For Soil Mechanics and Foundation Engineering*, AA Balkema,1997.vol.4, pp. 2047 – 2073.

[4] S. P. G.Madabhushi, D.Pate, S. K. Haigh," Geotechnical Aspects of the Bhuj Earthquake," *EFFIT Report on the Bhuj Earthquake*, Institution of Structural Engineers, London, UK,2005.

[5] IRC: 45, 1972. Recommendations for Estimating the Resistance of Soil below the Maximum Scour Level in the Design of Caissons of Bridges. IRC, New Delhi.
[6] IRC:78-1983, Standard Specifications and Code of Practice for Road Bridges, Section: VII, Foundations and Substructure. IRC, New Delhi.
[7] JRA, Japan Road Association," Specifications for Highway Bridges," Tokyo,2012.
[8] M.Saberi, C.D.Annan, and J.M.Konrad," On the mechanics and modeling of interfaces between granular soils and structural materials."Archives of Civil and Mechanical Engineering, vol.18, pp. 1562-1579.
[9] K.Karapiperis and N. Gerolymos," Combined loading of caisson foundations in cohesive soil: finite element versus Winkler modeling." Computers and Geotechnics., vol. 56, pp. 100-120,2014.
[10] A.Zafeirakos and N. Gerolymos," Bearing strength surface for bridge caisson foundations in frictional soil under combined loading." Acta Geotechnica., vol.11, no. 5, pp.1189-1208,2016.
[11] M.Kumar and K.Chatterjee," A Numerical Study on Lateral Load Response of Caissons in Static Conditions," In Geo-Congress 2020: Foundations, Soil Improvement, and Erosion, Reston, VA: American Society of Civil Engineers, pp. 15-22, 2020.
[12] G.Mondal, A. Prashant, and S.K. Jain," significance of interface nonlinearity on the seismic response of a well-pier system in cohesionless Soil." Earthquake Spectra, vol.28,no.3,pp. 1117-1145,2012.
[13] V.Kumar, " Foundation of Bridges on River Ganges in India," IABSE REPORTS, pp.351-368,1999.
[14] IRC:78-2000, Standard specifications and code of practice for road bridges, Section: VII, Foundations and Substructure (Second Revision). IRC,NewDelhi.
[15] F.McKenna and G.Fenves.(2001)," The OpenSees Command Language Manual:version1.2," Pacific Earthquake Engineering Center, Univ. of Calif., Berkeley. Available:http://opensees.berkeley.edu
[16] V.K. Papanikolaou, T. Kartalis-Kaounis, V.K. Protopapadakis, and T.Papadopoulos, "*GiD+OpenSees Interface : An Integrated Finite Element Analysis Platform*", Lab of R/C and Masonry Structures, Aristotle University of Thessaloniki, Greece,2017.
[17] IS:456- 2000, Plain and Reinforced Concrete. Code of Practice (4th revision).IS, New Delhi.
[18] Z.Yang, "Numerical Modeling of Earthquake Site Response Including Dilation and Liquefaction," Ph.D. Thesis, Department of Civil Engineering and Engineering Mechanics, Columbia University, New York, 2000.
[19] C.Anagnostopoulos and M.Georgiadis," Interaction of axial and lateral pile responses," Journal of Geotechnical Engineering, April, Vol.119, no.4, pp.793-798,1993.
[20] B.B.Broms, "Lateral resistance of piles in cohesive soils," ASCE Journal of Soil Mechanics and Foundation Eng. Div. vol 90, no.2, pp. 27–63,1964.