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INVESTIGATION OF SLURRY SUPPORTED TRENCH STABILITY UNDER SEISMIC CONDITION IN PURELY COHESIVE SOILS

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ABSTRACT

For city development, more and more tall apartments and office buildings are being built with basements that require slurry supported trenches to construct diaphragm wall. In the past, some investigators have studied the stability problem of trenches. It was indicated that the trench stability analysis is a three dimensional problem due to arching effect from the finite length of the trenches. However, it is not common to account for the seismic stability analysis of trenches. This study therefore aims to conduct research on the stability of slurry supported trenches under seismic conditions. In this study, finite-element upper and lower bound limit analysis methods (LA) are used to study the stability of trenches under bentonite pressure in purely cohesive clay by considering seismic conditions and to produce stability charts to be used for preliminary designs by engineers.

Key words: Finite-element limit analysis, slurry trenches, seismic coefficient, plastic zone.

1. INTRODUCTION

Trench excavation has been one of the oldest practices of geotechnical engineering and would continue being an essential part of this engineering field as long as the construction and modernization of our cities continue. Trenches for the construction of diaphragm walls of modern buildings and foundations have proven to be an efficient method to maintain soil stability. Whenever buildings, foundations and other underground structures are built, a form of water and soil lateral pressure retention is needed. For the construction of diaphragm walls, slurry supported trenches compared to other soil retention methods tend to cause less noise to the environment and safer for deep excavation projects in weak soils.

This study looked into one of the initial stages of diaphragm wall construction which is an excavated trench stabilized by bentonite slurry. In fact, the trench damages could delay the project and cost money on repaying and reconstruction, even if it is not caused by a seismic activity. Despite several studies conducted on trench stability, it is not common to consider seismic conditions. Generally speaking, engineers consider diaphragm wall trenches as temporary constructions and only conduct their designs under static conditions. In construction phase, a small to moderate earthquake with 30-year return period is usually used to determine the seismic coefficient, based on the site-specific seismic hazard analysis (SHA) or seismic design specification considering site effects. Therefore, the probability of such trenches being destroyed by these earthquakes is minimal. However, in most seismic active regions, the consideration would not be conservative. For Taipei city in Taiwan and the surrounding regions, average earthquakes are 170 times per year, including about 3 sensible earthquakes. Although the seismic vulnerability is low, it is difficult to conclude that the trench stability will not be influenced by seismic activities when comparing to diaphragm wall constructions in northern Taiwan.

Finite-element upper and lower bound limit analyses are believed to be suitable for this work. The methods developed by Lyamin and Sloan (2002a, 2002b) and Krabbenhoft *et al.* (2005) has been previously used by researchers on several studies, including analyses of two- and three-dimensional (2D & 3D) slope stability (Li *et al.* 2009, 2010; Tschuchnigg *et al.* 2015), strength reduction techniques (Tschuchnigg *et al.* 2015). However, this method has not been used analyzing the stability of slurry supported trenches, except Li *et al.* (2014) who used a quarter-trench model. Their study has inspired this study to move further into understanding the stability of slurry support trenches in purely cohesive soil with the consideration of seismic effects.

2. PREVIOUS STUDIES

For cohesionless soils, Morgenstern and Amir-Tahmasseb (1965) improved a 2D limit equilibrium theory for the stability analysis of a trench in cohesionless soils proposed by Nash and Jones (1963) by considering groundwater level (H_w), slurry level (H_{sl}), and the effect of suspensions in slurry density and its hydrostatic pressure. They defined arching effect and electro osmotic forces as dominant factors that affect the stability of slurry supported trenches. Suspensions were included in the analyses because they were important in finding the density of the bentonite slurry and hydrostatic pressure applied on the trench walls.

Morgenstern and Amir-Tahmasseb (1965) defined the factor of safety as the ratio of stabilizing force of the slurry and the horizontal force preventing wedge from sliding. Tsai and Chang (1996) used a 3D limit equilibrium method and considered arching

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effect in the formation of slip surface (wedge) for both horizontal and vertical sliding masses of a trench in order to improve the 2D model with end boundary effects. In fact, using 2D analysis for trench stability gives lower factors of safety which is not conservative.

A 2D analysis method for slurry trenches was proposed by Xanthakos (1994) in which the factor of safety (F) is expressed as the ratio of the hydrostatic slurry pressure p_{sl} to the active earth pressure p_a and the ground water pressure p_w.

$$F = \frac{p_{sl}}{p_w + p_a} \tag{1}$$

For the 3D factor of safety analysis, Tsai and Chang (1996) modified Eq. (1) by defining the factor of safety as the ratio of the tangent of the frictional angle to the tangent of the mobilized frictional angle as shown in Eq. (2). The factor of safety is obtained by iteration until the values for Eqs. (1) and (2) are almost identical with a difference of less than 1×10^{-2}.

$$\phi_m = \tan^{-1}\left(\frac{\tan\phi}{F}\right) \tag{2}$$

Tsai *et al.* (1998) took arching effect into account due to finite length of trench using limit equilibrium approach by considering vertical pressure acting upon a sandwiched soil layer. Their study modeled this sandwiched soil as a weak material composed between two plates. The stability of their trench was evaluated based on the slurry pressure preventing the weak soil layer from extrusion. They proposed different factors of safety for two different conditions. In Eqs. (3) and (4), c_u is undrained shear strength of weak soil. Equation (3a) indicated the vertical pressure (σ_v) is higher than the threshold (defined as $1.57c_u$), and Eq. (3b) indicated the vertical driving pressure is less than the threshold pressure.

$$F = \frac{p_{sl} - 0.43c_u}{\sigma_v - 1.57c_u} \qquad \text{for } \sigma_v > 1.57c_u \tag{3a}$$

$$F = \frac{p_{sl}}{0.43c_u} \qquad \text{for } \sigma_v \le 1.57c_u \tag{3b}$$

Tsai *et al.* (2000) conducted a full scale field experiment in sandy soil on the stability of a slurry filled diaphragm wall trench. Their study reported that the general failure of a slurry filled trench with time involves ground subsidence and cave in. The initial movement in their model trench was observed in the upper part, just below the guide wall. This shows that, the failure of trenches in cohesionless soils is not always a toe failure mode.

Filz *et al.* (2004) described the importance of the formation of filter cakes on the trench wall because an enhancing force to the lateral support provided by the bentonite slurry. This phenomenon was not clearly discussed by previous investigators. They however concluded that, the filter cake formation would be difficult to be produced if the surrounding soil is too coarse. This results in the slurry penetrating into surrounding soil (Filz *et al.* 1997) and providing less lateral support to prevent such soil movement.

Fox (2004) produced analytical solutions for the stability of slurry supported trenches. Limit equilibrium method was used to produce 2D and 3D analytical solutions for factors of safety and critical failure planes for both drained and undrained conditions. The trench failure mode considered by Fox (2004) is wedge failure. Li *et al.* (2014) presented stability charts for the stability of slurry supported trenches in purely cohesive soil. Their study used finite-element upper bound (UB) and lower bound (LB) limit analysis methods. Some of the shortcomings related to the limit equilibrium method such as assumptions of failure surfaces can be avoided. Han *et al.* (2015), Zhang *et al.* (2016), and Zhang *et al.* (2017) have investigated the stability of slurry supported trenches using various stability analysis methods and produced stability charts for trench design under static conditions. Moreover, the studies by Li *et al.* (2009, 2019, 2020) showed that the numerical limit analysis method is an ideal tool to perform pseudo-static analyses for slope stability evaluations.

Before Li *et al.* (2014), there were no stability charts produced to assess the stability of slurry supported trenches. The failure mechanisms of the trenches were defined by the plastic zones observed from the upper bound solutions. This study has been inspired by Li *et al.* (2014) with the addition of seismic forces to the trench model. The seismic force is considered by using pseudo-static approach which is a simplified method. By accounting for diaphragm wall construction time and sensible seismic events, complicated numerical modelling would not be necessary.

3. PROBLEM DEFINITION

In this study, a range of trench sizes has been adopted. Both half size and full size trenches are used to get a better understanding and view of the failure surfaces and surface influence zones (at ground surface). For the half trenches, it is expected to have a better view of the whole failure surface around the trench. Figure 1(a) and 1(b) show the models of the half and full trenches used in this study respectively. A simple cohesive soil model is used for both upper and lower bound limit analysis methods.

Reasonable mesh and the boundary sizes for the models are shown in Fig. 2. The meshes are not representative of the adopted mesh for all analyzed cases but are for illustration purposes only. The overall mesh dimensions were adjusted so that the statically admissible stress field for the lower bound analysis and a kinematically admissible velocity/plastic zones for the upper bound analysis were satisfied.

Figure 3(a) shows a typical triangular element adopted in the UB analysis (Lyamin and Sloan 2002b). Each node has two velocity components and each element is associated with a vector of three unknown stresses. Velocity fields obey an associate flow rule and satisfy their boundary conditions that are required to obtain UB solutions by minimize Eq. (4). Figure 3(b) displays a typical triangular element used in the LB analysis (Lyamin and Sloan 2002a). The stresses obey equilibrium and satisfy both their boundary condition and the yield function, and then LB solutions can be obtained by maximizing Eq. (5). In fact, UB considers internal power dissipation due to plastic shearing, and thus the UB plastic zones are used to discuss the trench failure mechanism.

$$W^{\text{int}} = \int_V \sigma\dot{\varepsilon}\, dV \tag{4}$$

$$Q = \int_{A_q} q\,dA + \int_V h\,dV \tag{5}$$

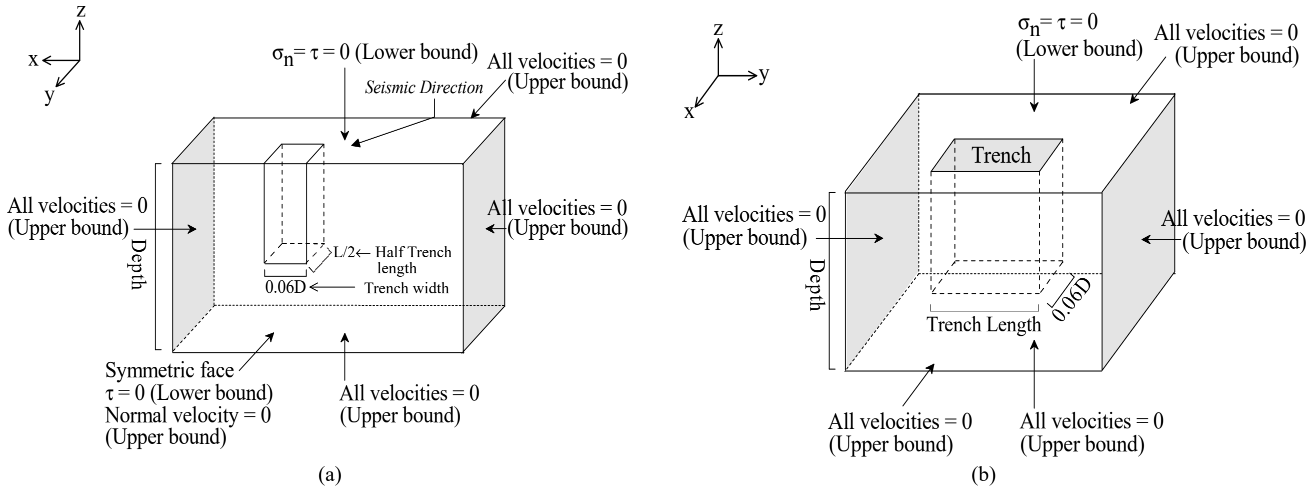


Fig. 1 3D representation of (a) half trench and (b) full trench

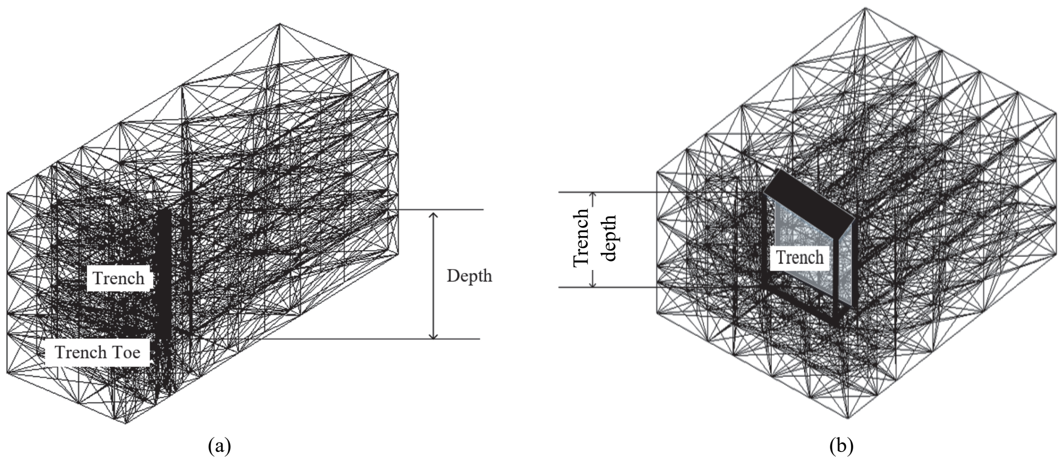


Fig. 2 Typical 3D finite-element limit analysis mesh for adapted (a) half trench and (b) full trench

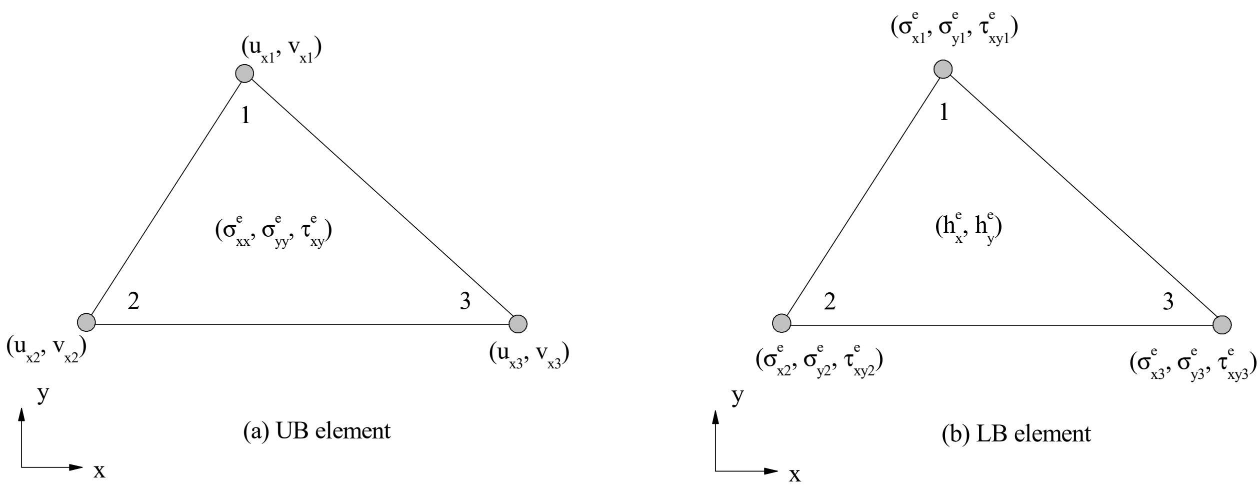


Fig. 3 Typical element adopted in finite-element limit analysis methods: (a) UB element and (b) LB element

The adjustment of mesh is important to capture the failure mechanisms properly and maintain the result accuracy. The bentonite slurry in the trench is assumed to be up to the top of the trench and thus the lateral pressures are provided increasing with depth. The full trench model gives a better view of the surface influence zones compared to the half trench model. This model is used in this study only to investigate the effects of the seismic forces in the directions of $\alpha = 45.0°$ and $\alpha = 67.5°$, as shown in Fig. 4.

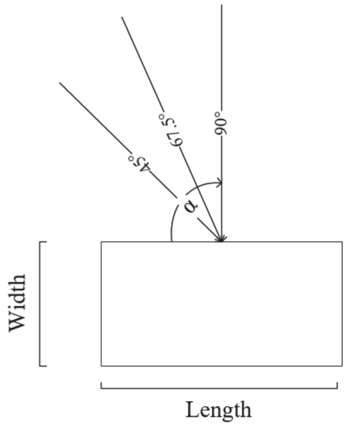


Fig. 4 Direction (α) of seismic forces applied to trench (top view of trench)

In this study, the model is a purely cohesive soil, the friction angle, $\phi = 0$ throughout the soil. Li *et al.* (2014) defined the linear increment of the undrained shear strength in normally consolidated as

$$c_u = c_{u0} + \rho z \tag{6}$$

where c_{u0} = undrained shear strength at the trench top; ρ = rate of increase of undrained shear strength with depth (kPa/m); and z = depth of the trench. Gibson and Morgenstern (1962) suggested that, when analyzing the end of construction stability of cuttings in normally consolidated clays, it is necessary to take into account the increase of strength with depth. The rate of increase in undrained shear strength with depth is defined by a parameter, ρ. It should be noted that the results obtained in this study are only applicable to purely cohesive soil.

In order to account for earthquake effects, seismic coefficient is adopted. It is generally expressed in relation to a percentage of gravity acceleration (Li *et al.* 2009). Based on the study of Li *et al.* (2014), trench stability is a 3D vertical slope stability problem which is demonstrated. Therefore, this study applied lateral seismic coefficient to the soil masses. In addition, the required construction time of the trench installation is very short. The excessive pore pressure in cohesive soil does not have enough time to be dissipated. It means that using undrained shear strength in analyses should be reasonable. Terzaghi (1950) recommended using seismic coefficient of $K_h = 0.1$ for severe earthquakes, $K_h = 0.25$ for violent-destructive earthquakes, and $K_h = 0.5$ for catastrophic earthquakes. In all cases, it is suggested that the design safety factor with respect to strength may be close to 1.0. Seed (1979) suggested the application of seismic coefficients of $K_h = 0.1$ for earthquakes of Richter's magnitude 6.5, and $K_h = 0.15$ for earthquakes of Richter's magnitude 8.5. For such cases, Seed suggested factor of safety of $F \geq 1.15$ for design purposes.

Theoretically, the lateral seismic loading is mainly induced by upward propagating shear waves from bedrock. Though different K_h values mentioned above in the literature, the seismic coefficient can be deduced from SHA or seismic code directly according to the seismic characteristics in the area considered. For simplicity, seismic coefficients of K_h = 0.1, 0.2, and 0.3, the range most frequently concerned, are adopted and applied to three different directions to the trench length which is common consideration. In this study, the seismic force was applied perpendicular to the trench, at 45 degrees and at 67.5 degrees as shown in Fig. 4. A reasonable trench boundary shown in Fig. 1 is employed based on seismic force direction. In fact, the seismic force can be simulated using two orthogonal components. Their resultant is the seismic force direction considered in this study. One of our aims is to find the effects of the seismic force direction on the stability of the slurry trench.

The ranges of input parameters used in this study are similar to those adopted by Li *et al.* (2014). As mentioned earlier, this study was aimed at generating stability numbers. These stability charts are expected to gain factors of safety for preliminary design of considering seismic effects. In the limit analyses, for given trench geometry (D, D/L), bentonite unit weight (γ_s), seismic coefficient (K_h), and soil inputs (c_{u0}, ρ), the optimized solutions of the upper-bound and lower-bound programs can be carried out with respect to the unit weight, γ. The dimensionless stability numbers, N for the upper bound and lower bound were calculated using Eq. (7) which was proposed by Li *et al.* (2014).

$$N = (\gamma - \gamma_s)DF / c_{u0} \tag{7}$$

where γ = soil saturated unit weight; γ_s = slurry unit weight; and F = factor of safety. In fact, limit analyses give the results at limit state, and thus F = 1. Equation 7 is modified based on the stability number proposed by Yu *et al.* (1998) for inhomogeneous undrained cut slopes. This modification accounted for the effects of slurry pressure on a slurry filled trench in undrained purely cohesive clay. This can be used to estimate the trench stability as proven by the cases demonstrated by Li *et al.* (2014).

In Fig. 5, H_s and γ_s denote the slurry height and slurry unit weight, respectively. In this study, the slurry level is assumed to be at the top of the trench and the lateral support provided by the guide wall is ignored. The slurry pressure acting on the trench wall increases linearly towards the toe of the trench. The inhomogeneous soil profile and bentonite slurry pressure adopted by Li *et al.* (2014) are used in this study to perform analysis (Fig. 5). The results obtained by Li *et al.* (2014) will be used for comparison purposes.

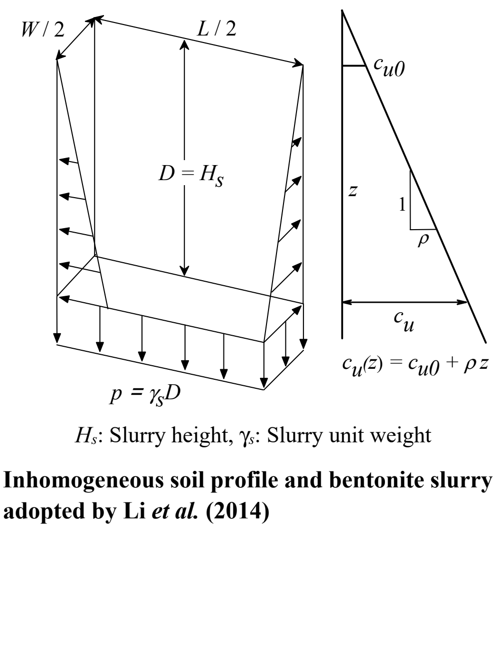


Fig. 5 Inhomogeneous soil profile and bentonite slurry pressure adopted by Li *et al.* (2014)

In this study, there was no common trend on how trench length to depth ratio, seismic force or rate of increase of undrained shear strength with depth (trench strength) affects the computing time of our analyses. However, the analyses of the full trench models require more computing time than the half trench models. The computing time moreover, depends on the mesh density and size. In this study, the mesh and boundary sizes were made to accommodate of both plastic points and stress fields and computing time. Despite, successful analysis on the seismic coefficient of K_h = 0.1, this study's progress on larger seismic coefficients was hindered by some numerical problems. When the seismic coefficients were increased to certain value, the results produced were not satisfactory because of unreasonable failure surfaces which will be discussed in Section of Results and Discussions. In fact, this phenomenon will be influenced by trench size and soil strength.

This challenge inspired this study to define the border horizontal seismic coefficient, K_{hb}. To obtain K_{hb}, the horizontal seismic coefficient for every trench model based on the upper bound limit analysis was increased slightly until its failure surface becomes unreasonable. Some trenches have reached the maximum target for horizontal seismic coefficient of K_h = 0.3. Stability charts for such models for K_h = 0.2 were therefore produced and presented. According to the analysis results and associated charts, when a seismic coefficient of K_h with proper return period is determined by SHA or seismic code provisions considering site effects, the stability number is obtained and thus the factor of safety against trench failure can be used to design.

4. RESULTS AND DISCUSSIONS

A slurry to water unit weight ratio (γ_s/γ_w) of 1.1 was considered to accommodate for a slurry unit weight higher than that of water (Ng and Yan 1999; Arai *et al.* 2008; Li *et al.* 2014). In this study the strengths provided by adjacent walls and guide wall have not been taken into account. Similar assumptions were used in previous studies (Tsai and Chang 1996; Ng and Yan 1999; Gourvenec and Powrie 1999; Fox 2004; Arai *et al.* 2008; Li *et al.* 2014). A range of L/D ratios between 0.1 and 1.0 and a range of rate of increase of undrained shear strength with depth between 0.0 to 5.0 were investigated, as shown in Fig. 4. The results of the stability numbers are bracketed within 20%. The solutions were presented using Eq. (7).

Figure 6 shows the stability numbers for the numerical upper bound and lower bound limit analysis for the half trench model and when the seismic forces are at a direction perpendicular to the trench face. It can be noticed that, the stability numbers increase with decreasing L/D ratio. This shows that, factor of safety of a trench decreases with the increase of trench length.

Figures 7 and 8 present the stability numbers for the upper bound and lower bound limit analysis of trench stability under seismic coefficient of K_h = 0.1 at an angle of 67.5° and 45° respectively to the trench face. These results were generated from the analysis on a full trench model. Similar trend to Fig. 6 was observed. It can be observed that the change in stability number is insignificant for different α values. However, the change in the seismic direction has a more significant effect on the surface influence zone. In addition, it can be observed in Figs. 6 to 8 when the rate of increase of undrained shear strength with depth is increased, the stability numbers increase.

Table 1 presents the border seismic coefficients (K_{bh}) which are determined by the proper trench failure mechanism. Theoretically, trench failure should be a toe failure mode. In this study, the failure surface was found to increase to infinitely long or deep when K_h exceeds a certain value. There is no proper failure mechanism which can be obtained for such cases. In reality, the trench failure surface is not infinite. Moreover, this value will be influenced by trench dimensions and soil strength. In this study, K_{bh} is therefore defined to represent the maximum K_h while a reasonable failure mechanism can be observed.

Figure 9 shows some of the irregular failure surfaces and how they transformed when the seismic coefficients were changed. The mesh sizes and boundary sizes were adjusted in an attempt to regulate the failure surfaces. However, increasing the length of the

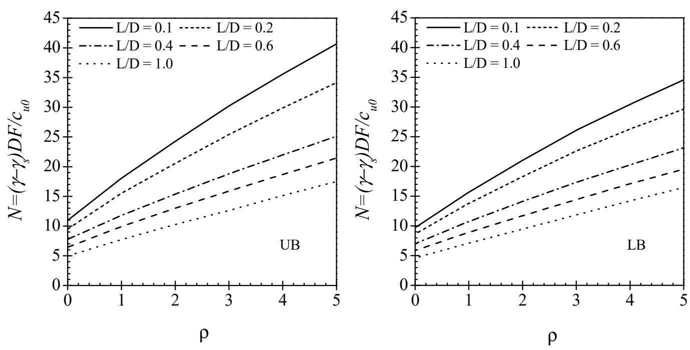


Fig. 6 Stability numbers for lower and upper bound analyses of K_h = 0.1 (α = 90°)

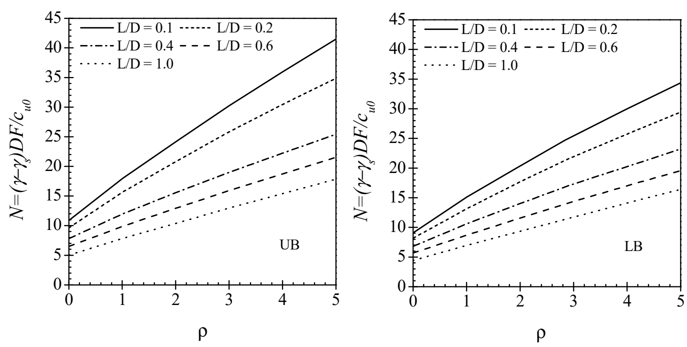


Fig. 7 Stability numbers for lower and upper bound analyses of K_h = 0.1 (α = 67.5°)

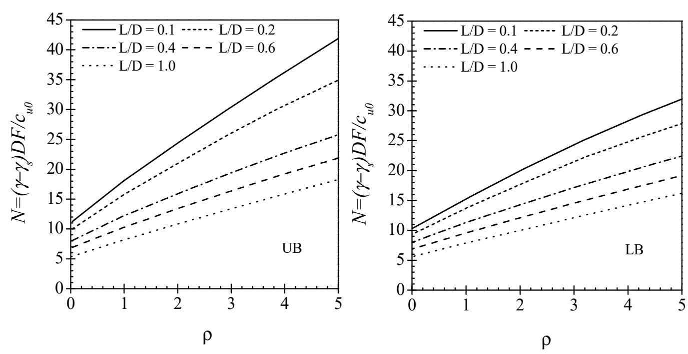
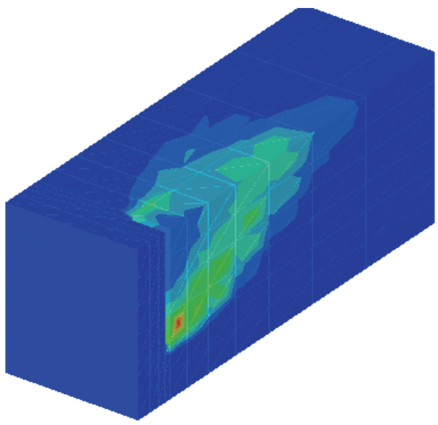


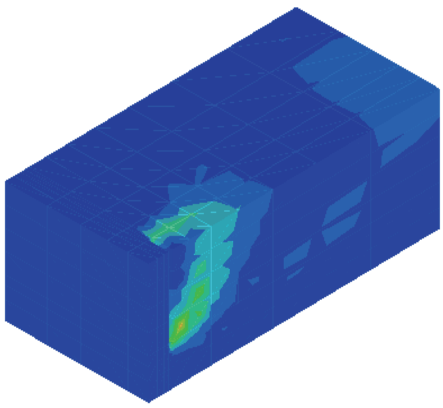
Fig. 8 Stability numbers for lower and upper bound analyses of K_h = 0.1 (α = 45°)

Table 1 Border seismic coefficient (K_{bh}) value from normal to abnormal failure surface ($\alpha = 90°$)

ρ	$L/D = 0.1$	$L/D = 0.2$	$L/D = 0.4$	$L/D = 0.6$	$L/D = 1.0$
0.00	0.110	0.125	0.135	0.140	0.150
0.25	0.125	0.130	0.150	0.150	0.180
0.50	0.130	0.140	0.165	0.180	0.200
1.00	0.140	0.150	0.190	0.200	0.240
3.00	0.160	0.200	0.260	0.300	0.300
5.00	0.183	0.230	0.300	0.300	0.300



(a) Reasonable Failure Surface ($L/D = 0.6$; ρ = 0.25; $K_h = 0.15$)



(b) Unreasonable Failure Surface ($L/D = 0.6$; ρ = 0.25; $K_h = 0.16$)

Fig. 9 Transformation between reasonable and unreasonable failure surfaces with increasing K_h value

boundary to ten times the trench depth had no effect on the convergence of the affected models. It can be observed from Fig. 9 that, at the border seismic coefficient, the trench has a toe failure surface. However, when seismic forces larger than the border seismic coefficient for a particular trench applied, the failure surface extends to the soil below the trench with an infinite length perpendicular to the trench which is not reasonable.

Figures 10 to 12 show the stability numbers generated when the seismic coefficient, $K_h = 0.2$ perpendicular to the trench face is applied. The analyses were conducted only for the trenches which have border seismic coefficients of $K_{bh} \geq 0.2$. Similar to previous results for $K_h = 0.1$, increasing ρ increases the stability number of a slurry trench. Compared to the results for $K_h = 0.1$, increasing the seismic coefficient to $K_h = 0.2$, reduces the stability number to about 12%.

According to Sloan (2013), contours of plastic or power dissipation, such as those shown in Fig. 13, provide a clear indication of zones of intense plastic shearing, and are useful tools for visualizing collapse mechanisms when using finite-element limit analysis to solve practical stability problems in geotechnical engineering. It can be noticed that the failure planes mostly form angles of θ = 45 ± 5° with the horizontal. The value of θ could be slightly influenced by the mesh and trench sizes. Generally speaking, the smaller trenches produced steeper failure planes compared to the larger trenches with more global and longer surface influence zones.

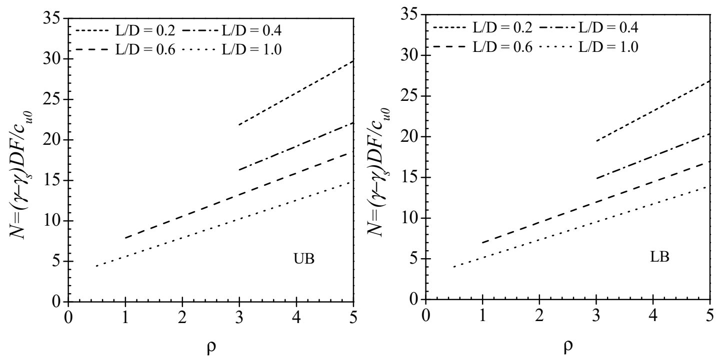


Fig. 10 Stability numbers for lower and upper bound analyses of $K_h = 0.2$ ($\alpha = 90°$)

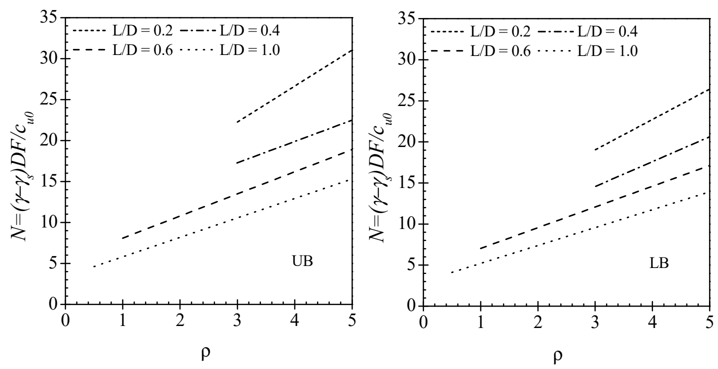


Fig. 11 Stability numbers for lower and upper bound analyses of $K_h = 0.2$ ($\alpha = 67.5°$)

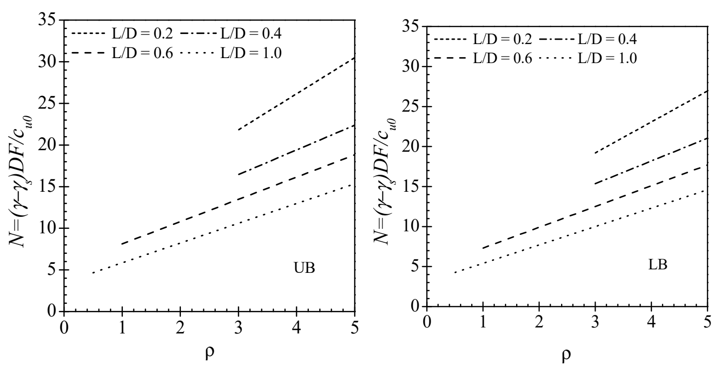


Fig. 12 Stability numbers for lower and upper bound analyses of $K_h = 0.2$ ($\alpha = 45°$)

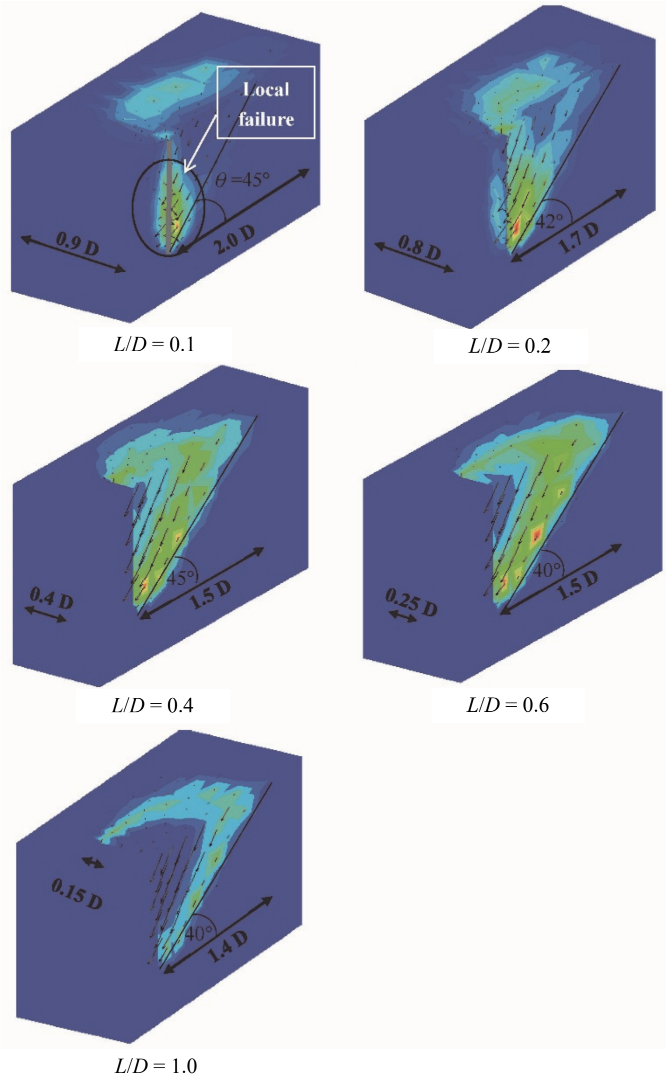


Fig. 13 Upper bound velocity fields and plastic zones for ρ = 1.0 for various trench size (α = 90°)

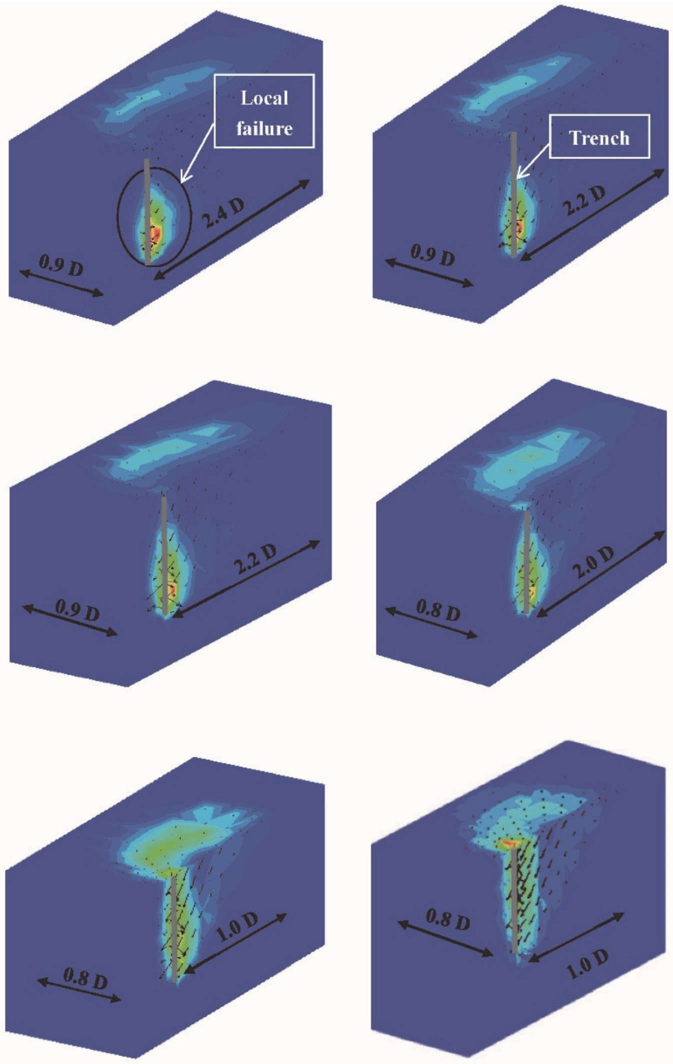


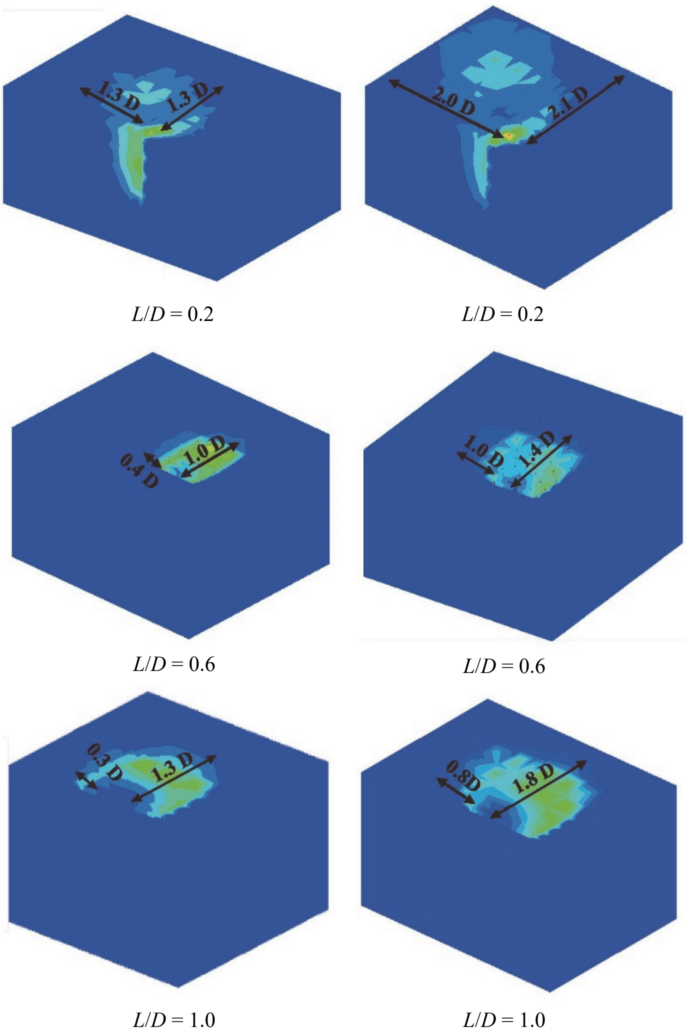
Fig. 14 Upper bound velocity fields and plastic zones for *L*/*D* = 0.1 for various ρ values (α = 90°)

In Fig. 14, when the value for ρ increases gradually from 0.0 to 1.0, the length of the influence zone did not change. Similar results were reported by Li *et al.* (2014). However, as the value of ρ increases to 3.0, the size of the failure surface decreases significantly to just about 1.5 times to the depth of the trench and to the same size as the depth of the trench (1.0 D) when ρ = 5.0. This is due to the fact that, the soil strengths both below and above the trench have increased significantly. The general influence distance parallel to the trench length increases with decreasing trench sizes. Moreover, as the size of the trench increases, the length of the surface influence zone perpendicular to the trench simultaneously decreases. However, from the plastic zones, it can be noticed that for the smaller *L*/*D* ratio, the failure zone is not identical in formation to the general failure wedge. The failure zone around the trench is longer with the larger trenches and more representative of a typical failure wedge. Figures 13 and 14 show that smaller *L*/*D* ratios and ρ are more likely to have local failures.

Figure 15 shows the potential failure modes from a full trench model with the seismic force acting at 45° to the trench face. It can be observed from the obtained stability numbers that, changing seismic direction does not have significant effects on the stability numbers. However, the surface influence zone of the trench failure is not always around the soil perpendicular to the trench but mostly in the direction of the seismic forces. It means that a change in the direction of earthquake forces can change the position of the surface influence zone.

Moreover, in Fig. 15, the length of influence zone is longer when $K_h = 0.2$ compared to the results when $K_h = 0.1$. For a given trench size, the same trend was observed for all other trench models for different ρ and α values. It should be noted that the general failure mode of the trenches is toe failure. Thus, the soil strength below the trench toe does not have much influence on the general stability of the trench, as reported by Li *et al.* (2014).

Li *et al.* (2014) applied their generated stability numbers into an assumed case. In their case a soil with strength, $c_u = 20$ kPa and unit weight, $\gamma = 20$ kN/m^3 was used. Their proposed trench depth (*D*) is 20m as shown in Fig. 16. From the solutions for *L*/*D* = 0.1, $\gamma_s/\gamma_w = 1.1$, and $K_h = 0.0$ the obtained factors of safety are 1.3 for UB and 1.11 for LB. If the solutions of $K_h = 0.1$ are used, the factors of safety decrease, 6.15% for UB and 7.2% for LB, as shown in Table 2. It was found that the decrement of safety factor is caused by an increment in the seismic coefficient.



(a) K_h = 0.1 (b) K_h = 0.2

Fig. 15 Upper bound velocity fields and plastic zones for different trench sizes with ρ = 3.0 for different seismic magnitudes (α = 45°)

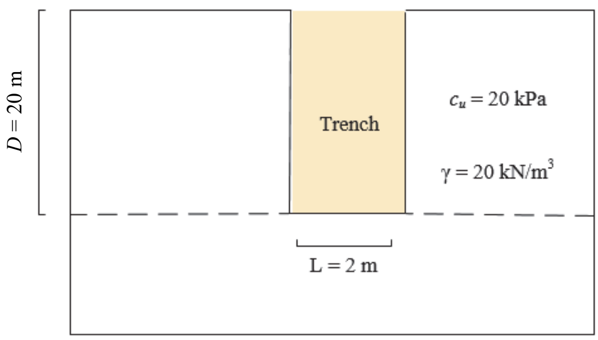


Fig. 16 Trench model for assumed case

Table 2 Comparison between trench stability under seismic and static conditions with an assumed case

Compared cases	N (UB)	N (LB)	F (UB)	F (LB)
ρ = 0.0, L/D = 0.1, γ_s/γ_w = 1.1 and K_h = 0.0	11.7	10.0	1.3	1.11
ρ = 0.0, L/D = 0.1, γ_s/γ_w = 1.1 and K_h = 0.1	10.97	9.29	1.22	1.03

5. CONCLUSIONS

This study investigated slurry supported trench stability considering various trench length to depth (L/D) ratios, rates of increase of undrained shear strength with depth, and seismic coefficients and their directions. The solutions for the upper bound and lower bound are bracketed within 20%. Based on the presented results, the following conclusions were made:

1. Half-trench or full-trench size should be adopted adequately to account for the applied seismic force direction. It can be observed that the plastic zones surrounding the trench which cannot be considered when only quarter-trench model is adopted.
2. Increasing the seismic coefficient increases the lateral stresses opposing the lateral contribution provided by the bentonite slurry, thus reducing stability numbers and factor of safety. In addition, the applied seismic force direction does not significantly affect the stability numbers, but changes the shape, size, and position of the surface influence zone.
3. Increasing the rate of increase in undrained shear strength with depth from 0.0 to 1.0 does not have significant effects on the length of the surface influence zone of trenches. However, it was found that when ρ > 1, the surface influence zone decreases with increasing ρ value.
4. As the length of the trenches got longer, (larger L/D ratio) the length of the surface influence zones perpendicular to the trench face decreases. Moreover, the length of the influence zone parallel to the trench decreases with increasing trench length as well.
5. Slurry density does not have a significant effect on the generated stability numbers. However, the factor of safety of the trench can be highly influenced by the ratio of L/D, ρ, and seismic coefficient.

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DATA AVAILABILITY

This study does not generate new computer codes. All data is included in the paper.

CONFLICT OF INTEREST STATEMENT

The authors declare no conflict of interest.

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