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Original Research Article

Numerical Analysis of the Multirow Arrangement of Small Diameter Steel Piles for Landslide Prevention

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Abstract

This paper intends to elucidate the prevention mechanism of small diameter steel piles (SDSP) and how multirow arrangement of the said piles affects the reinforced slope failure in landslide countermeasure. In this study, finite element analysis (FEA) employing Mohr-Coulomb's elastic-perfectly plastic soil model was carried out to simulate the real condition in which, the effect of the varying ground densities (Dr=30% and Dr=80%) and cross sections (10 mm x 10 mm square and 3 mm in diameter piles) of the reinforcing rods in piles' mechanism of landslide prevention were considered. Attempts also have been made to study the effect of introducing multiple rows arrangement of SDSP in terms of different cases of parametric study focusing on the effect of single, double and triple rows arrangements of those piles. Based on the results, it was observed that the shearing resistance in different pile cross sections is found to be significantly influenced by the variation of SDSP arrangement. However, irrespective of the piles arrangements, failure mode of a densely compacted ground is mainly governed by soil's shearing resistance mobilized at a higher strain, while bending stiffness (EI) of the reinforcing material is more dominant in loose ground condition.

Keywords: *Small diameter steel piles, shearing resistance, finite element analysis, lateral soil movement, landslide prevention, numerical modeling, slope stability, Toyoura sand, Mohr-Coulomb's soil model, ground density.*

1. Introduction

Landslide occurence in both natural and cut slopes represent a major threat not only to human life and properties but also indirectly to the environment because such disaster could culminate to immeasurable catastrophic lost. The immediate consequence of an excessive lateral soil movement leads to the decreasing piles shearing resistance; developed as a result of piles embedment into the underlaying layer, till the critical state is reached. The amount of shearing resistance taken by the stabilizing piles in landslide prevention differs substantially based on pile toes condition, ground support by lower stratum and the anchorage length of pile embedment whereas, the rate of shear is significantly governed by soil properties and lateral movement mechanism. At present, the installation of cast in-situ passive piles is widely adopted to stabilize active landslide prone areas as well as preventive measure in stable slopes. Slopes stabilization using passive (preventive) piles, with minimum diameter of 300 mm (Taniguchi T., 1967), is one of the oldest methods adopted in landslide prevention measures. Mechanism of such measure has been rigorously studied by various researchers (Ito T. and Matsui T., 1975; Fukuoka M., 1977; Poulos H. G., 1995; Chen L. T. et al., 1997; Chen C. Y. and Martin G. R., 2002), from which the results have been integrated as design elements in actual practice. In recent years, a new type of pile called small diameter (90 mm-300 mm) steel pile otherwise known as micropile has been developed and is expected to function both as passive piles as well as reinforcing rods in slope stabilization technique (Hazarika H. et al., 2011; Watanabe N. et al., 2011; Mujah D. et al., 2012). Passive piles that provide lateral resistance are installed vertically in a single row arrangement.

Fig. 1(a) shows the load transfer mechanism of the sliding mass above the failure surface, assumed to be strengthened by the discretely placed piles, by forming a barrier that resists horizontal soil movement due to lateral force. In passive piles, the resisting force comes mainly from the pile response in terms of shear and bending resistances. In contrast, earth reinforcements are installed in the direction normal to the slope surface. Thus,

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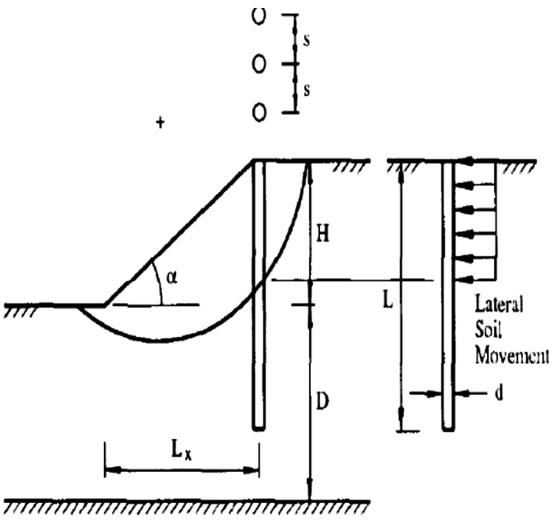


Fig.1(a)

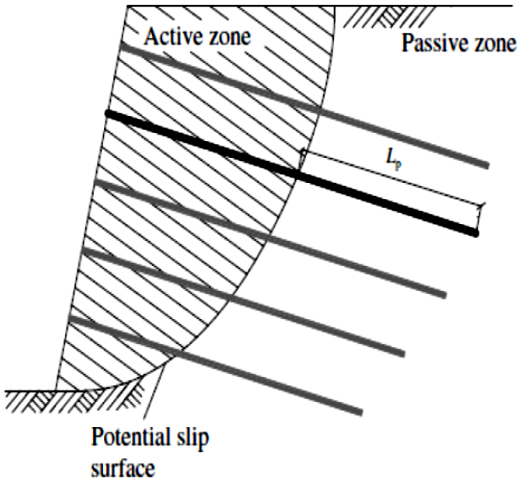


Fig1.(b)

Fig. 1 Comparison between passive piles and earth reinforcement in slope stabilization ─ (a) Passive piles single row arrangement; (b) Earth reinforcement using steel rods (after Lee et al., 1995).

reinforcing effect is developed by the pullout resistance generated between the embedded steel bars and the surrounding soil. Fig. 1(b) illustrates the typical reinforced slope with steel reinforcing rods. Though previous studies clearly explained the prevention mechanism of large diameter passive piles in single row however this research aims to pave a way into looking at the potential of multiple rows arrangement of SDSP that combines both linear and planar countermeasures in landslide prevention and also try to describe their fundamental deformation mechanism using a 2D numerical model developed in PLAXIS 8.2 finite element analysis.

2. Brief Literature Review on Selected Landslide Issues in Malaysia

With the increased developments that have encroached into the hilly areas over the past two decades, Malaysia experiences frequent landslides with a number of major slope failures which cause damage and inconvenience to the public. These landslides include newly completed slopes, such as the recent failure at Putrajaya in 2007 as well as old slopes, such as the collapse of the rock slope of the PLUS Expressway at Bukit Lanjan (2003), which was completed more than ten years ago. Some of these landslides have claimed lives for example, the notorious collapse of Tower 1 apartment of Highland Towers which claimed 48 lives in 1993 (Gue and Cheah, 2008). Climate conditions in Malaysia are characterized by relatively uniform temperature and pressure, high humidity and particularly abundant rainfall with annual rainfall intensity over 2500mm. Most of the landslides in two monsoon seasons of Malaysia are induced by the high rainfalls and more than 80% of landslides were caused by man-made factors, mainly design and construction errors (Gue and Tan 2006).

According to Jamaluddin (2006), results of extensive studies on many cases of slope failures in Malaysia indicate that the slope failures are mostly attributed to human factors such as negligence, incompetence, lack or poor maintenance system, ignorance of geological inputs, unethical practice and various negative human attitudes. This is also supported in the paper by Gue and Tan (2006) where the authors have similar findings in their respective investigation cases on slope failures. The authors reported that 88% of the 49 cases of slope failures in Malaysia investigated are man-made slope failures due to either design errors or construction errors. These errors are mainly due to the lackadaisical human attitudes. Their study revealed that man-made slope failures are due to either design errors or construction errors. The authors also mentioned that only a small percentage of slope failures investigated in Malaysia are caused by geological features. It is a well-known fact that in a tropical climate with a continuous heavy and prolonged rainfall during the two monsoons in a year, slope failures in Malaysia are not uncommon. As such, the effect of expected intense rainfall on the slope stability should have been taken into account in the slope design. Despite that, there are yet many reported slope failure cases, particularly man-made slope failures in Malaysia. Table 1 shows the summary of the causes of the major landslide events in Malaysia. According to the National Slope Master Plan (NSMP, 2008) published by the Public Work Department (PWD) Malaysia, with specific reference to Malaysia, the causes of landslides can be summarized as shown in Table 2. In any case, human causes (design and construction errors) can be prevented provided that precautionary measures are carried out with due diligence. The risk of landslides can be mitigated with proper assessment of the effect of ground conditions and foreseeable construction activities to the surrounding slopes during design and construction

Table 1. Causes of the major landslide events in Malaysia (after Gue and Tan, 2006)

Date	**Location**	**Main causes**	**Slope type**
11th Dec 1993	Highland Towers	- inadequate design - improper construction - triggered by rainfall	Man-made slope
30th June 1995	Genting Sempah (debris flow)	- trigerred by heavy rainfall	Natural slope
6th Jan 1996	Gunung Tempurung	- adverse geological features - trigerred by rainfall	Man-made slope
30th Aug 1996	Pos Dipang (debris flow)	- inadequate FOS - trigerred by rainfall	Natural slope
28th Nov 1998	Paya Terubung (rockslide)	- inappropriate design - trigerred by rainfall	Man-made slope
7th Feb 1999	Sandakan (lanslide)	- inadequate FOS - trigerred by rainfall	Natural slope
15th May 1999	Bukit Antarabangsa (landslide)	- inadequate design - improper construction - trigerred by prolonged rainfall	Man-made slope
28th Jan 2002	Ruan Changkul (landslide)	- trigerred by rainfall	Man-made slope
20th Nov 2002	Taman Hillview (landslide)	- inadequate design of the adjacent slope - trigerred by rainfall - old landslide location	Man-made slope
26th Nov 2003	Bukit Lanjan (rockslide)	- adverse geological condition - long term weathering - prolonged rainfall	Man-made slope

*FOS = Factor of Safety

Table 2. Common causes of landslide in Malaysia (after NSMP, 2008)

Common causes of landslide in Malaysia
Abuse of prescriptive methods
Inadequate study of past failures
Design errors (including insufficient site-specific ground investigation)
Lack of understanding on testing and care
Lack of maintenance
Lack of appreciation of water
Underestimating existing groundwater table
Inadequate capacity of surface drainage
Construction errors
Combination of the above

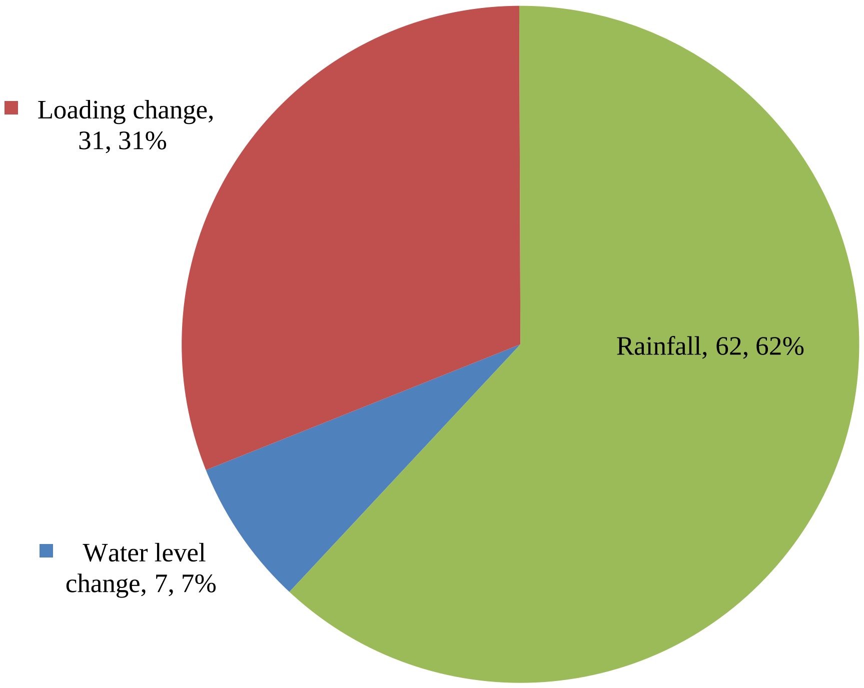


Fig. 2 Landslide triggering factors based on selective Malaysian case history (after NSMP, 2008).

stages. Therefore, appropriate design of slope strengthening works shall then be carried out based on the stability assessment. The NSMP report also suggested that the most common landslide triggering factors can be summarized as in Fig. 2.

3. Small Diameter Steel Piles (SDSP)

Remediation of slope failures requires stabilization alternatives that address causes of slope instability. Slope reinforcement using pile stabilization practices can be an effective method of remediation in preventing slope movements in weak soils where enhanced drainage does not provide adequate stability. Soil load transfer to pile elements from the downslope soil movement, as occurs in the slope failures, is a complex soil-structure interaction problem. The significant differences in existing design procedures of pile stabilization suggest that the stabilizing mechanisms are not fully understood. The downslope soil movement of slope failures induces unique, unknown lateral load distributions along stabilizing piles. The reliable estimation of these load distributions is important, because the influence of piles on the global stability of the slope depends directly on the pile loading condition (Thompson and White, 2006). SDSP model was simulated based on the actual SDSP currently available in practice. The actual to the modeling ratio of the pile size used in the present study is tabulated in Table 3. Since the adoption of the SDSP is practically new to the real practice of landslide prevention, there have been limited sources of references available. However, earlier descriptions of the use of steel piles as slopes reinforcing agents particularly used in Japan, for both natural and cut-slopes countermeasures have been discussed in details by (Ito and Matsui, 1975; Takano et al., 1995; Cai and Ugai, 2003; Shimaoka et al., 2003). They reported that among the novelties of using steel piles are due to their aseismicity, environmental friendly materials and methods, labor-saving at construction sites and further reductions in construction costs as compared to the presently adopted bored in-situ concrete piles. Hence, it is timely to adopt such method in Malaysia to address the issues of lateral earth movement induced by landslides without compromising the concept of sustainability, which the proposed method has promised. The main reason for considering the application of this particular type of reinforcement for slope stabilization lies within the unique characteristics of SDSP piles which act not only as reinforcement but also protection to soil. The usage of SDSP offers great number of advantages which include high tensile strength, resilient durability and non-biodegradable properties of the parent material (i.e. steel). On the other hand, the disadvantages of SDSP as compared to conventional reinforcement include high cost in terms of construction, maintenance and repair. However, slope stabilizing technique by means of SDSP has been applied in increasing quantities in European countries such as in the United States of America, France, Germany and also in the Asian region in countries like Japan and Hong Kong indicating its positive prospect.

Table 3. Actual test to model test ratio

Parameter	Modeling in PLAXIS	Actual laboratory test
Pile size	3 mm - 10 mm	90 mm - 300 mm
Ratio	1	30

4. Mathematical Formulation

Mathematical model can be defined as the combination of dependent and independent variables and relative parameters in the form of a set of differential equations which defines and governs the physical phenomenon (S. Chakraborty et al., 2012). The sliding soil mass above the failure surface is assumed to be reinforced by the placed rows of piles that resist soil movements and transfer loads to the more stable underlying layers. Fig. 3 shows a passive pile subjected to lateral soil movement, where the soil mass is divided into an unstable layer (the passive pile portion) and a stable layer (the active pile portion) (Chen, 1994).

4.1 Governing Equations

In their study, Jeong et al., (2003) has introduced a model to compute load and deformations of piles subjected to lateral soil movement based on the transfer function approach. The problem is decomposed into two components. First, the pressure-displacement (P–δ) curves induced in the substratum are determined either from measured test data or from finite element analysis. Second, a coupled set of P–δ curves is used as input to study the behavior of the piles which can be modeled as a beam resting on non-linear soil spring supports. Simple numerical solution procedures are developed for fairly general conditions (non-linear stress-strain behavior at the pile-soil interface and non-homogeneous soil conditions). The governing equations for the pile deflection, (w) can be expressed in separate forms for the pile segments along its z axis at node, (i) above (Eqn. 1) and below (Eqn. 2) the interface (Fig. 4). Also, the pressure, (q) acting on the model wall during the analysis is calculated based on Eqn. 3.

$$EI\left(\frac{d^4 w}{dz^4}\right)_i = p = K_i\left[\left(y_s\right)_i - w_i\right] = K_i\delta_i \tag{1}$$

$$EI\left(\frac{d^4 w}{dz^4}\right)_i + K_i w_i = 0 \tag{2}$$

$$q = A_c\left[\frac{1}{N_\phi \tan\phi}\left\{\exp\left(\frac{D_1 - D_2}{D_2} N_\phi \tan\phi \tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right) - 2N_\phi^{1/2}\tan\phi - 1\right\} + \frac{2\tan\phi + 2N_\phi^{1/2} + N_\phi^{-(1/2)}}{N_\phi^{1/2}\tan\phi + N_\phi - 1}\right]$$
$$-c\left(D_1\frac{2\tan\phi + 2N_\phi^{1/2} + N_\phi^{-(1/2)}}{N_\phi^{1/2}\tan\phi + N_\phi - 1} - 2D_2 N_\phi^{-(1/2)}\right) + \frac{\gamma\bar{z}}{N_\phi}\left\{A\exp\left(\frac{D_1 - D_2}{D_2} N_\phi \tan\phi \tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right) - D_2\right\} \tag{3}$$

Where,

w = lateral pile displacement
y_s = free-field soil movement at each depth before pile installation
Ki = elastic constant of soil (Es)
EI = flexural rigidity of the pile
δ_i = relative displacement between y_s and w
q = pressure acting on the model wall
A_c = cross-sectional area of the piles
N_ϕ = bearing capacity factor
ϕ = internal friction angle
D_1 = center to center distance between piles
D_2 = clear distance between piles
c = cohesion value of the soil
γ = unit weight of the soil
$\bar{z}$ = depth along the pile measured from ground surface

4.2 Safety Factor of the Stabilized Slope

The slope-pile stabilization scheme analyzed in this study is shown in the Fig. 5. The conventional Bishop simplified method is employed to determine the critical circular sliding surface, resisting moment M_R and overturning moment M_D. The resisting moment generated by the pile is then obtained from the pile shear force and bending moment developed in the pile at the depth of the sliding surface analyzed. It is assumed that the lateral soil movement exerted by the sliding slope on the pile results in the mobilization of shear forces and bending moment. Thus, the safety factor of the reinforced slope with respect to circular sliding is calculated as shown in Eqn. 4.

$$\begin{aligned} F &= F_i + \Delta F \\ &= \frac{M_R}{M_D} + \frac{V_{cr}.R.\cos\theta - M_{cr} + V_{head}.Y_{head}}{M_D} \end{aligned} \tag{4}$$

Where,

Fi = safety factor of unstabilized slope

ΔF = increased safety factor of slope reinforced with pile

Mcr = bending moment at critical surface

Vcr = shear force at critical surface

V_{head}= shear force at pile head.

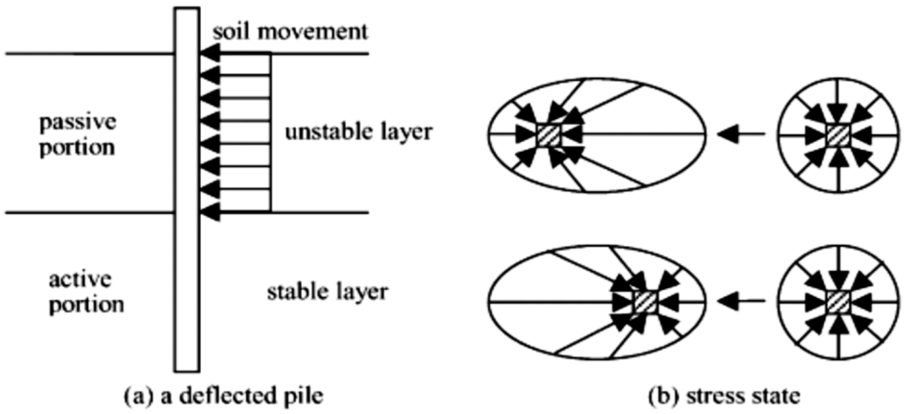


Fig. 3 A pile undergoing lateral soil movement ─ (a) A deflected pile; (b) Stress state (after Jeong et al., 2003).

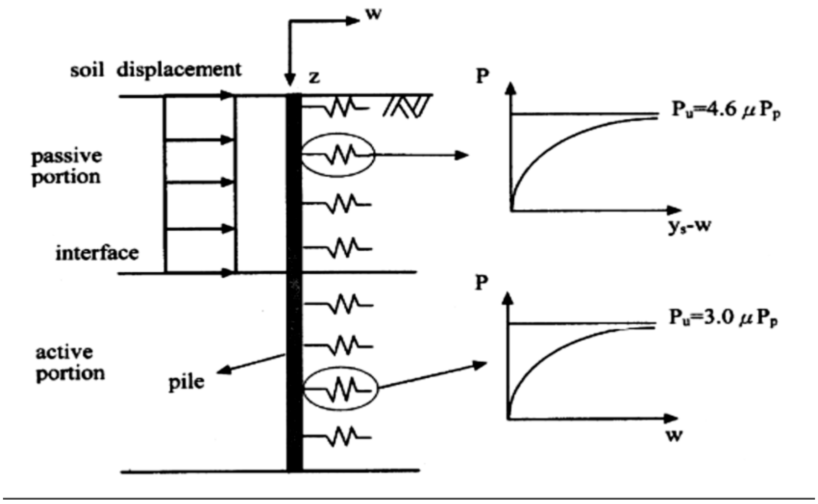


Fig. 4 A pile subjected to lateral soil displacement (after Jeong et al., 2003).

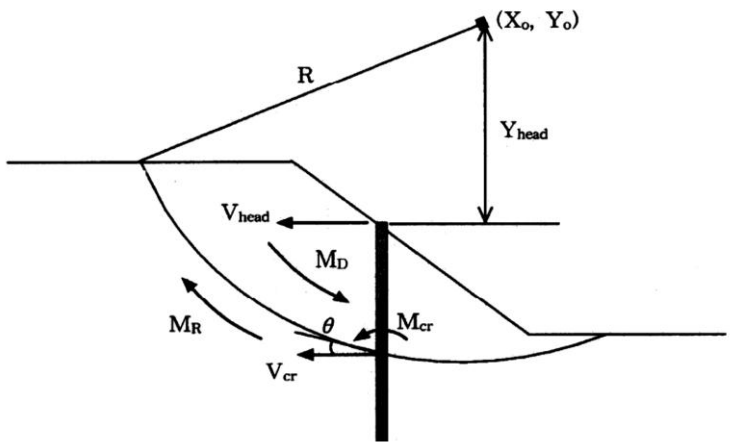


Fig. 5 Forces on stabilizing piles and slope (after Jeong et al., 2003)

4.3 Microcomputer Program

A microcomputer based computer program has been developed using uncoupled formulation to analyze the pile -slope stability problem as described in Sections 4.1 and 4.2 as depicted in Fig. 6

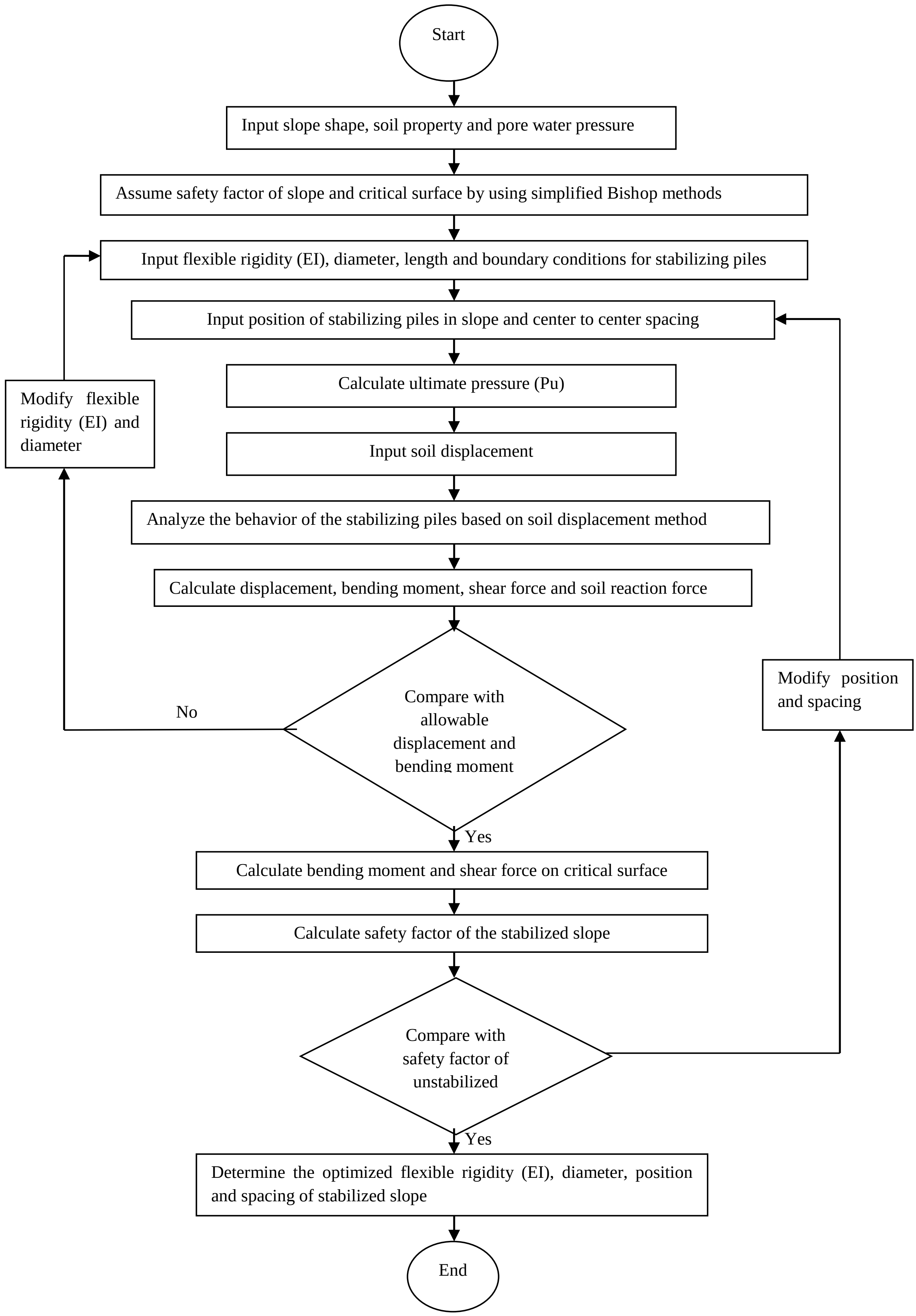


Fig. 6 Flow chart of the computer analysis.

5. Numerical Simulation and Parametric Study

5.1 PLAXIS 8. 2 (2D) Finite Element Analysis Package

PLAXIS Version 8.2 is a finite element package intended for the two-dimensional (2D) analysis of deformation and stability in geotechnical engineering. Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time-dependent and anisotropic behavior of soils/rocks. In addition, since soil is multiphase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in soil. Although the modeling of the soil itself is an important issue, structural modeling and the interaction between the soil and the structures concerned is of vital significance as well. PLAXIS is equipped with features to deal with the various aspects of complex geotechnical structures. In this research, Mohr-Coulomb's elastic-perfectly plastic soil model was employed in which five parameters namely Young's modulus (E), Poisson's ratio (ν), the cohesion (c), the friction angle (ϕ) and the dilatancy angle (ψ) were considered.

5.2 Boundary Conditions

By selecting the standard fixities from the menu, PLAXIS automatically imposes a set of general boundary conditions to the geometry model. These boundary conditions are generated based on the following rules:

- Vertical geometry lines for which the x-coordinate is equal to the lowest or the highest x-coordinate in the model obtain a horizontal fixity (Ux=0).
- Horizontal geometry lines for which the y-coordinate is equal to the lowest or the highest y-coordinate in the model obtain a full fixity (Ux=Uy=0).
- Plates that extend to the boundary of the geometry model obtain a fixed rotation in the point at the boundary (ϕ_z=0) if at least one of the displacement directions of that point is fixed.

5.3 Model Soil Properties

The model soil properties adopted in the numerical analysis is tabulated in Table 4. As can be seen from the table, the Mohr-Coulomb's elastic-perfectly plastic soil model which requires five parameters namely Young's modulus (E), Poisson's ratio (ν), the cohesion (c), the friction angle (ϕ) and the dilatancy angle (ψ) were used.

Table 4. Soil model properties

Parameter	Name	Sand	Unit
Material model	Model	Mohr-Coulomb	–
Material behavior	Type	Drained	–
Soil behavior above phreatic level	γdry	16 – 17	kN/m^3
Horizontal permeability	Kx	1	m/day
Vertical permeability	Ky	1	m/day
Young's modulus	E	30000 – 80000	kPa
Poisson's ratio	ν	0.3	–
Cohesion	C	1.4	
Friction angle	ϕ	30 – 34	
Dilatancy angle	ψ	0.4	kPa
Interface strength ratio	R	0.6 – 1.0	
Global coarseness	Coarseness	loose – dense	

5.4 Model SDSP Properties

In order to model the SDSP which acts as reinforcement for landslide countermeasure, the available plate material model was applied to which its properties are shown in Table 5.

Table 5. SDSP model properties

Parameter	Name	Pile	Unit
Material model	Model	Plate	–
Material behavior	Type	Elastic	–
Normal stiffness	EA	1.85 x 10^9	kN/m
Flexural rigidity	EI	1.4 x 10^5	kNm^2/m
Equivalent thickness	d	0.03	m
Weight	w	0.35	kN/m/m
Poisson's ratio	ν	0.15	–

5.5 Model Wall Properties

The model wall which acts as a barrier to counter the front displacement of the soil mass during landslide was modeled using the plate material model to which its properties are shown in Table 6.

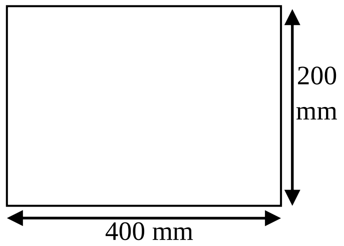
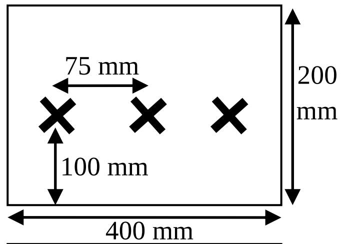
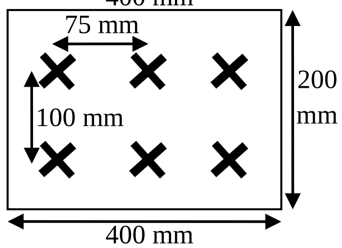
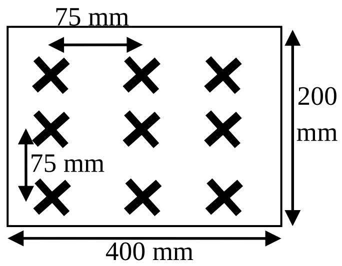
Table 6. Wall model properties

Parameter	Name	Wall	Unit
Material model	Model	Plate	–
Material behavior	Type	Elastic	–
Normal stiffness	EA	1.96 x 10^9	kN/m
Flexural rigidity	EI	1.25 x 10^5	kNm^2/m
Equivalent thickness	d	0.03	m
Weight	w	0.25	kN/m/m
Poisson's ratio	ν	0.15	–

5.6 Parametric Study

To examine the most effective means of using piles for stabilizing slopes, a series of numerical analyses on stabilizing piles were performed based on the major influencing parameters intended for this study such as the effect of the multirow arrangement of the proposed SDSP (single row, double row and triple row arrangements) as shown in Table 7. Furthermore, the effect of ground relative densities (loose ground condition, Dr=30% and dense ground condition, Dr=80%) is also considered as depicted in Fig. 7(a) and Fig. 7(b) respectively.

Table 7. Summary of the SDSP model arrangements

Arrangement type	No. of piles	Notations	Layout
Unreinforced	0	NIL	
Single row	3	3C x 1R	
Double row	6	6C x 2R	
Triple row	9	9C x 3R	

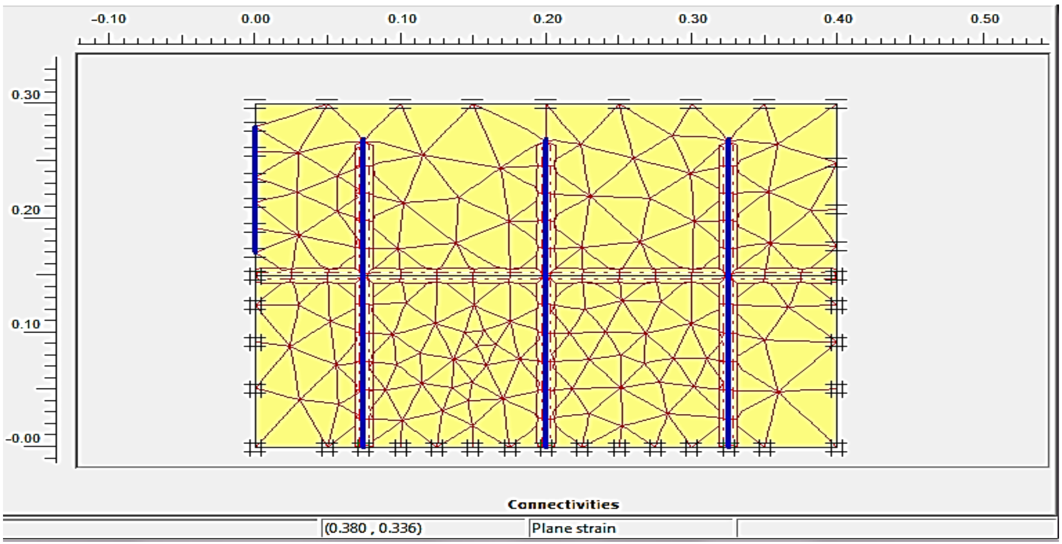
C = No. of pile columns R = No. of rows of the arranged pil ✕ = denotes □ 10 mm x 10 mm square and ○ 3 mm circular aluminium bars

6. Results and Discussion

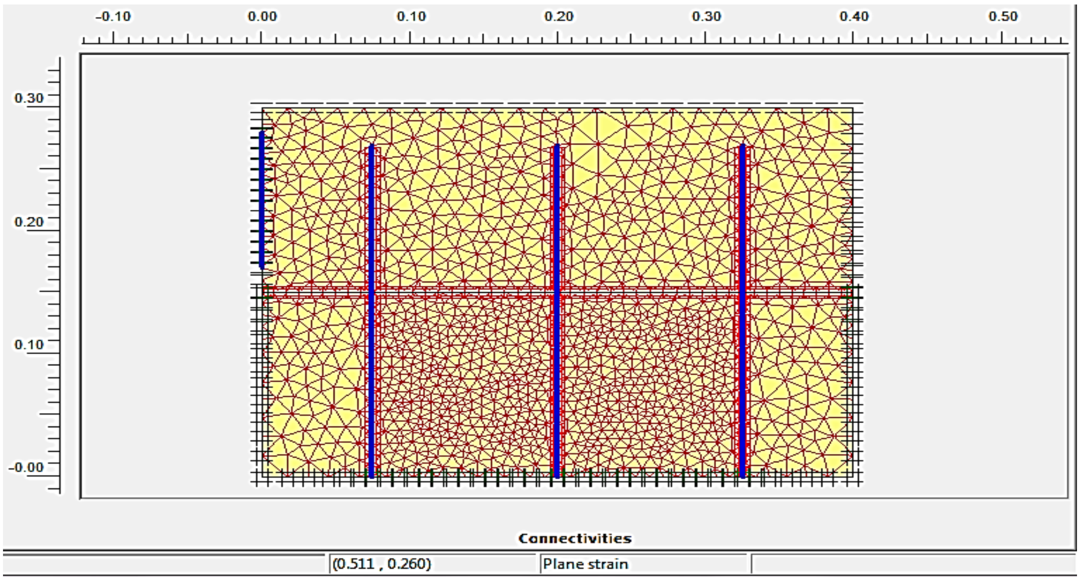
Hereafter, the results from both the FEA and also the rigorous mathematical approach are compared based on the pile deflection as a result of the soil mass displacement, bending moment profiling of SDSP and last but not least, the earth pressure acting on the model wall. Hereby, the parametric study i.e. the effect of the multirow arrangement of SDSP and also the effect of the relative ground densities have simultaneously taken into consideration in the presented results for better outcome interpretation

6.1 Pile Deflection due to Soil Mass Displacement

Fig. 8 portrays the deflection behavior of SDSP. Deflection of both circular and square piles in loose ground was observed to be dependent of the EI of the reinforcing material. No apparent correlation between pile shape and ground condition was found in dense ground since all piles were displaced in the range of 0.2 mm – 3 mm



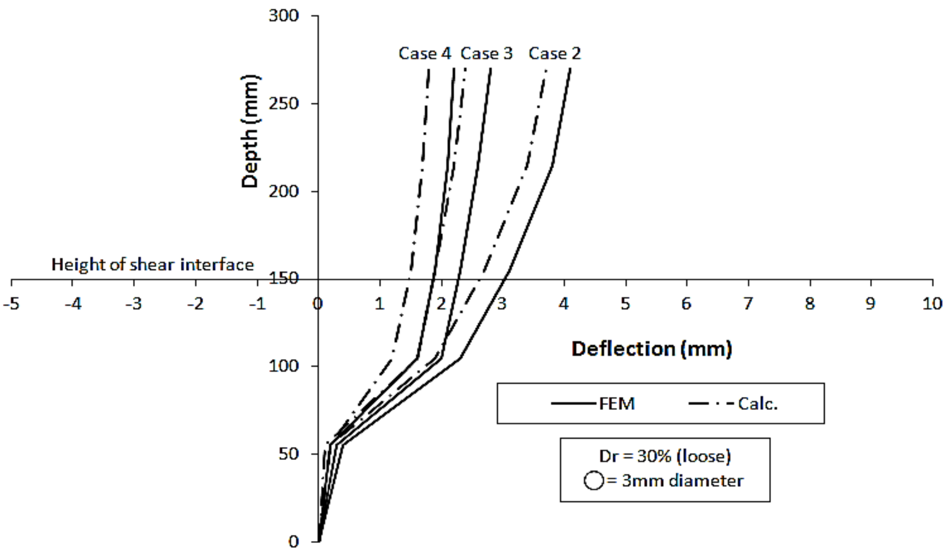
(a)



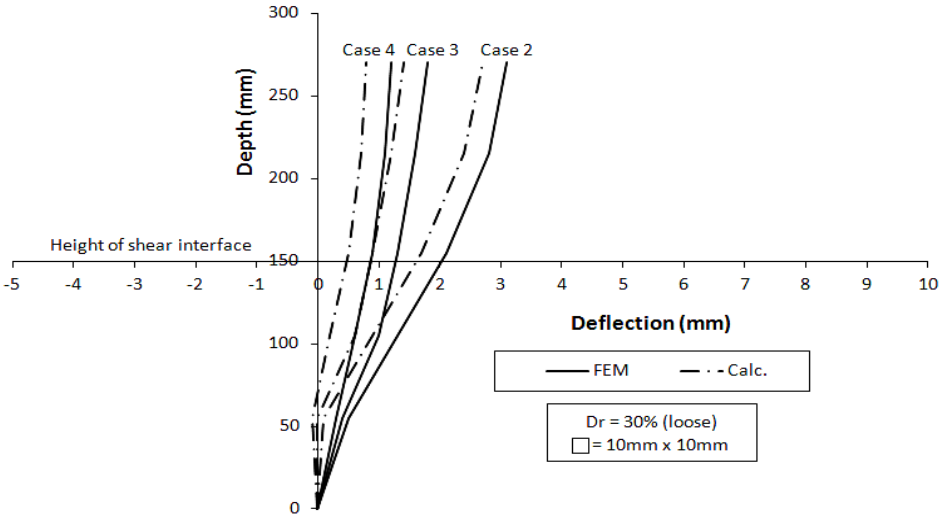
(b)

Fig. 7 Sample of mesh generations for the FEA ─ (a) Mesh generation for Dr = 30% soil model; (b) Mesh generation for Dr = 80% soil model.

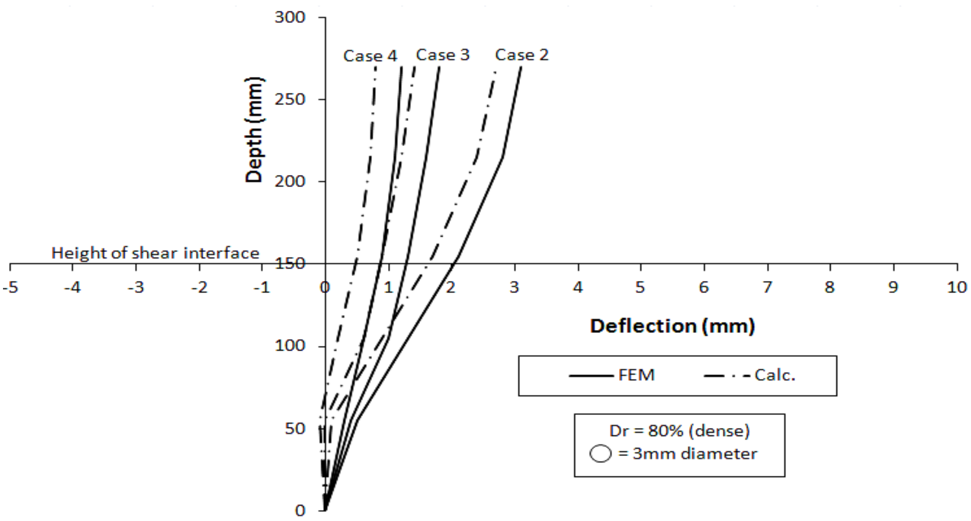
due to the confining effect of the densely compacted soil. It was also observed that the changes in ground densities had significantly influenced soil's dilatancy. The variation of the normal stress distribution depicted in Fig. 9, as a result of dilative and contractive sand behavior, contributes to the lower pile resistance in loose ground condition regardless of the piles arrangement. This explains the negative normal stress and deflection in loose ground especially under the height of shear interface. Highest deflection values are recorded in loose ground condition (Dr=30%) as loose ground lacks particles interlocking. Slight deflection was observed at the end tip of the piles even though the piles were fixed was an evidence of mass soil's particle movement due to its loose condition. Calculation prediction is shown to have a good agreement in all cases however FEA was overestimated for all cases. Deflection of both square and circular piles in loose ground condition was observed to be dependent on the piles shape and bending stiffness as stiffer material tends to exhibit higher plasticity. Smaller deflection values were observed in a dense ground condition as compared to the loose condition. The pile end tips exhibit no apparent deflection due to the confining effect of the surrounding soil that prevented free movements. Likewise, calculation prediction is shown to have a good agreement in all cases however, FEA was overestimated in all cases. Deflection of both square and circular piles in loose ground condition was observed to be independent on the piles shape, size and bending stiffness. It was observed that for both cases, the piles were all displaced in the range of 0.2 mm – 3 mm.


Case 4 Case 3 Case 2
Depth (mm)
Height of shear interface
Deflection (mm)
FEM
Calc.
Dr = 30% (loose)
= 3mm diameter

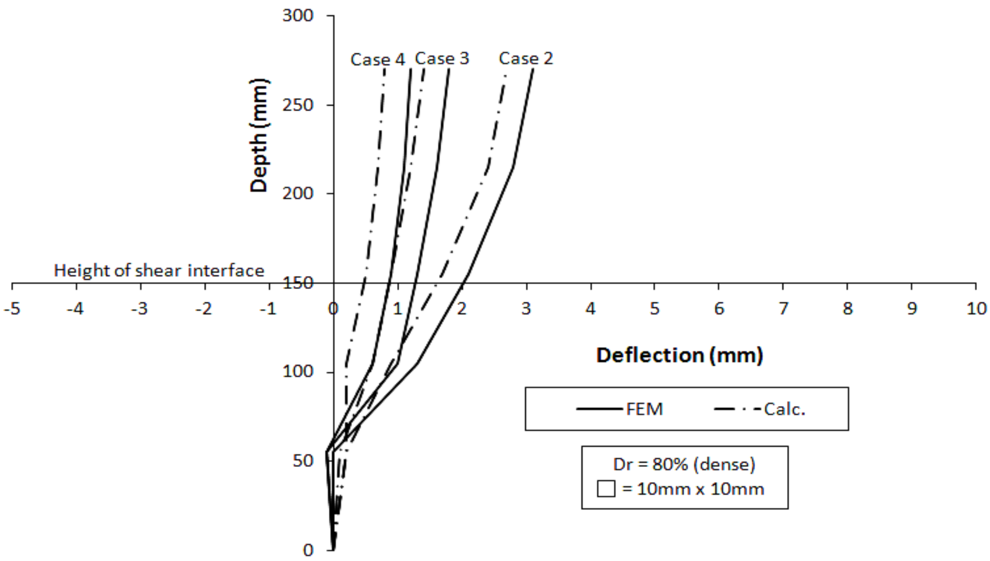
(a)


Case 4 Case 3 Case 2
Depth (mm)
Height of shear interface
Deflection (mm)
FEM
Calc.
Dr = 30% (loose)
= 10mm x 10mm

(b)


Case 4 Case 3 Case 2
Depth (mm)
Height of shear interface
Deflection (mm)
FEM
Calc.
Dr = 80% (dense)
= 3mm diameter

(c)



(d)

Fig. 8 Comparison of pile deflection relationship — (a) Circular piles 3 mm diameter in Dr=30% (loose ground); (b) Square piles 10 mm x 10 mm in Dr=30% (loose ground); (c) Circular piles 3 mm diameter in Dr=80% (dense ground); (d) Square piles 10 mm x 10 mm in Dr=80% (dense ground).

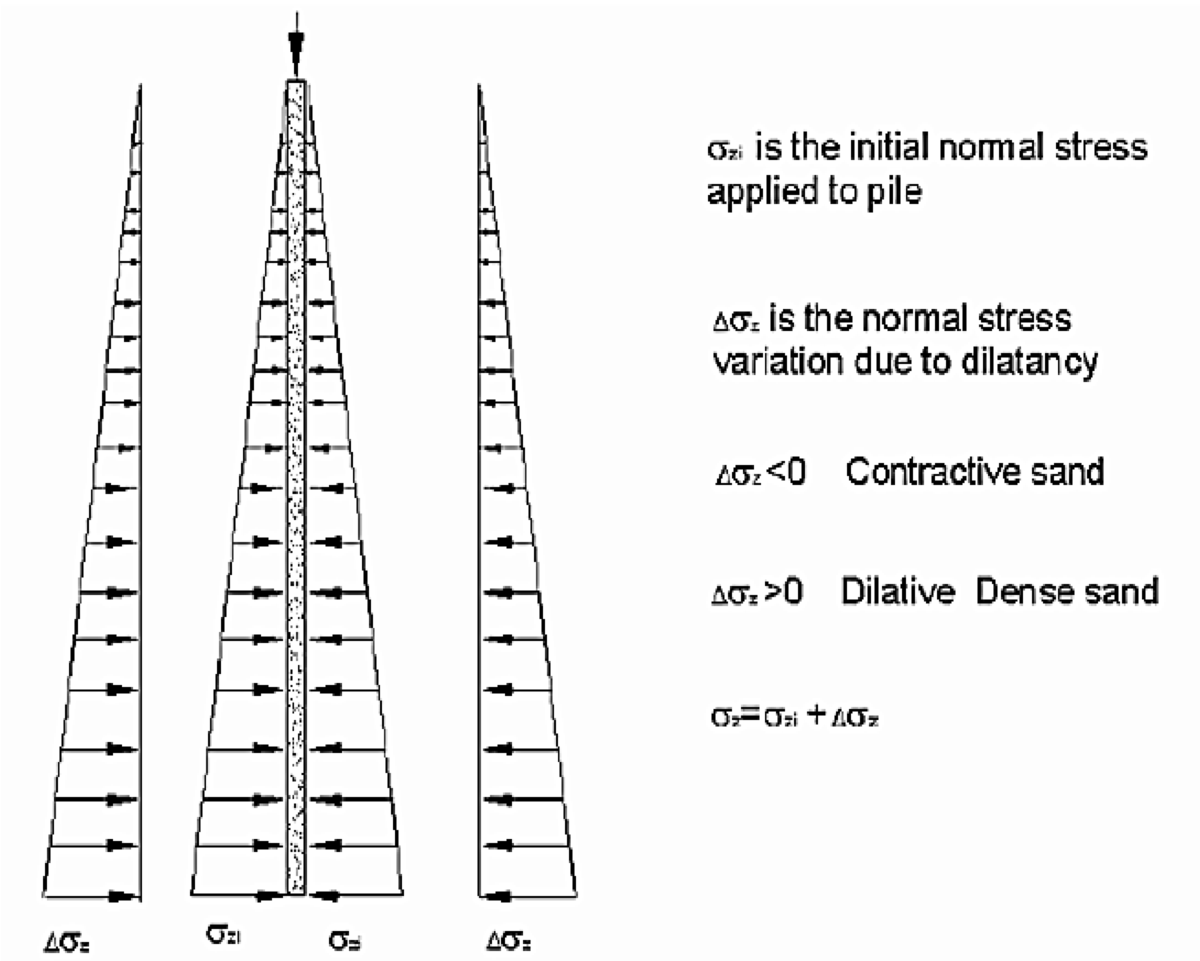
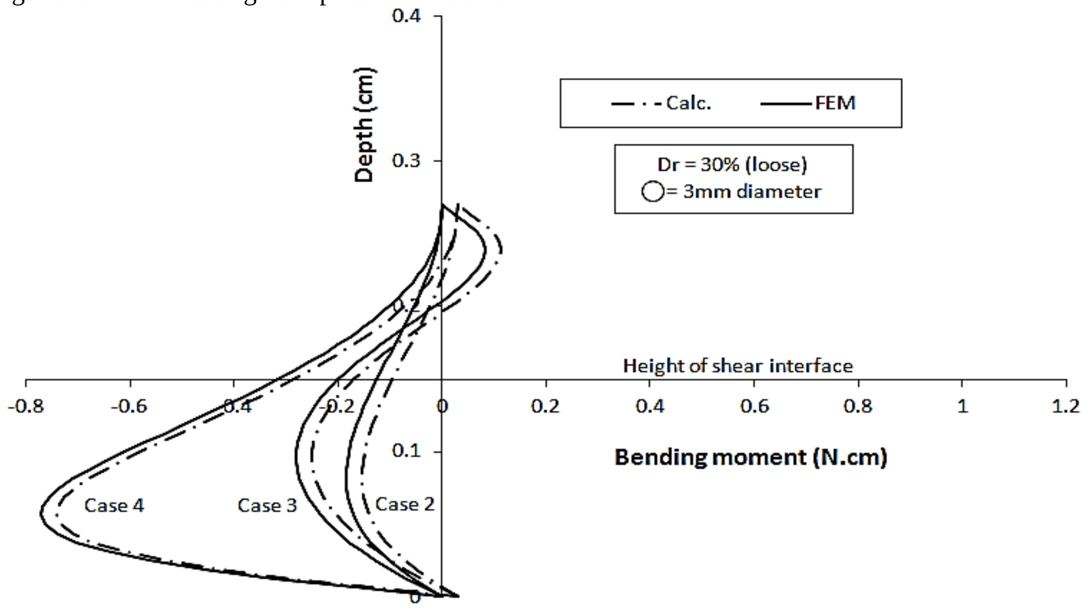


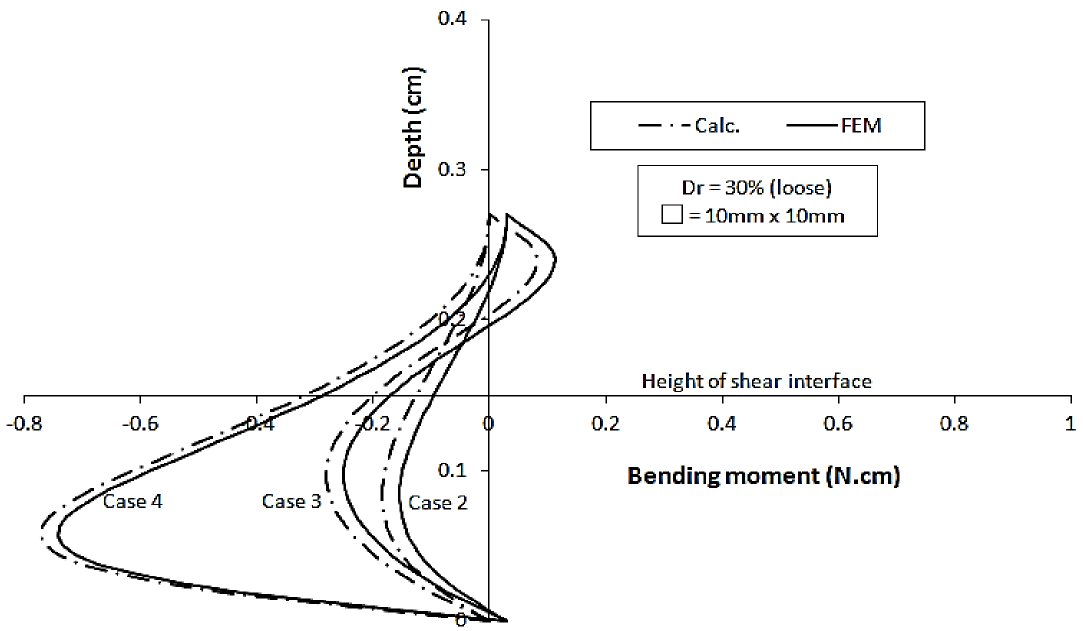
Fig. 9 Variation of normal stress distribution.

6.2 Bending Moment Profiling

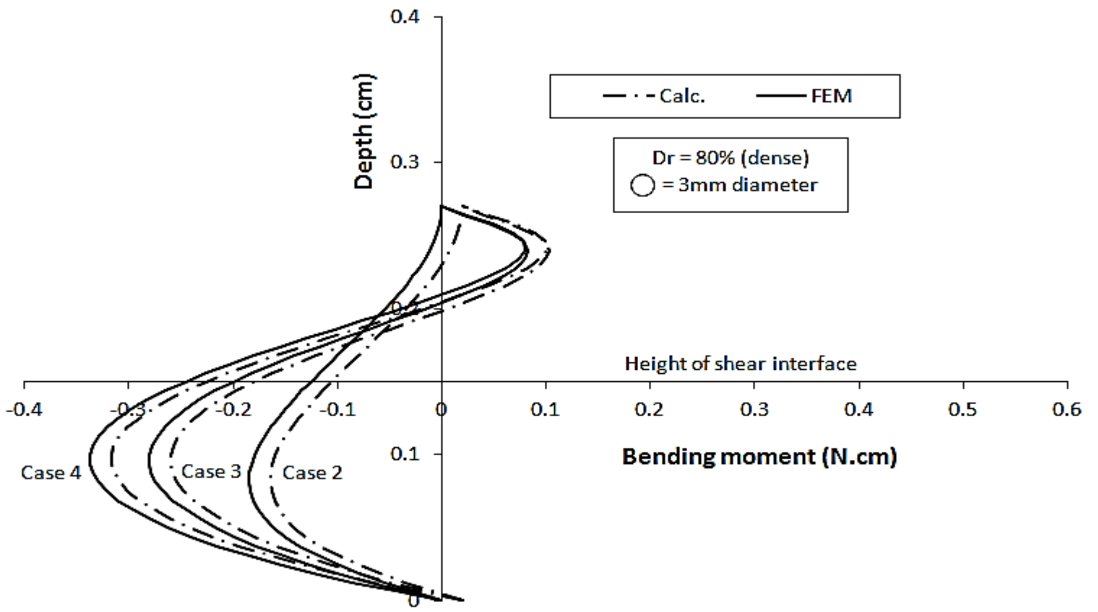
Bending moments that appeared in the vicinity of the pile toes at the lower part of the shear interface as shown in Fig. 10 are expected because no rotation in both X and Y planes is allowed (fixed boundary condition). The large bending moments generated at pile toes can be minimized by considering appropriate piles spacing and designated embedded length into the potential slip surface. Higher values of bending moments are observed in loose ground condition due to greater piles' deflection. The results show that both calculation and FEA results are in good agreement. However, FEA tends to overestimate circular pile arrangement than rectangular piles due to the assumption made in the model based on 2D plane strain in which circular piles are assumed to be square in shape thus, affecting the results. Since no rotation is allowed at the toe of the piles, a relatively large value of bending moments are expected at the lower part of the shear interface. As compared to loose ground condition, the values of bending moments are smaller in dense ground due to the confining effect of the interlocking soil. Similarly, FEA tends to overestimate circular pile arrangement than square piles due to the same reason as discussed previously. Large amount of bending moment values are also observed in the pile toe as a result of piles' deflection.



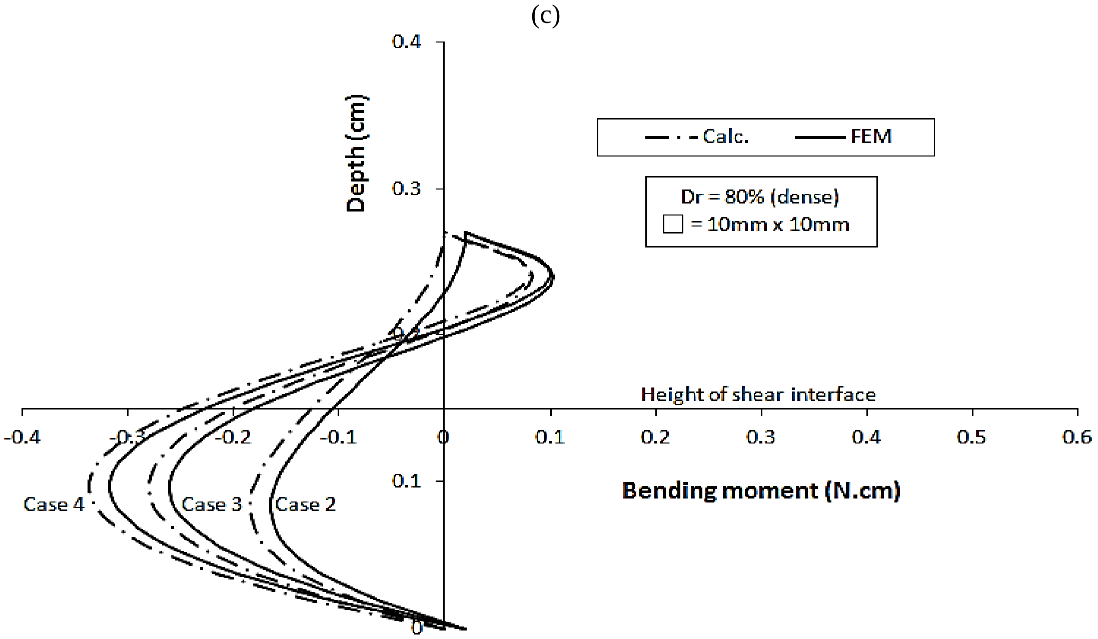
(a)



(b)



(c)

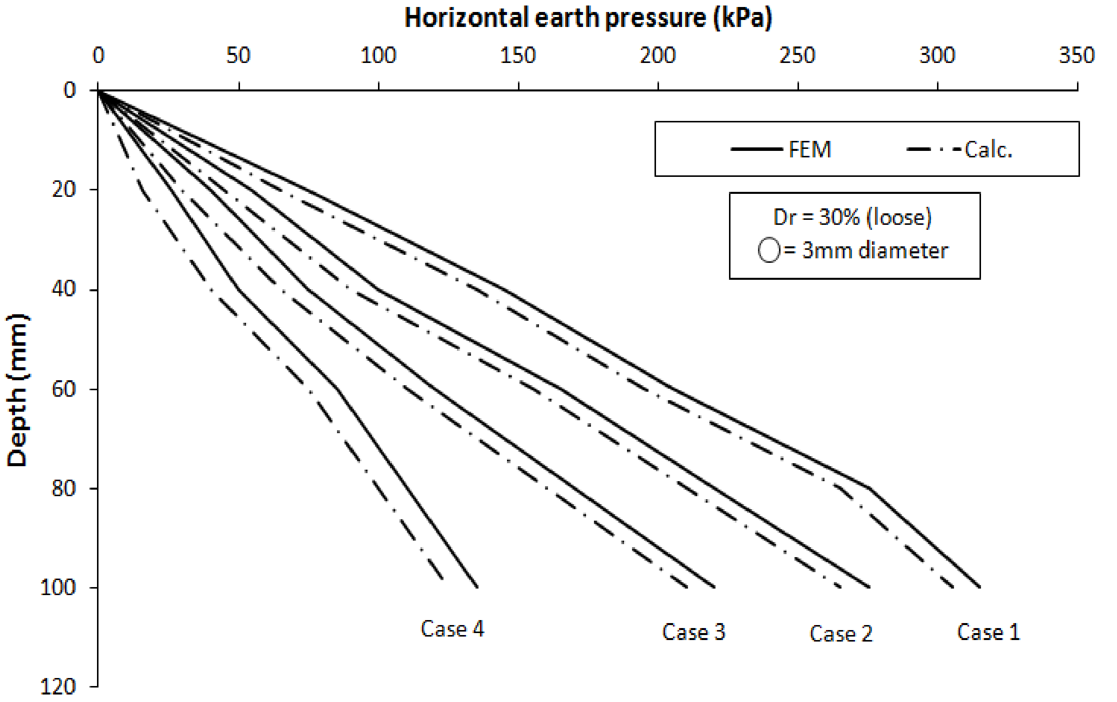


(d)

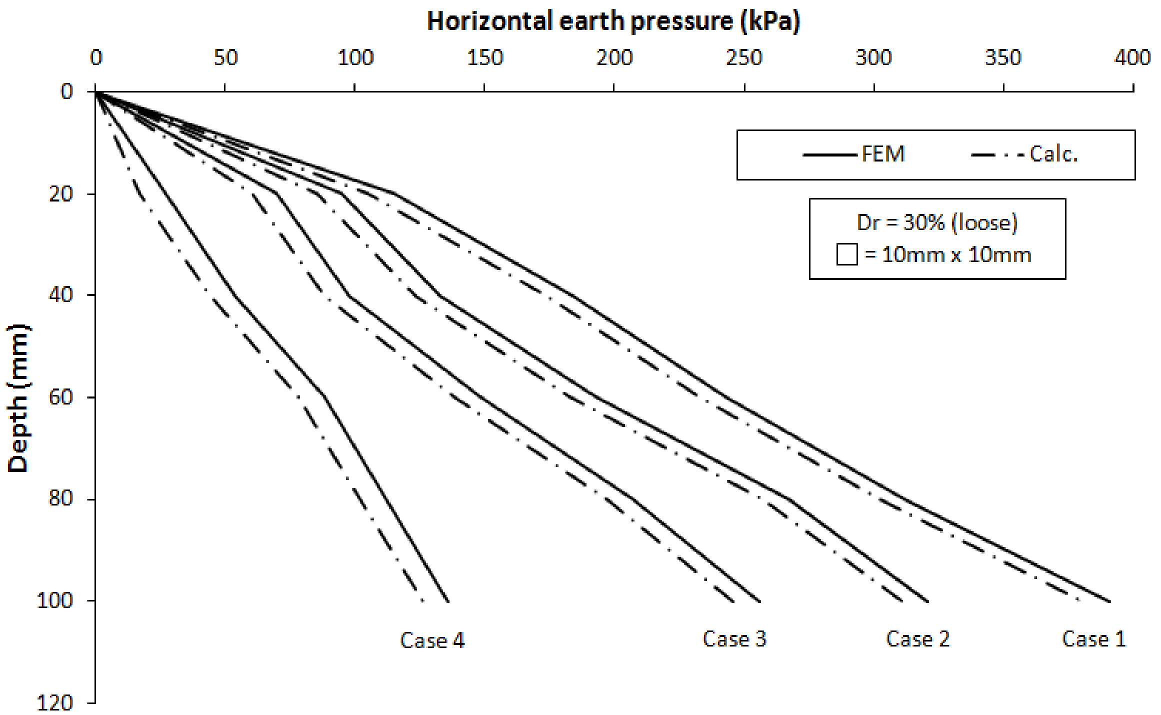
Fig. 10 Comparison of bending moment relationship – (a) Circular piles 3 mm diameter in Dr=30% (loose ground); (b) Square piles 10 mm x 10 mm in Dr=30% (loose ground); (c) Circular piles 3 mm diameter in Dr=80% (dense ground); (d) Square piles 10 mm x 10 mm in Dr=80% (dense ground).

6.3 Variation of Earth Pressure acting on the Model Wall

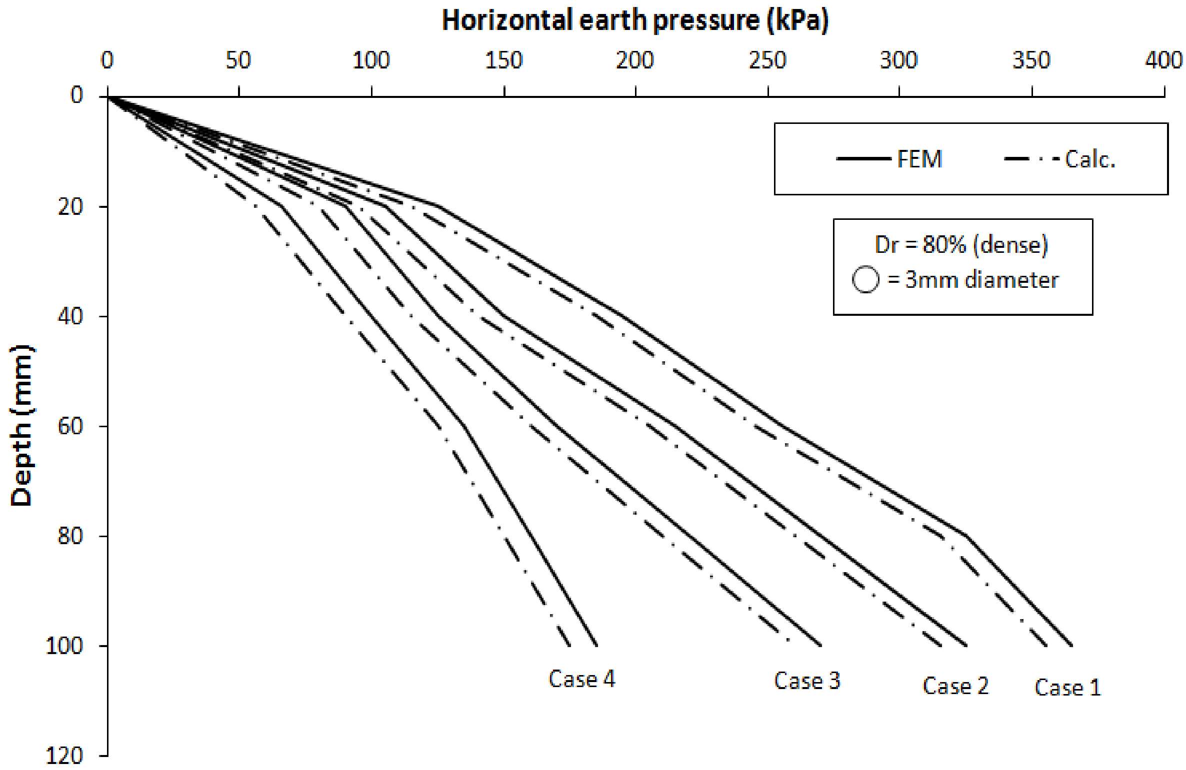
Horizontal earth pressure was obtained from the impact of soil lateral movement with the front side of the model wall. The variation of the earth pressure acting on the model wall is portrayed in Fig. 11. Less pressure is exerted to the unreinforced condition (Case 1) in both ground conditions because with the presence of the multiple arrangements of piles, the soil is able to absorb higher loading while full stress transfer is fully optimized. Bending stiffness (EI) of the reinforcing material was observed to be crucial in determining prevention mechanism in loose ground. All calculation procedures are shown to have a good agreement however, FEA results seemed to be overestimated. In loose ground condition, horizontal earth pressure was obtained from the soil lateral movement on the front side of the model wall. Calculation prediction is shown to have a good agreement in all results however, FEM analysis was overestimated for all cases. Reinforced square piles (10 mm x 10 mm) provided greater wall's resistance capacity as compared to the circular piles (3 mm dia.) due to their greater shape and material's bending stiffness (EI). Similarly, in dense ground condition, horizontal earth pressure was obtained from the soil lateral movement on the front side of the model wall. Calculation prediction is shown to have a good agreement for all results however, FEM analysis was overestimated for in all cases. It was obeserved that greater amount of earth pressure is yielded in a denser ground condition (Dr=80%) and material's bending stiffness (EI). In order to show that the soil is indeed experiencing passive pressure, the minimum passive pressures (Pp) are showed in all ground density cases.



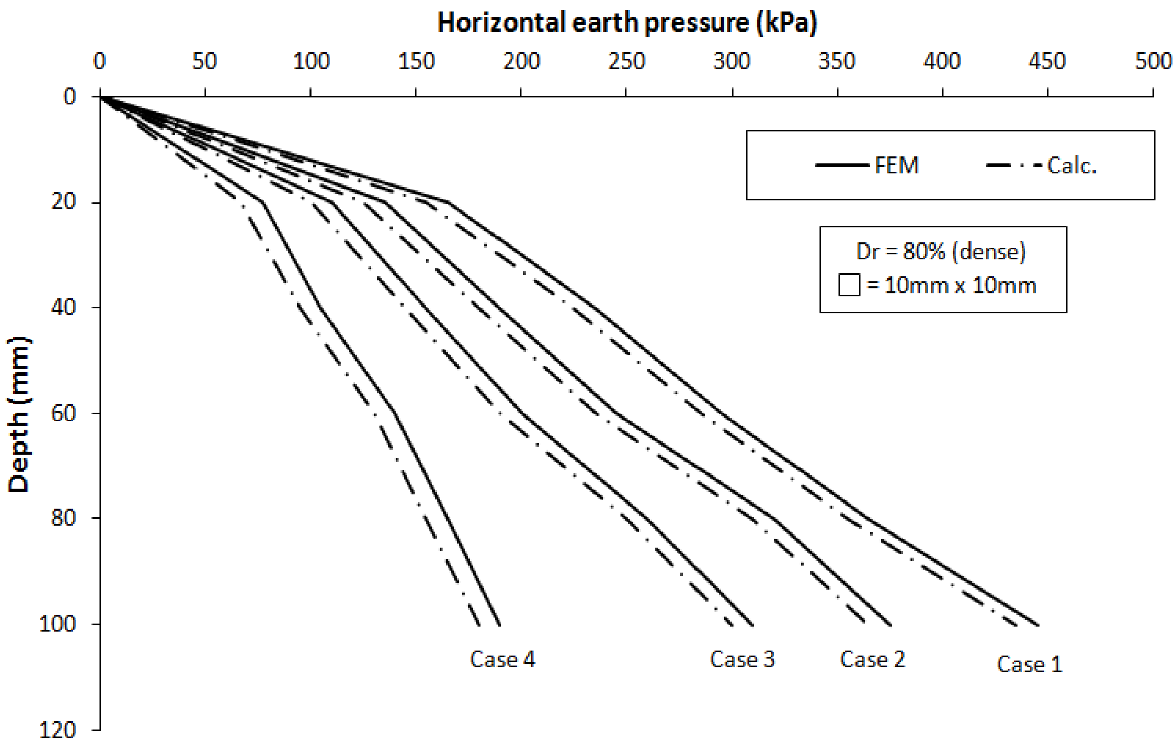
(a)



(b)



(c)



(d)

Fig. 11 Comparison of earth pressure relationship acting on the model wall — (a) Circular piles 3 mm diameter in Dr=30% (loose ground); (b) Square piles 10 mm x 10 mm in Dr=30% (loose ground); (c) Circular piles 3 mm diameter in Dr=80% (dense ground); (d) Square piles 10 mm x 10 mm in Dr=80% (dense ground).

7. Numerical Analysis Validation

The present uncoupled method is based on the load-transfer of row of piles subjected to lateral soil movement developed by (Jeong et al., 2003). The validity of the uncoupled model was tested by comparison with other's coupled method of analysis result. Cai and Ugai (2000) performed numerical analysis to investigate the effect of stabilizing piles on the stability of a slope. They performed a coupled analysis based on a three-dimensional finite element method with an elastoplastic constitutive model and shear strength reduction technique. The actual factor of safety is the ratio of the soil's shear strength to the reduced shear strength at failure. So, in shear strength reduction technique, the factor of safety is calculated using a finite element method by reducing the soil shear strength until collapse occurs. The numerical results by their coupled analysis were compared with those obtained by present method (uncoupled analysis).

An idealized slope with a height of 10 m and a gradient of 1 V: 1.5 H and a ground thickness of 10 m are analyzed with a three-dimensional finite element mesh, as shown in Fig. 12. A steel tube pile with an outer diameter (D) of 0.8 m was used. The piles are treated as a linear elastic solid material and are installed in the middle of the slope with Lx=7.5 m and the center-to-center spacing=3D. The piles are embedded and fixed into the bedrock or a stable layer. The material properties for prediction purpose were selected based on their results, as shown in Table 8. The horizontal soil movement was assumed; the profile was back calculated by fitting their calculated lateral deflections on different head conditions to that computed by the present method.

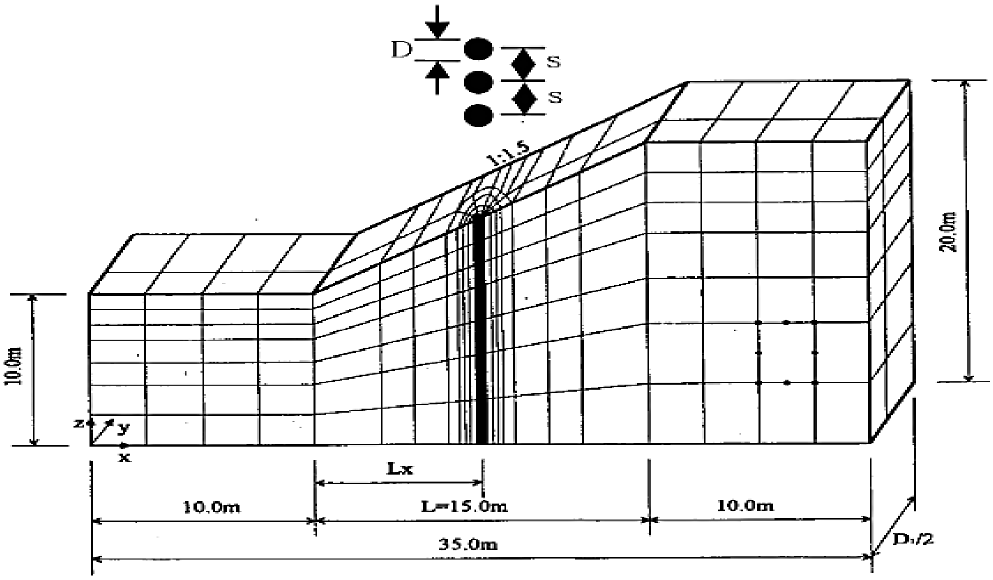


Fig. 12 Model slope and finite element mesh (after Cai and Ugai, 2000).

Table 8. Material properties and geometries (after Cai and Ugai, 2000)

Material	Model	Properties	Values
Soil	Mohr-Coulomb	Unit weight (kN/m^3)	20
		Cohesion(kPa)	10
		Friction angle (°)	20
		Elastic modulus (kPa)	2.0 x 10^5
		Poisson's ratio	0.25
		Coefficient of earth pressure at rest, K_0	0.66
Steel Pile	Isotropic elastic	Unit weight (kN/m^3)	78.5
		Elastic modulus (kPa)	2.0 x 10^8 – 6.0 x 10^7
		Poisson's ratio	0.2
		Diameter (m)	0.8
Interface	Elastic modulus (kPa)		2.0 x 10^5
	Poisson's ratio		0.25
	Cohesion (kPa)		10
	Friction coefficient, η		0.364

When the slope is not reinforced with piles, the present method and Cai and Ugai (2000) shear strength finite element method gave safety factors of 1.13 and 1.14 respectively, these compare well with each other. The failure mechanism in the shear strength reduction finite element method was represented by the difference between the nodal displacements just before failure and the nodal displacements when the shear strength reduction factor is equal to unity.

On the other hand, the safety factor of a slope on pile spacing is shown in Fig. 13. As expected, the rate of increase in the safety factor increases with decreasing the pile spacing. This figure also shows that the present method (uncoupled analysis) can obtain a quite similar rate change but higher value in the safety factor compared to the shear strength reduction finite element method (coupled analysis) proposed by Cai and Ugai (2000). However, Bishop's method based on limit equilibrium method cannot consider the influence of the pile head conditions on the safety factor due to the limit of Ito-Matsui's pressure equation, which is derived for rigid piles.

Fig. 14 shows coupling effects in the safety factor on pile positions obtained in this study with the solution presented by Cai and Ugai (2000). The coupled results, obtained with the shear strength reduction finite element method show that the improvement of the safety factor of slopes reinforced with piles is largest when the piles are installed in the middle of the slopes, irrespective of pile head conditions. However, present uncoupled solution shows that the piles should be placed slightly closer to the top of the slope for the largest safety factor. This is the same as the results of the Bishop's method.

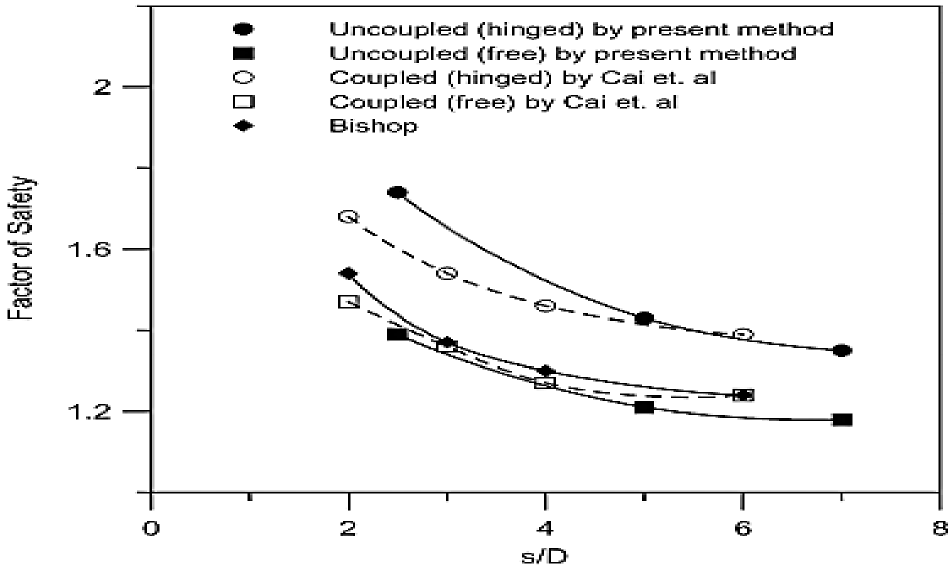


Fig. 13 Effect of pile spacing on safety factor.

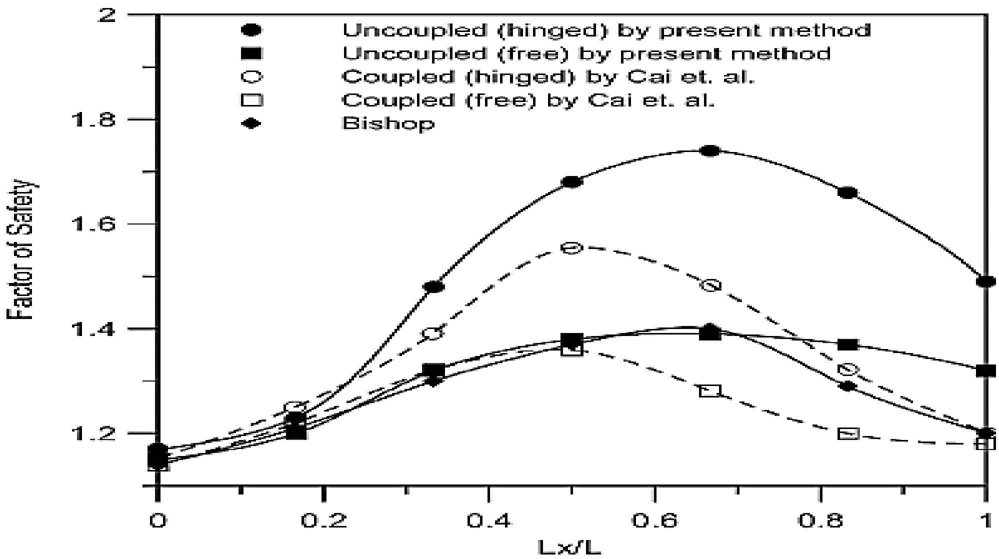


Fig. 14 Effect of pile positions on safety factor (s/D=3).

The reason for this is that when the piles are placed in the middle portions of the slopes, the strength of the soil-pile interface is sufficiently mobilized by the fact that the pressure acting on the piles is larger than that on the piles in the upper portions of the slopes. This figure also shows that the safety factor of slopes by uncoupled analysis is larger than that by coupled analysis. This clearly demonstrates that there exists pile/slope coupling; so that the critical surface invariably changes due to addition of piles and thus, the uncoupled analysis considering a fixed failure surface is limited in its application.

8. Conclusions

In this research, the prevention mechanism of SDSP was studied through the numerical analysis in which the effectiveness of the reinforcing effect of SDSP in landslide prevention is validated by their long term ability in resisting a relatively large deflection through both the theoretical and analytical analyses. From the findings, the following conclusions could be made:

1) FEA was found to be in good agreement with the calculated results though overestimation was expected due to the assumed 2D plane strain simplification in the presumed conditions for both the circular and rectangular bars in the numerical simulation.

2) Resistance to both lateral and axial forces is significantly enhanced with multirow arrangements of SDSP in landslide prevention.

3) In loose ground, the reinforcing effect is generated mainly through the bending stiffness (EI) of the reinforcing materials while in a densely compacted ground, shearing resistance is mobilized at a considerably higher strain, denoting the increased of the reinforced soil's strength.

4) Failure mode in dense ground is governed by the shearing resistance of the reinforced soil while material's EI becomes a dominant factor in loose ground condition regardless of the piles arrangement.

5) Regardless of pile sizes (○3mm or □10mm), material's EI plays a significant role in ensuring the overall reinforcing capacity of the piles. In case when more than 2 rows of piles are arranged, the coupled effect of both reinforcement and countermeasure should be carried out simultaneously.

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