Dynamic Analysis of Cantilever Sheet Pile Walls in Liquefiable Soil Using FLAC2D

Akshay Pratap Singh and Kaustav Chatterjee

Abstract Retaining walls are generally used to support backfill soil. However, collapse of retaining walls due to earthquake forces has been observed in the past decades. In the present study, numerical modeling of a cantilever sheet pile wall passing through both liquefiable and non-liquefiable soil is carried out using finite difference-based computer program FLAC2D. The numerical analysis has been executed in cohesionless soil under both liquefiable and non-liquefiable soil conditions. The acceleration-time history of 1940 El Centro and 1989 Loma Prieta earthquakes is considered as the input dynamic loadings. From the results obtained, it is observed that both displacement and bending moment of sheet pile wall are increased in liquefiable soil as compared to non-liquefiable soil due to significant reduction in shear strength and stiffness of the soil. The present results can be used for design of cantilever sheet pile wall passing through liquefiable soil in earthquake prone areas.

Keywords Dynamic analysis · FLAC2D · Cantilever sheet pile walls · Liquefiable soil

1 Introduction

Failure of retaining walls due to sliding, overturning, or bearing capacity has been observed during various past earthquakes. Although failure mechanisms of rigid retaining walls have been analyzed significantly [4, 6]; however, due to complex nature of dynamic soil–structure interaction, an accurate study on flexible cantilever sheet pile wall is still lacking in literature. Flexible cantilever sheet pile walls are used to support moderate height of excavation in both cohesive and cohesionless

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A. K. Dey et al. (eds.), *Proceedings of the 7th Indian Young Geotechnical Engineers Conference*, Lecture Notes in Civil Engineering 195,
https://doi.org/10.1007/978-981-16-6456-4_2

soil as a temporary or permanent structure. It derives its stability from the passive resistance of soil near its toe.

The simplest approach in the seismic design of retaining structure is pseudo static approach in which the seismic forces are assumed constant over a time and space by equivalent acceleration expected at the ground surface assuming a collapse mechanism. Although it is convenient to use but, its assumptions make it incapable for a real-time damage of structure. Further, in a seismically active zone, a cantilever sheet pile passing through a saturated loose, cohesionless soil may be subjected to liquefaction induced by earthquake forces, causing it to fail. Therefore, considering the need of present scenario, this paper investigates the dynamic analysis of cantilever sheet pile wall in liquefiable as well as non-liquefiable soils using finite difference-based computer program FLAC2D [5].

2 Numerical Modeling of Sheet Pile

The numerical modeling of a cantilever sheet pile wall passing through both liquefiable and non- liquefiable soil is carried out using an explicit finite difference-based computer program FLAC2D (Fast Lagarangian Analysis of Continua) assuming a two-dimensional and plane strain problem. The finite difference method is a well-established approach that can be used for analysis of any problem in geotechnical engineering. The analysis is done in two phases; namely excavation phase and dynamic phase. In excavation phase, Mohr–Coulomb elastic perfectly plastic soil model is used for establishing in-situ stress, and excavation with wall installation is done assuming shear modulus of soil as 0.3 times the original shear modulus of soil [3]. The boundary conditions of the model are standard boundary conditions. Fixed boundary conditions are provided at the base of the soil model in both x- and y-directions under static conditions, while for lateral boundaries, the x-direction was fixed and y-direction was kept free. In dynamic phase, the seismic motion is applied at the base of soil model. Free field and quiet boundary conditions are applied in the model for preventing reflections at the boundary.

An elastic-perfectly plastic Mohr–Coulomb model with non-associated flow rule is used for modeling the non-liquefiable soil (properties given in Table 1), and Finn-Byrne model is used to simulate liquefiable soil. For present study, the model assumed is shown as in Fig. 1 with excavation height $h = 3$ m and embedded depth $d = 5$ m. The sheet pile used is of type SKZ 20 [8] with properties given in Table 2. The dimensions of grid are chosen carefully to avoid filtering of high frequencies and must be less than one-tenth of the wavelength associated. The sheet pile is represented by beam elements.

In seismic analysis, material damping developed due to viscous properties of soil are also applied using the numerical fits to Seed and Idriss [7] data for sand as given in FLAC2D. The S-shaped curve of secant shear modulus versus logarithm of cyclic strain is used in default hysteresis model. The secant modulus M_s is expressed as:

Table 1 Soil properties considered in present study [adopted from Chatterjee et al. [2] and Bowles [1]]

Properties	Values
Density (kg/m^3)	1400
Young's modulus (MPa)	36
Shear modulus (MPa)	13
Poisson's ratio	0.38
Friction angle (°)	30
Bulk modulus (MPa)	50
Soil–wall interface angle (°)	20

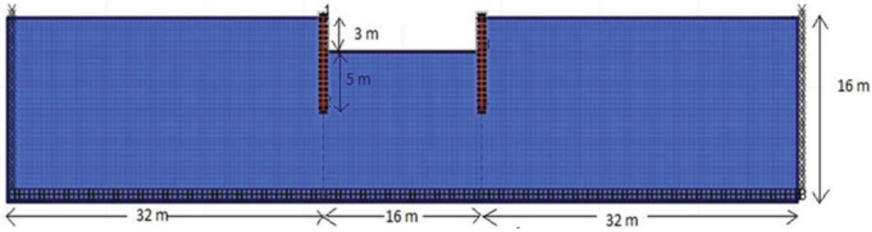


Fig. 1 Sheet pile model considered in the present numerical study

Table 2 Properties of sheet pile adopted from Skyline steel [8]

Type	Cross section area (cm^2/m)	Section modulus (cm^3/m)	Moment of inertia (cm^4/m)
SKZ 20	127	1704	34,618

$$M_s = s^2(3 - 2s) \tag{1}$$

where $s = (L2 - L)/(L2 - L1)$ and L is logarithmic strain. The parameters $L1$ and $L2$ are two extreme values of logarithmic strain and having values of -3 and 1, respectively, in the present study. For liquefaction analysis, Finn-Byrne liquefaction model is used which is given in FLAC2D with water table corresponding to dredge level in model. The water is modeled with a density of 1000 kg/m^3 and bulk modulus of 2 GPa.

2.1 *Input Seismic Motions*

Two different acceleration-time histories are applied at the base of model in this present analysis, i.e., 1940 El Centro and 1989 Loma Prieta input motions. The properties of these two ground motions such as maximum ground acceleration, Arias intensity, response spectrum, and others are given in Table 3.

Table 3 Strong motion parameters of earthquake motion in present study

Earthquake strong motion parameters	1989 Loma Prieta Earthquake	1940 El Centro Earthquake
Date of occurrence	18 October 1989	19 May 1940
Moment magnitude (Mw)	7	6.9
Bedrock level acceleration (g)	0.33	0.2
Arias intensity (m/s)	1.108	1.126
Response spectrum (m/s^2)	12.26	0.639
Peak ground displacement (m)	0.1226	6.32

2.2 Validation of Present Study

In order to validate the present numerical model, the results of this model in dry sand were compared with the results of Conti et al. [3]. In the numerical analysis, Conti et al. [3] used an elastic perfect plastic Mohr–Coulomb model for the dry soil with soil friction angle $\phi = 35°$, cohesion $c = 0$, and density $= 2.04$ Mg/m^3. The retaining walls were modeled having a diameter of 0.6 and 0.7 m spacing bored piles with bending stiffness $= 2.7 \times 10^5$ kNm^2/m and soil–wall interface angle 20°. The total length of retaining wall was 8 m with excavated depth $H = 4$ m. 1976 Friuli earthquake, which occurred in Tolmezzo, having peak ground acceleration 0.35 g is used for comparing the present results with that of Conti et al. [3].

It is observed from Fig. 2, that the bending moment obtained in the present study shows a similar trend and matches reasonably well with the bending moment magnitudes of Conti et al. [3] under both static as well as dynamic condition. The slight

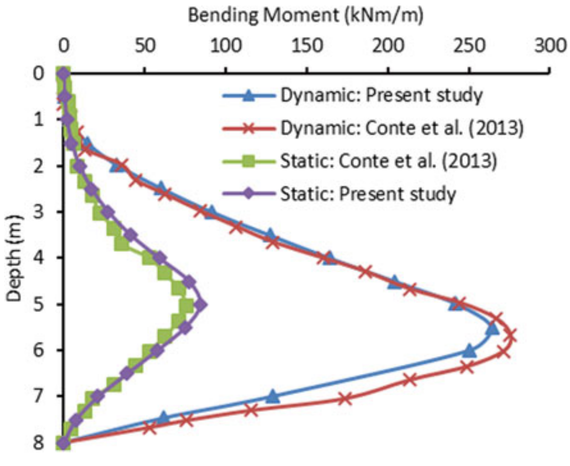


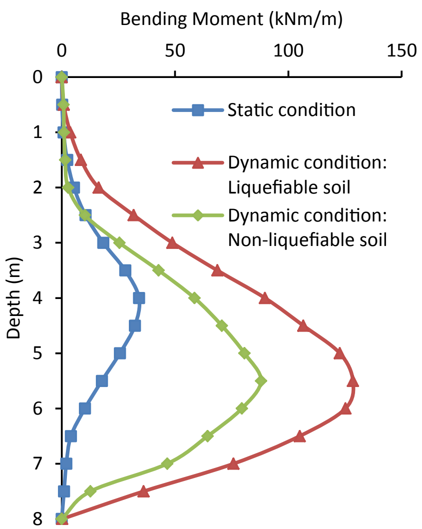
Fig. 2 Comparison of bending moment obtained in the present study with previous results for both static and dynamic conditions

variation in results may be attributed to the grid size difference of two models. In present study, a uniform grid size of 0.5 m is maintained in the model, while in Conti et al. [3] a grid size of 0.33 m was considered near the embedded wall consisting of total 4838 elements.

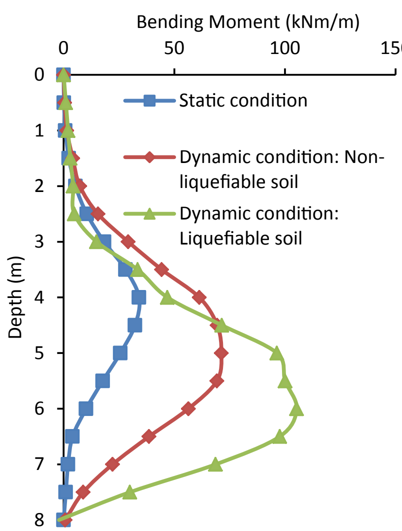
3 Results and Discussion

After validating the present numerical model with the results of Conti et al. [3], a parametric study of sheet pile wall passing through liquefiable soil and subjected to two different earthquake motions is carried out using FLAC2D computer program. The dimensions of the model are kept constant (excavated height $h = 3$ m and embedded depth $D = 5$ m) when subjected to both the ground motions, and bending moment, lateral displacement and net pressure data are analyzed for static as well as seismic condition.

In static conditions, the maximum bending moment observed is 34.1 kNm/m which increased significantly under dynamic conditions. When 1940 El Centro ground motion having bedrock level acceleration 0.2 g is applied at the base of the soil model, the bending moment increased to 88 kNm/m in non-liquefiable soil, and for liquefiable soil conditions, the bending moment increased to 128 kNm/m as shown in Fig. 3a. The increase was about 2.5 and 3.75 times in non-liquefiable and



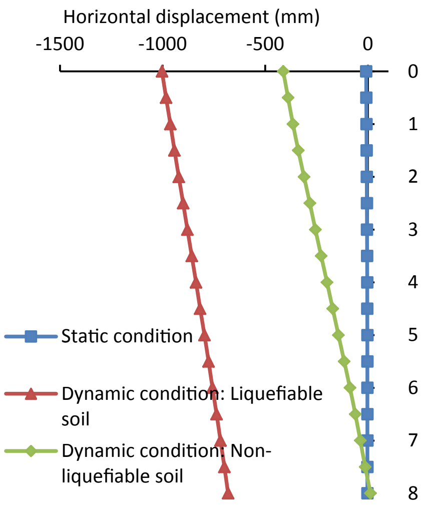
(a)



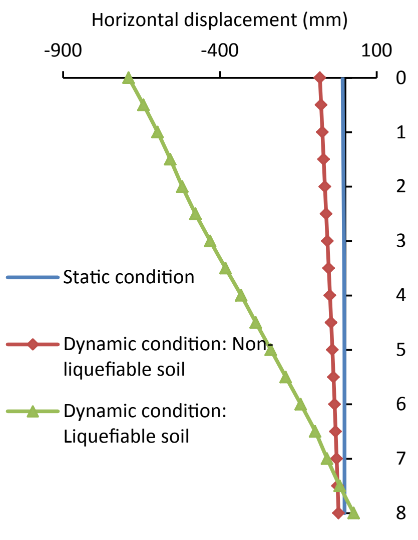
(b)

Fig. 3 Comparison of bending moment for **a** 1940 El Centro earthquake and **b** 1989 Loma Prieta earthquake liquefiable soil, respectively, as compared to static conditions. Similarly, when 1989 Loma Prieta ground motion is applied, the maximum bending moment increased to 71.3 kNm/m and 105.3 kNm/m in non-liquefiable and liquefiable soil, respectively, as shown in Fig. 3b. On comparing the bending moment due to both ground motions, it is observed that maximum bending moment occurred due to 1940 El Centro ground motion with bedrock level acceleration 0.2 g than 1989 Loma Prieta ground motion having bedrock level acceleration of 0.33 g. This may be due to the large bracketed duration of former seismic motion. It is also observed that the depth of sheet pile at which the maximum bending moment occurs becomes more in dynamic condition than static condition. Further, the maximum bending moment occurred at the instants after the time of peak ground acceleration. At the time of maximum acceleration, the backfill soil remains at active limit condition, and after that, the horizontal stresses are greater than the corresponding active values on retained side [3].

The displacement of the sheet pile is also influenced by the input seismic motions. In static condition, the wall displacement measured 8.55 mm which drastically increased to 412.3 and 1000 mm in non-liquefiable soil and liquefiable soil, respectively, for 1940 El Centro ground motion as shown in Fig. 4a. The lateral wall displacement observed in 1989 Loma Prieta earthquake, as shown in Fig. 4b, are 81 and 691 mm in non- liquefiable soil and liquefiable soil, respectively, which are less than 1940 El Centro ground motion. From net pressure diagrams as shown in Fig. 5, it is observed that the active pressure on backfill side is higher in dynamic condition than static condition. The increase in dynamic active earth pressure in backfill soil



(a)



(b)

Fig. 4 Comparison of lateral wall displacement for **a** 1940 El Centro earthquake and **b** 1989 Loma Prieta earthquake

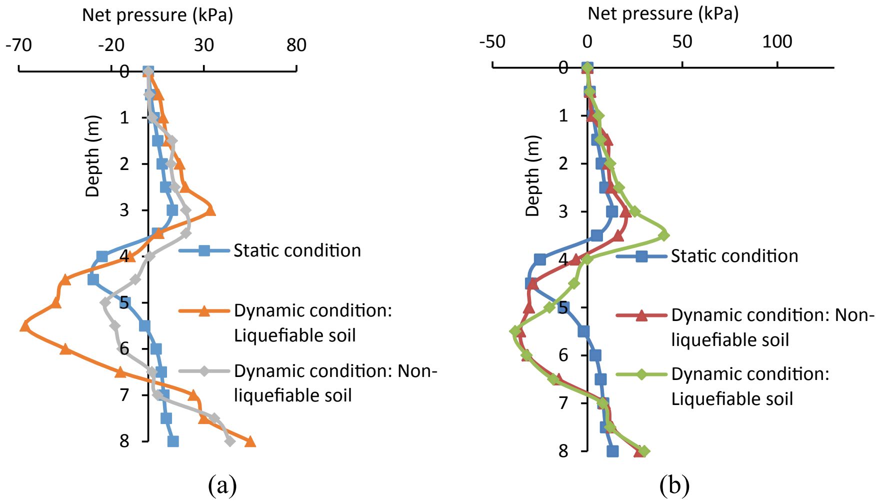


Fig. 5 Comparison of net pressure for **a** 1940 El Centro earthquake and **b** 1989 Loma Prieta earthquake

Table 4 Maximum bending moment and horizontal displacement values at different conditions

Parameters	Static condition	Dynamic condition			
		1940 El centro earthquake		1989 Loma Prieta earthquake	
		Non-liquefiable soil	Liquefiable soil	Non-liquefiable soil	Liquefiable soil
Maximum bending moment (kNm/m)	34.1	88	128	71.3	105.3
Horizontal displacement (mm)	8.55	412.3	1000	81	691

contributes to the mobilization of net passive earth pressure below the dredge level. The maximum bending moment and horizontal displacement in the sheet pile for static and dynamic conditions are tabulated in Table 4.

4 Conclusion

The present study analyzes the influence of two earthquake motions on a cantilever sheet pile wall in both liquefiable and non-liquefiable soil through numerical analysis using FLAC2D computer program. The results obtained from the present numerical study are compared with available solutions in literature and good agreement between the results are observed. The major conclusions drawn from the present study are:

- The maximum bending moment under dynamic conditions are greater than the static conditions.
- The bending moment observed in the sheet piles are more in liquefiable soil when compared to non-liquefiable soil under dynamic conditions. This is due to the reduction in shear strength and stiffness of the saturated soil.
- The maximum bending moment occurred after the time instants of peak ground acceleration as the horizontal stresses increased after limit conditions at time of peak ground acceleration is attained.
- The bending moment occurred and horizontal displacement was observed to be higher for 1940 El Centro earthquake having a bedrock level acceleration of 0.2 g than 1989 Loma Prieta earthquake having bedrock level acceleration of 0.33 g, due to large bracketed duration of the former motion.
- Hence, the present results using FLAC2D can be used for design of cantilever sheet pile wall passing through liquefiable soil in earthquake prone areas.

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