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Research Article

Impact on slopes with development of shear band

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Abstract
All the major civil engineering structures in the hilly regions lead to strain localization in slope mass significantly during the occurrence of seismic excitation. These activities cause destruction and failure, which are increasing sharply over the last few years. Therefore, the paper is focused to investigate the inelastic deformations in slopes with three different inclinations of slope i.e., 1:2.25, 1:2 and 1:1.75 with the consideration of accumulation of strain in slope. The inelastic deformation in slopes under intense localization of shear strain is observed considering modified cam-clay (MCC) material model in plane strain condition. The non-linear finite element analyses have been conducted on slopes considering Chamoli excitation (1999) with a peak ground acceleration (PGA) of 0.36g. The impact of peak response of acceleration, displacement and deformation has been investigated and found that the amplification response increases with height and inclinations of slope. It is also observed that the development of shear band on slopes results in some degree of instability and intense mark of strain localization in all the slopes. The slope of 1:2 is not found collapsed at all but showing local failure, whereas the slope having inclination 1:2.25 is found relatively stable. The inclination of 1:1.75 results complete collapse due to formation of shear band. The crest of the slope is showing the maximum response in terms of acceleration and displacement in each case. Therefore, the formation of shear bands is thought to be an important aspect in understanding soil-slope stability and failure patterns.

Keywords
Slope, Inelastic deformations, Shear band, Strain localization, Plain strain, Modified cam-clay.

1.Introduction
It is common to observe the failure of slopes at the occurrence of seismic events. The conventional way of slope stability was based on estimation of factor of safety. This method for finding the stability of the slope was not able to predict the deformation behavior in slope. The concept of strain localization was studied in the last two decades to know the behavior of slope. The behavior was accurately estimation of the failure zone within the slope mass. This type of failure occurs either in tension or localization of strain in soil mass. It was found that many slopes have failed in this manner just because of tendency of progressive in strain accumulation behavior. The strain softening material behavior leads to progressive failure [1–4]. The majorities of Geotechnical engineering problems are caused by the creation of a small shear band or the localization of substantial shear strain [5–10].

Shear zone is a term that refers to a concentration of shear deformation on a slope that corresponds to a slippage failure of the surface (*Figure 1*).

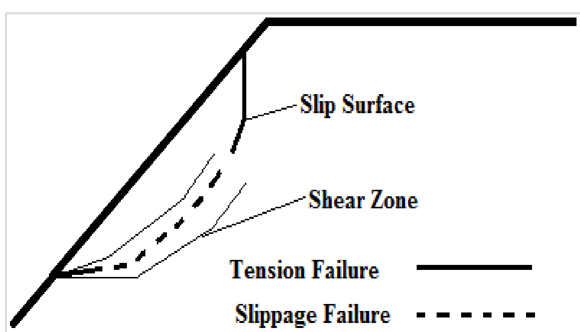

Figure 1 Formation of shear zone and failure surfaces [8]

The stability of slopes was carried out by studying the shear band and formation in the slopes. The development of shear band was exposed to be prevalent in softening material. Although, the shear band development is a crucial aspect in

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understanding soil-slope instability. The fundamentals of shear zone development and shear band formation have been investigated in this study. A prior study, on either hand, was mostly based on stress field analysis, which cannot be adequately captured in field or model experiments. For this analysis certain assumptions have to be made in order to elucidate the failure mechanism. To analyze the progressive failure strain softening behavior needs to be considered during the numerical analysis. Very few researchers have considered this aspect for the study. The impacts of seismic load on soil-slope have been studied to investigate the shear band development using finite element model (FEM). To simulate the strain localization, softening behavior and development of shear band, three inclinations of 1:1.75, 1:2 and 1:2.25 have been considered. The modified cam-clay (MCC) material model was employed for the analysis of strain softening behavior considering different slope inclinations. Impact of inclination of slope was also considered in this study.

The current study is to investigate the impact on the slopes with development of shear band under seismic excitation. To fulfill the objective, recent literatures on shear band phenomena in slopes, their findings and problems considered in this work have been discussed in section 2. The adopted methodology and modeling work in FEM has been described in section 3. The results obtained has been discussed in section 4. The work carried out in this paper has been discussed and summarized in section 5. Final conclusions which include limitations and future scope have been described in section 6.

2.Literature review

Kang et al. (2022) [11] studied the concave and convex shape of the slope under traditional methods of limit equilibrium to evaluate the factor of safety and failure surface considering the strength reduction method. It was concluded that concave slope had shown increases in the lateral stress on both sides of the soil. Thus, the significant difference was observed in case of concave slope result with the proposed method. Hazeghian and Soroush (2022) [12] predicted the new way of orientation of shear band in dense soil using the discrete element method with three surfaces of Roscoe, Coulomb and Arthur surfaces. The result concluded that the orientation of shear band completely depend on characteristics behavior of granular soil under the sample is subjected. Liu et al. (2022) [13] performed three methods compression test, digital image correlation and scanning electron microscopy to study the development of shear band in sensitive soil under low to high confining pressure. It was observed that a large number of shear band emerges at a high confining pressure, which changes the internal properties of soil. The failure of soil occurred at the point of completely developed shear bands in the specimen. Wu et al. (2022) [14] conducted various experiments for investigating the physical and mechanical characteristics of shear band. The nonlinear stress-strain curve was plotted to characterize the creep, microstructure, deformation behavior of shear band in landslides region. It was concluded that deformation of shear band significantly depends on the confining pressure and moisture content. Chen et al. (2022) [15] conducted experimental and analytical approach for an embankment slope under seismic excitation to evaluate the peak ground amplification factor using PLAXIS, FEM. The numerical results were in good agreement with experimental results. Chang et al. (2021) [16] analysed the instability of the slope and foundation considering the orientation of shear band based on Drucker-Prager elastoplastic constitutive model under anisotropic medium. It was concluded that the orientation of the shear band pattern and bifurcation significantly affect the bearing capacity of soil-slope-foundation. Ying and Huang (2021) [17] implemented new propagation criterion for catastrophic failure in an infinite slope for clay and sand. Elastic deformation of shear strain was studied considering the process zone approach in the shear band. It was observed that shear band propagation occurred along the thin layer of weak material. Yanqui (2021) [18] studied the development of shear band in dense clay having rhombic frictionless spheres packing. The results of packing model reported that the experimental results were in good agreement. Chen et al. (2021) [19] studied the slope failure in landslide considering a novel method of coupled Eulerian-Lagrangian based on Monte Carlo simulation under seismic excitation. The study concluded that shear band phenomena developed along the lower strength weak layer of soil and the behavior of spatially varying soil converges as slope length increases. Zhang and Puzrin (2021) [20] established the time and depth integrated FEM to keep submarine structure away from the landslides. The weak layer in the submarine was studied through shear band propagation considering the large deformation analysis. Nitka and Grabowski (2021) [21] conducted a 3-dimensional (3D) model test to know the behavior of the localized zone in grained material. It was concluded that shear band evolution with 3D model test was found in very good agreement with numerical results. Hsu et al. (2020) [22] studied the ground vibration impact on the development of shear band in slopes. The slope stability for large scale landslide areas was performed and observed that shear band and ground vibration resulting lower shear strength parameter. Kwak et al. (2020) [23] studied the shear band characteristics and deformation behavior on normally consolidated and over-consolidated clayey soil. The detailed analysis of entire shear failure from initial state to post failure state has evaluated by particle image velocimetry. The failure process was concluded in four stages and shear band formation was observed in the third stage i.e., softening stage. Lanting et al. (2020) [24] investigated slope deformation and shear band evolution using transparent soil to see the geometry of shear band. The result showed that deformation is synchronous to the propagation of shear band. Also geometry of shear band influenced significantly to the deformation of slope. Zhang et al. (2020) [25] proposed a novel methodology for propagation of a planar shear band. The behavior of planar shear band propagation in weakened zone of offshore structure. The behavior was analyzed using finite element and finite difference numerical modelling and found in good agreement with the numerical results. They also found that the aspect ratio of initiation zone significantly affects the behavior of a planar shear band. The proposed method can be utilized in the stability of slopes, hazard prediction, and vulnerability assessment for offshore developments. Zhang and Wang (2020) [26] explored the retrogressive spreading failure in infinite slopes. The large deformation concept was developed to study the propagation of quasi-horizontal shear band in slopes. In this study, slope stability with shear band propagation criterion for sensitive clay is established. Wang et al. (2019) [27] studied the shear band development based on large-deformation FEM method. Seismically induced post-earthquake failure was observed. The various types of failures resulted during the analysis. They concluded that this approach is helpful in seismically large-scale landslides.

Kido and Higo (2019) [28] performed triaxial compression on unsaturated sand to study the behavior of shear band. In this study x-ray micro computed tomography was observed for deformation stage in the soil. Shinoda et al. (2015) [29] conducted an experimental and numerical analysis of slopes behavior for different inclinations. In this study shake table test was performed on the slopes. The results were calculated in terms of dynamic shear strain. The result was in very good agreement with numerical study. This was concluded that the proposed method is also very useful for rock slope stability. Zhang et al. (2015) [8] conducted the series of centrifuge model tests on the slope model to observe the behavior of soil, slope inclination, and loading condition. Shear zone, a novel concept was introduced for describing the shear deformation localization in the slope. They observed that plane strain samples fail always along a well-defined shear zone and thus progressive failures in slope. Troncone (2005) [30] has investigated deep excavations of the slope using a nonlocal elasto-viscoplastic model with FEM and suggested that progressive failure start from the toe of slope. They studied progressive failure in slope models due to deep excavation.

It was found from the literature that shear band mostly developed in the weak section of the slope. It helps researchers to predict the slip or failure plane in slopes. Most of the study came out through the strain accumulation behavior in slope results in development of shear band. The experimental study was also performed to see the real behavior of band formation and its propagation in slope by many researchers [28, 29]. It was observed that the shear band propagation results progressive failure in slope. Most of the study found these progressive failures started to propagate from the toe of the slope and leads to tension crack at the crest of slope [31]. The formation of shear band phenomenon was more prominent found in clay soil. Shear band phenomenon in static cases were possible and observed in the past studies [12, 23, 32–36].

The detailed study on shear band discussed above directs that the seismic excitation impact can be significantly essential for shear band development. A few researchers had considered the seismic effect on shear band development with elementary boundary condition. As far as elementary boundary condition, not able to absorb the reflected waves at the boundary considering seismic excitation. The slope failure in a progressive manner due to formation of shear band has not been given much attention in the past studies. However, such study was found very limited which provide the inelastic stability of soil-slope with viscous boundary and progressive failure. This aspect has motivated to analyze the inelastic behavior of slope under such condition with seismic excitation. In this study, progressive failure behavior in slope, viscous boundary was considered and results were compared with other slope models.

2.1Problem statement

The vulnerability of slopes has increased significantly over the past few years due to increase in occurrence of seismic excitation, the large extent of construction activities and localization of strain in slope. Recently, slope failures have occurred at Kalijhora in Darjeeling district, India on National Highway-31 (*Figure 2*). Considering this, engineers and researchers are struggling with behaviour of slopes and influence zone of soil in sloping region. Hence, the present study is carried out to find the influence of seismic loading on the development of shear bands in soil slopes of different inclinations (Vertical: Horizontal) 1:1.75, 1:2 and 1:2.25. The size of the soil domain is 115 m long, 50 m wide and 60 m tall as mentioned in *Figure 3*. The vertical height of the slope was assumed constant throughout all the inclinations. The slopes for all three inclinations were considered in the present study.

The recent studies direct that slope failure due to accumulation of strain found very commonly in slopes [14–18]. This aspect of behavior motivates to study the response of soil mass during slope failure. The deformation behavior in entire slopes and response at most vulnerable locations such as the toe of slope (Node A), mid of slope (Node B) and at the crest of the slope (Node C) are considered. Also, the effect of boundary condition on development of shear band is discussed.

Figure 2 Slope failures at Kalijhora on Sikkim Highway, in Indian Himalayan (images by author)

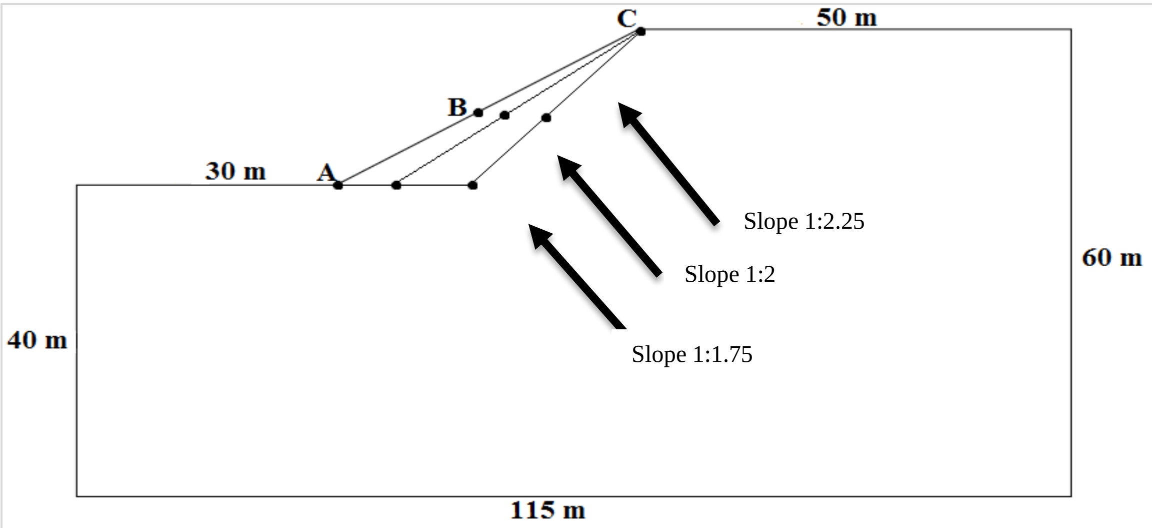


Figure 3 Slope model with three different inclinations

Seismic excitation of Chamoli earthquake (1999) with peak ground acceleration (PGA) of 0.36g (*Figure 4*) was applied as an input motion at the base of soil-slope to investigate the amplification response and behavior of the slope under shear band development.

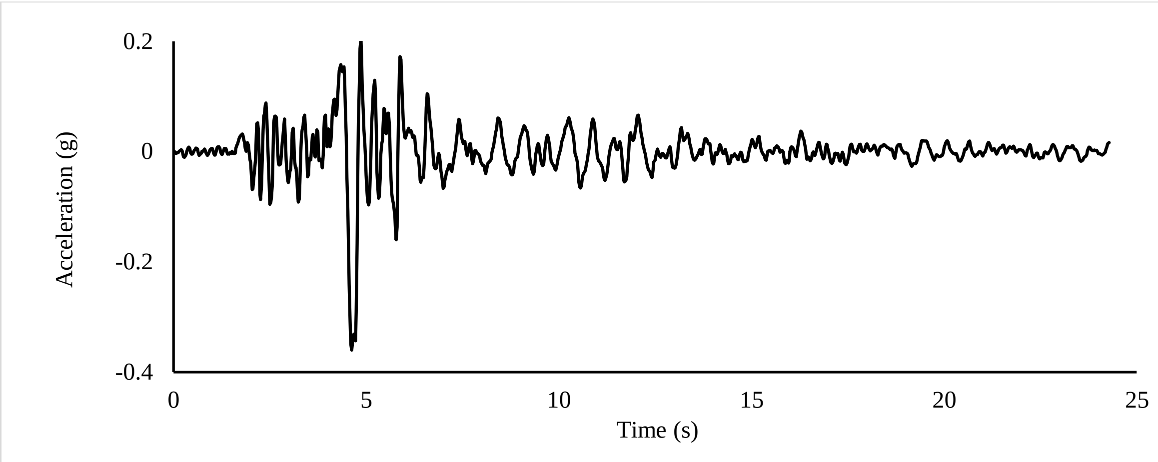


Figure 4 Chamoli (1999) earthquake acceleration-time history

3.Methodology

The finite element numerical modeling was adopted for predicting the stress and strain accumulation and behavior of shear band development in slope mass. The modeling of soil was conducted in 2-dimensional (2D) using the MCC material model. A FEM with 15 nodded triangular elements in plain strain condition were used to find the localized shear strain in the slope and development of shear band (*Figure 5*). Lysmer and Kuhlemeyer (1969) [37] boundary was applied at the side boundaries of soil-slope model.

Table 1 shows the material parameters which are essential to define MCC material model. A material property of slope is shown in *Table 2*.

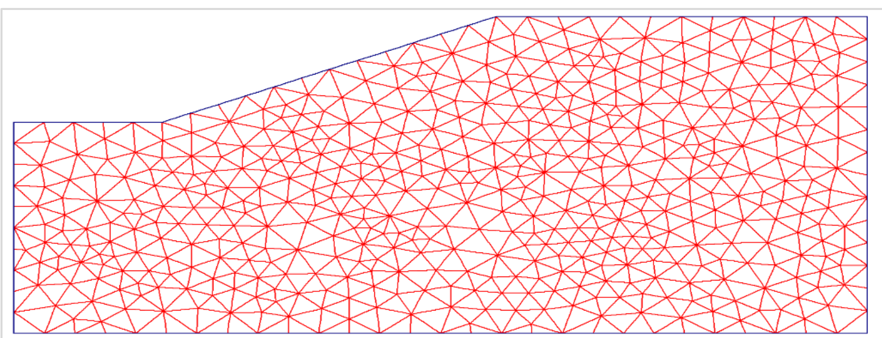


Figure 5 FEM model of slope

Table 1 Material behavior of soil slope after Zhang et al. (2013) [38]

Parameters of MCC material model	
Poisson’s ratio (ν)	0.3
Swelling Index (κ)	0.05
Compression Index (λ)	0.08
Tangent on Critical State Line (M)	0.701
Initial Void Ratio (e)	0.37

Table 2 Soil slope parameters after Zhang et al. (2013) [38]

Properties of soil	
Dry Density of Soil	19.6 kN/m^3
Damping	15%
α and β Co-efficient	0.2805 & 0.1212
Slope Inclination	1:1.75, 1:2 and 1:2.25

The behavior of slope in shear was investigated in PLAXIS 2D software using strain analysis. The peak shear strain was chosen to represent the slope shear deformation. The fifteen node quadrilateral elements were used to find the maximum shear strain in soil. The responses of acceleration and displacement at above discussed response nodes with three different inclinations were calculated.

The steps involved in the method adopted is shown in the flow diagram (see *Figure 6*).

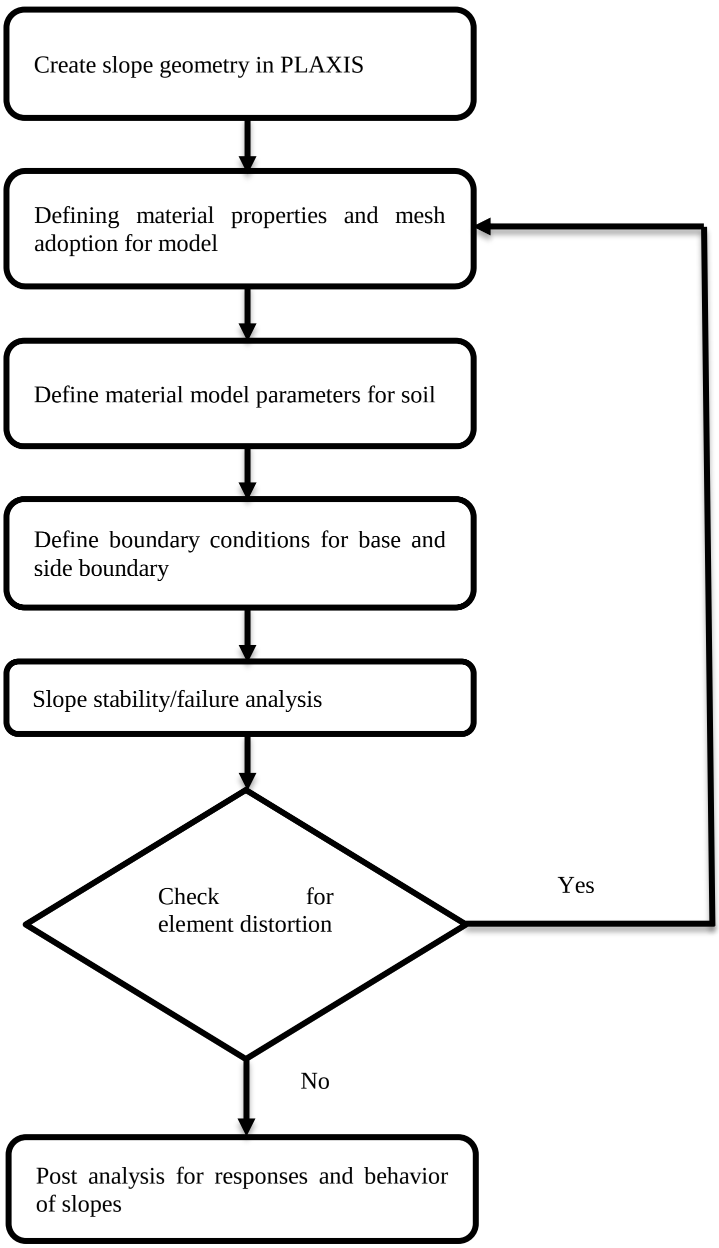


Figure 6 Flow diagram for FE analysis of slope

In order to evaluate the stability of the slope due to deformation theory, behavior of the slope has been carried out in PLAXIS. A non-linear finite element analysis was used to assess the behavior of slopes. The MCC material model with an associated flow rule for the soil was considered in this approach. The viscous boundary condition was applied on the lateral boundary of model and base was restrained in vertical (downward) direction with applying roller on the boundary (i.e. y-displacement is zero at the base). The lateral boundaries were allowed to displace in vertical (downward) direction (i.e. x displacement is zero at left and right boundaries). The stepwise procedure is shown in the flow diagram (see *Figure 6*). The slope geometry was created with the dimensions given in the *Figure 3* as well as the physical material properties defined for the soil used. After this the boundary condition was applied by generating finer mesh. The effect of stress and strain is seen in the slope by giving seismic excitation at the base of the model. The changing behavior of the slope was observed with the progress of shear strain and stresses corresponding the stresses and strain accumulation in finite element of slope model.

3.1Yield function of modified cam-clay model

Critical state theory can be used to analyze the elasto-plastic strain hardening/softening model. Critical state theory is the basis of cam-clay (CC) and MCC models. This model is appropriate for materials that exhibit limitless deformation while maintaining constant stress or volume. The logarithmic relationship exists between key parameter (mean stress and void ratio) of model. The advantage of MCC model is for largest value of mean effective stress, yield surface become horizontal. Therefore, no incremental deviatoric plastic strain takes place for a change in mean effective stress. Three variables, mean stress, deviatoric stress, and specific volume, are used to describe the state of a soil sample in critical state mechanics. The specific volume V is defined as in Equation 1

$$V = 1 + e \qquad (1)$$

The compression of soft soil under isotropic stress and drained conditions produces virgin consolidation line.

The Equation 2 shows the virgin consolidation line (*Figure 5*).

$$V = N_c - \lambda \ln(-p) \qquad (2)$$

Swelling line will be represented as shown in Equation 3.

$$V = V_s - \kappa \ln(-p) \qquad (3)$$

The values λ, κ and N are characteristic properties of a particular soil.

λ = slope of the $V - \ln p$ normal virgin consolidation line.

κ = swelling index (slope).

N_c = at unit pressure, the value of specific volume (of normal compression line).

V_s depends on loading history soil.

State of soil represented on line abd is normally consolidates whereas below this line is over consolidated. Outside the virgin consolidated line there is no existence of soil (*Figure 7*).

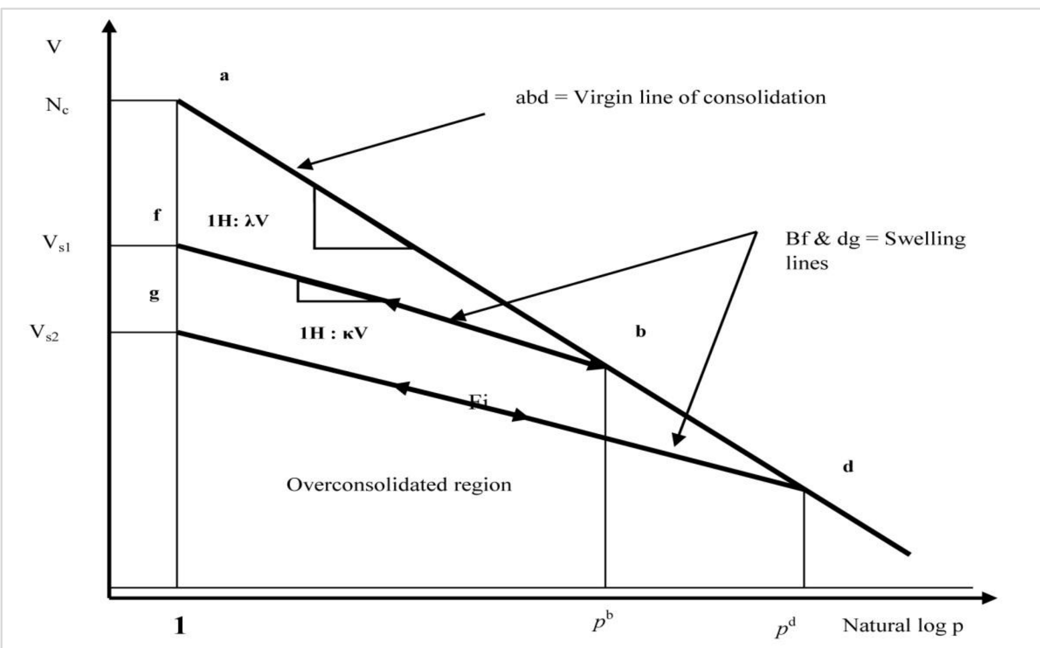


Figure 7 State of soil in consolidation (Oedometer) test

MCC model assumed a logarithmic relation between void ratio and the mean effective stress in virgin isotropic compression as shown in Equation 4. Which also gives the material compressibility in primary loading

$e - e^0 = - \lambda \ln (p/p^0)$ (4)

where, λ is the CC compression index in isotropic condition.

Compressibility of material can be determined in case of unloading and reloading condition by Equation 5.

$e - e^0 = - \kappa \ln (p/p^0)$ (5)

Modified cam-clay model yields as a function of Equation 6:

$$f = \frac{q^2}{M^2} + p'(p' - p_p) \quad (6)$$

p_p = preconsolidation stress corresponds to infinite unloading and reloading lines in (p'-e) plane.

Boundary of stresses at elastic state is defined as yield surface. Yield surface corresponding to f = 0 become an ellipse in p' – q - plane (*Figure 8*). Elastic and plastic strain increment can be determined from the stress path within this boundary and cross the boundary respectively.

The intersection line in *p'*- *q* – plane gives the critical state line (CSL) as Equation 7.

$q = mp'$ (7)

M = tangent of the CSL and gives the yield surface shape

q = ultimate deviatoric stress,

p = mean effective stress.

There are an unlimited number of ellipses, each corresponding to a different value of p_p. The failure surface may be on the left side or right side of the CSL. The values of q become very large in this side and softening of material i.e. plastic yielding occurs.

At critical state, shearing a soil sample for a long time eventually leads to a situation where more shearing can be done without any changes in stress or volume. The CSL defines this state, which is known as CSL. *Figure 8* shows a straight line passing through the origin with the slope M is CSL.

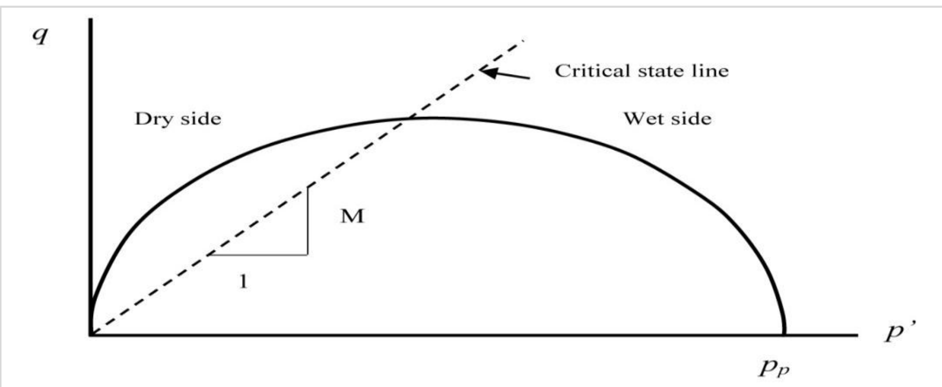


Figure 8 MCC model yield surface in $p' - q$ - plane

3.2Hardening and softening of soil

The hardening of the material as a result of plastic volumetric strain and compaction causes a drop in the void ratio and specific volume. Expansion of yield surface with respect to load increment from *n* to *n + 1* step is defined as Equation 8.

$$p_{c_{n+1}} = p_{c_{n+1}} \exp\left(\frac{v_n \Delta\varepsilon_v^p}{\lambda - \kappa}\right) \quad (8)$$

Where, $p_{c_{n+1}}$ = preconsolidation pressure.

When yielding happens to the right of where the CSL contacts a yield surface, hardening and compression occur called wet side of yield surface (subcritical side).

The soil material softens and dilates if yielding occurs to the left of the intersection of the CSL and yield surface (called the dry or supercritical side). After stress state reaches the initial boundary in a softening regime, the decrease in yield stress curve.

4.Results

The behavior and development of shear band are analyzed for three different inclinations. To understand the development of the shear band results in four different steps i.e., immediate act of seismic load up to 1.5 sec, 1.5 sec to 2.5 sec, 2.5 sec to 5.9 sec and after the completion duration of earthquake considered. Displacement and acceleration response of soil slope for these inclinations are also presented here.

4.1Behavior and development of shear band in soil slopes

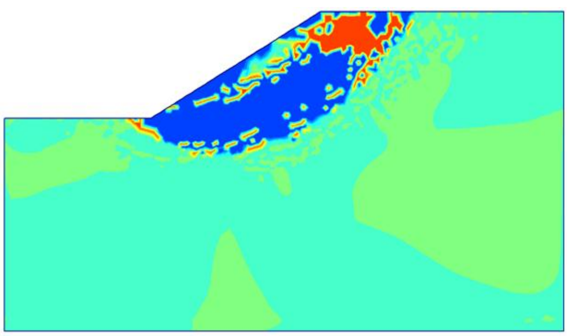
The nature of sensitivity and instability of the slope can be understood from the accumulation of strain which leads to the formation of shear band in slope. The results were presented in terms of deformation with increasing shear strain and strain accumulation in the slope (for the brevity).

Figure 9 shows the behavior of soil slope and development of shear band for inclination of 1:1.75. The deformation pattern between the start of seismic load to end of seismic load in four steps are depicted here. *Figure 9 (a)* corresponds to the immediate application of seismic load to 1.5 sec and it can be observed that the failure in slope is tension failure. Due to the tension, formation of the slip surface near the crest of the slope takes place.

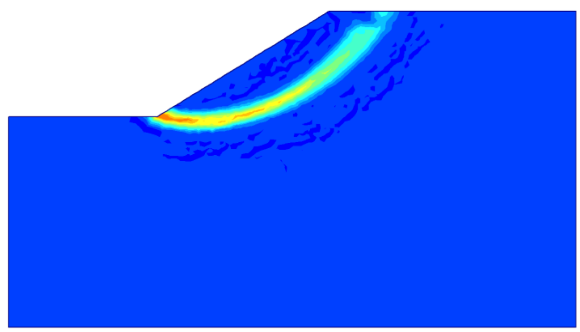
Figure 9 (b) shows the strain localization initiate at the toe of the slope after 1.5 sec in the slope mass. In case of strain softening behavior of soil, progressive failure starts to occur at the toe of the slope and progress towards the crest of the slope. This result is compared and validated with Zhang et al. (2013) [38].

Figure 9 (c) shows the deformation behavior at peak stage of seismic loading. The formation of shear band up to half of the height of slope and progressing towards the crest of the slope were found. This is another significance of progressive failure in strain softening behavior. This formation of shear band is occurring at peak amplitude of seismic load. The deep yield zone is likely to observe near the crest of the slope. The significance of deep yield zone is formation of tension failure at that zone. This was observed at the initiation of seismic load i.e. in first step.

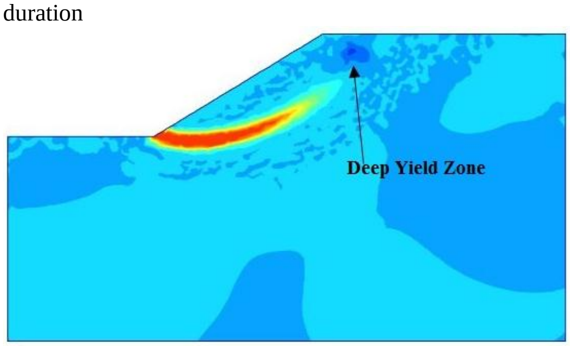
Figure 9 (d) shows the accumulation of strain is completely within the shear band. Complete mobilization of plastic strain in slope mass and formation of shear band along slope has also taken place. This is observed that the formation of shear band starts from the toe to the crest of the slope. So we can say that the mobilization of plastic strain in softening material is a progressive phenomenon and it happens because of inertial effects.



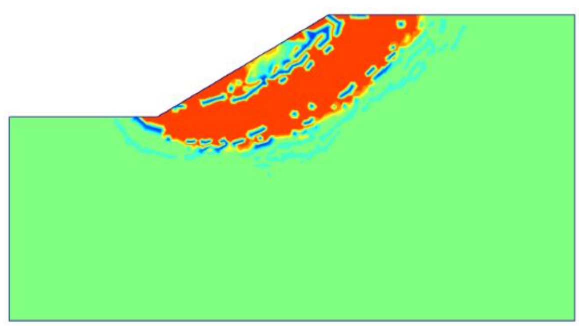
(a)Immediate application of seismic load up to 1.5 sec. duration



(b)1.5 sec. to 2.5 sec. duration



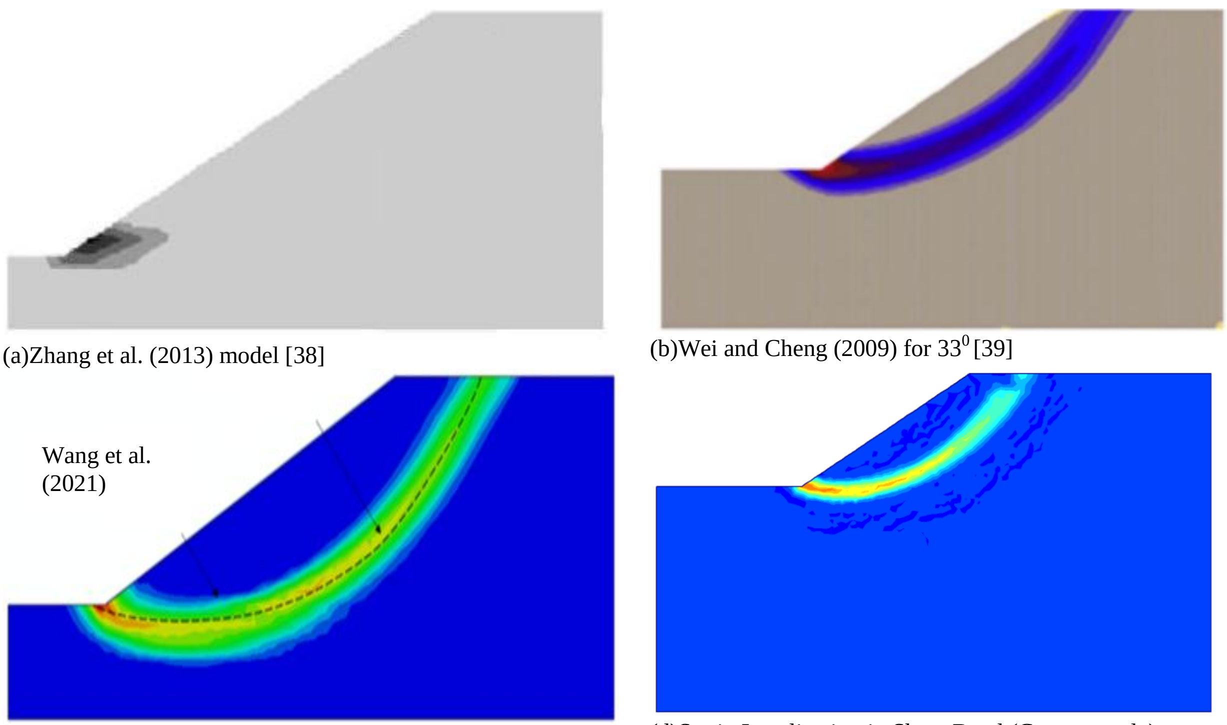
(c)2.5 sec. to 5.9 sec.



(d)After completion of seismic load

Figure 9 Behavior of the soil slope and development of shear band at various duration of times for 1:1.75 inclination

The behaviour of slope under development of shear band was validated with three typical slope examples of almost same slope angles such as 26^0 from Zhang et al. (2013) [38], 33^0 from Wei and Cheng (2009) [39] and 45^0 from Wang et al. (2021) [40]. It is essential to mention that all the slope model used accumulation and propagation of strain within the slope to address the critial failure surface. The induced failure surface in current study found in good aggrement with other slope model considrered. The failure surface obtained in current study was shown with others slope failure surface for slope angles of 26^0, 33^0 and 45^0 as shown in *Figure 10* (for brevity only one slope has shown here). It was also found that the strain accumulation in progressive shear band failure in softening type soil starts from the toe of slope. The current study also came with the same trend. Finding of current study suggest that progressive failure in slope start from toe and progress towards crest of slope. It has validated the present model the strain localized within the slope mass. The current study was also validated with the two material constitutive model and boundary effect. Zhang et al. (2013)[38] and Wang et al. (2021)[40] considered the Mohr-Coulomb material model in ABAQUS to observe the critical failure surface in progressive shear band and found in a similar trend with the material model (see *Figure 10* (a, c and d)). Further this study was compared with Wei and Cheng (2009) [39] and found similar trend in shear band development, which was pertfomed in ABAQUS for slope angle 33^0 (see *Figure 10b*). The study suggests that MCC matgerial model has shown the true behavior of slope under seismic condition.



(a)Zhang et al. (2013) model [38]

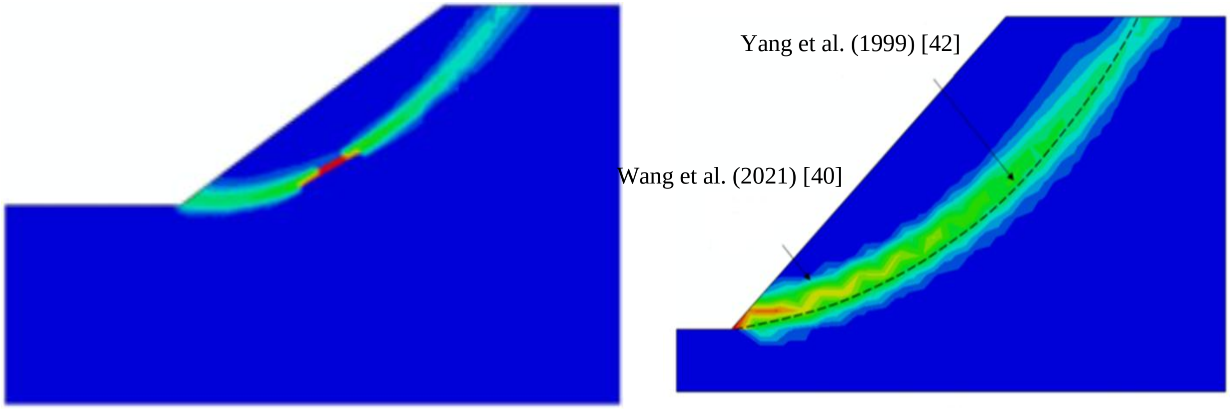
(b)Wei and Cheng (2009) for 33^0 [39]

(c)Wang et al. (2021) for 26^0 [40]

(d)Strain Localization in Shear Band (Current study)

Figure 10 Validation of strain localization in progressive shear band

The effect of boundary condition can be seen clearly in critical failure surface for 33^0 slope angle as it was found that strain accumulation started from approximately at mid slope (see *Figure 11 (a)*). It was indicating that elementary boundary condition makes the slope vulnerable to some extent [41]. The current study used the viscous boundary conditioin to simulate the behavior of shear development in slope. In all the combination of slopes angle, observed that the propagation of strain in shear band started from toe of slope. The most vulnerable slope for 45^0 slope inclination was also observed that the propagation of shear band started from toe of slope (see *Figure11 (b)*).

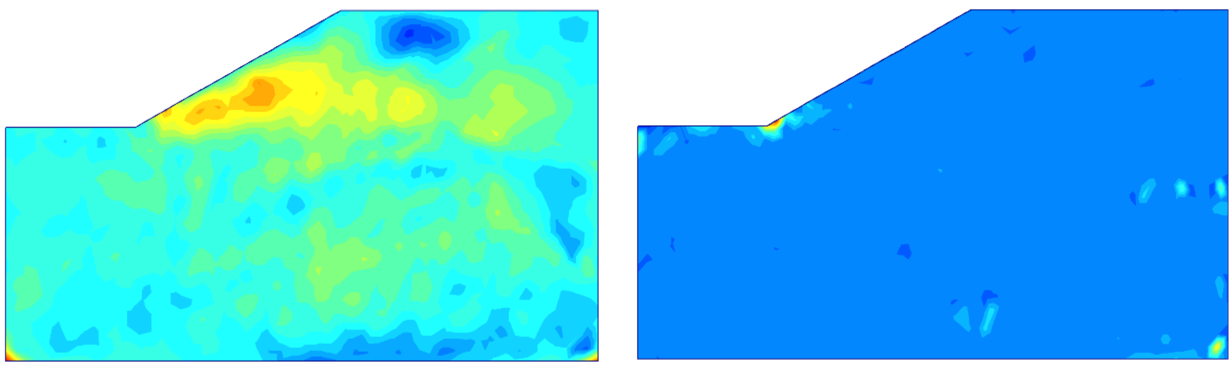


(a)Critical failure surface for 33^0 from Wang et al. (2021) [40]

(b)Critical failure surface for 45^0 from Wang et al. (2021) [40]

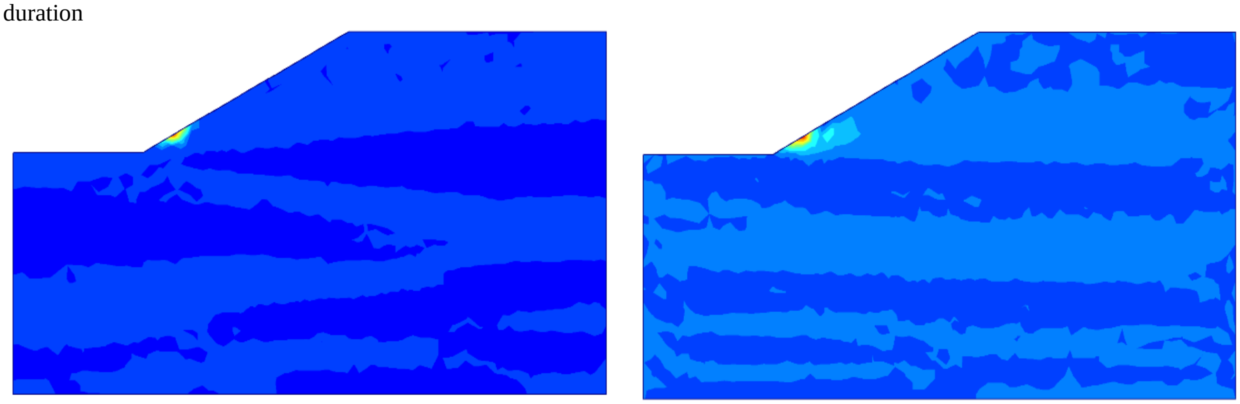
Figure 11 Validation of strain localization in progressive shear band

Figure 12 and *Figure 13* show the behavior of soil slope and development of shear band for inclination of 1:2 and 1:2.25, respectively. It can be observed in both of the cases that the overall slope remain stable and strain localization takes place at toe (*Figure 12(b), 12(c)* and *Figure 13(b), 13(c)*). *Figure 11(d)* shows the formation of shear band near to the toe, but further propagation of the shear band was not taken place. Since the stress mobilized in this condition (*Figure 12* and *Figure 13*) is lesser than the resisting stress, so deviation in the behavior of the slope is taking place.



(a) Immediate application of seismic load upto 1.5 sec. duration

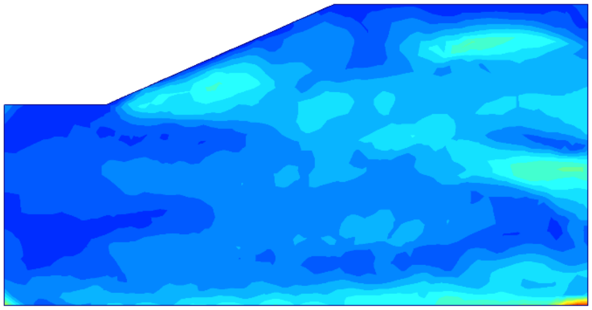
(b) 1.5 sec. to 2.5 sec. duration



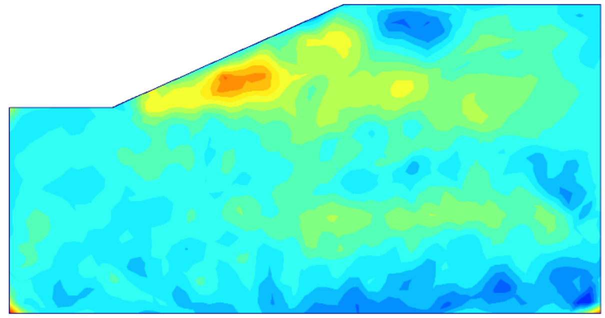
(c) 2.5 sec. to 5.9 sec.

(d) After completion of seismic load

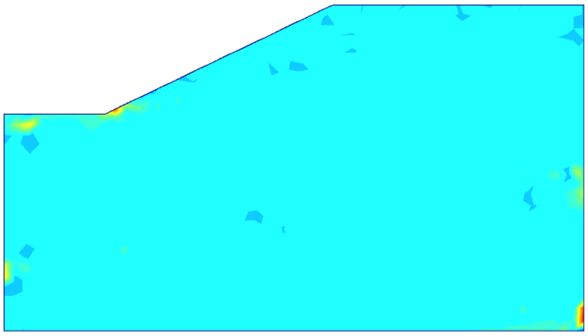
Figure 12 Behavior of the soil slope and development of shear band at various duration of times for 1:2 inclination



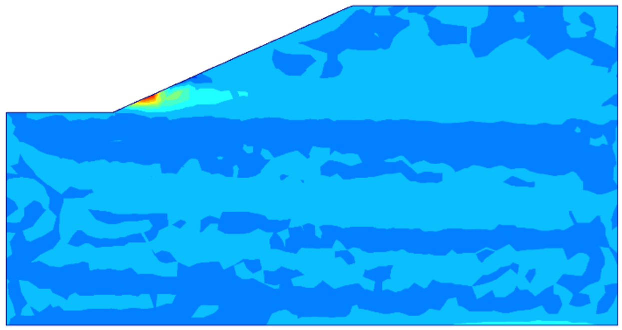
(a) Immediate application of seismic load upto 1.5 sec. duration



(b) 1.5 sec. to 2.5 sec. duration



(c) 2.5 sec. to 5.9 sec



(d) After completion of seismic load

Figure 13 Behavior of the soil slope and development of shear band at various duration of times for 1:2.25 inclination

4.2Effect of inclination on soil slope response under seismic condition

Response of slope for three different inclinations at three points Toe of Slope A, Mid of Slope B and Crest of Slope C have been observed. To describe the impact of seismic load at these points, PGA amplification factor (Ω) and maximum displacement response is used. PGA amplification factor is the ratio between the peak acceleration (PGA_o) at each point on the slope surface and the peak acceleration (PGA_i) of seismic wave input from the bottom of model. Equation 9)

$$\Omega = \frac{PGA_o}{PGA_i} \quad (9)$$

PGA Amplification factor for slope 1:1.75, 1:2 and 1:2.25 have been obtained at above mentioned points. Response of slope at points A, B and C are plotted in *Figure 14*. It can be observed that the PGA amplification factor increases with increase in the slope. This factor is also found minimum at the toe, while maximum at crest. Similar response is found for all slopes. But the coefficient is greater at crest (point C) for higher slope i.e. for slope 1:1.75.

The maximum displacement response of slope is presented in *Figure 15*. It can be observed that the displacement at point C i.e. the point at crest is higher for all slopes. With increase in the height of the point from the toe, displacement of the point is found increasing. It means under seismic load relative displacement exist between two different points of the slope. It can be further observed that with increase in the inclination of the slope displacement of the points on the surface of the slope (Point A, B, and C) increases. Here it can also be observed that at higher inclination of slope curve showing the displacement of point is not linear like at lower inclination. In case of higher inclination, displacement of point increases with increase in the height of the point from toe, but rate of increment of displacement decreases. In other words, it can be said that tendency of relative displacement decreases at higher inclination of the slope.

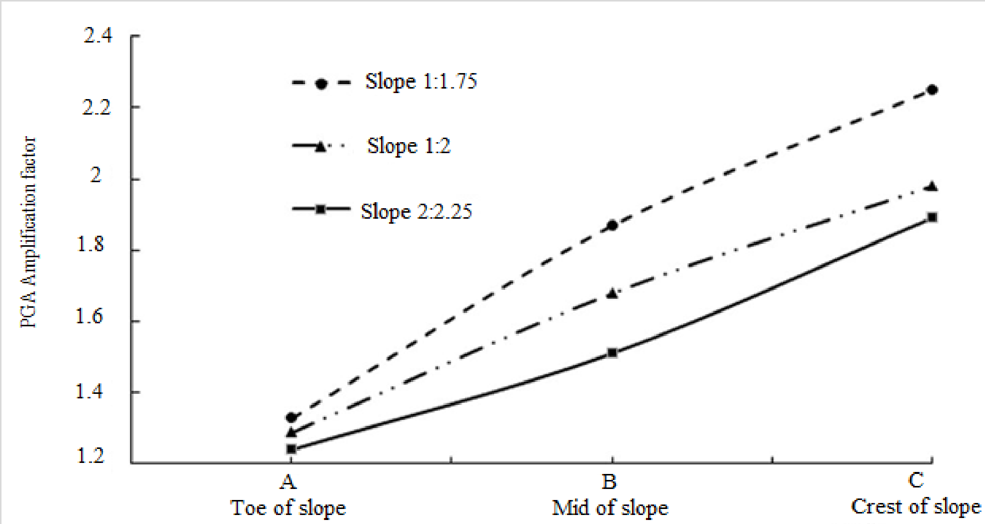


Figure 14 Horizontal PGA amplification factor for slope

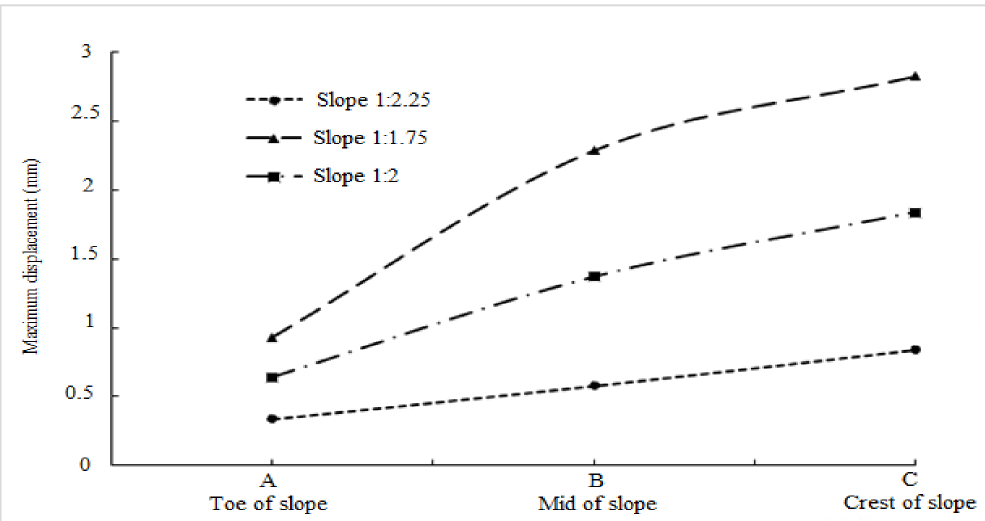


Figure 15 Horizontal displacements for slope

5. Discussion

A numerical study was performed on three inclinations of slopes to predict the instability of the slope under seismic loading condition only. The true failure surface under the effect of progressive strain accumulation in the slope was implemented with MCC material model and compared with the other material models and boundary conditions. The current material model showed the actual behavior of critical failure surface. In the case of most vulnerable slope inclination, it was found that slip surface forms exactly within the shear zone. Of all the cases of slopes, progressive failure was initiated from toe and tendency to progress seems to be towards the crest of the slope.

In the case of slope inclinations 1:2 and 1:2.25, the development of shear band was started from toe, but the further propagation of the shear band was not taking place in the slope mass due to stable behavior of slope.

Although stresses were mobilized in the slope mass under seismic loading condition, but the slope may remain stable. It is recommended that post seismic excitation, the study should be performed to ensure the stability of slopes. The viscous boundary is recommended to be implemented on the lateral boundaries of slope model (if any case) so that the reflection of waves can be observed at the boundaries. The result of current work will pave the way for structure safety on slopes.

An overall analysis of result came to the slope with inclination 1:1.75 was found most vulnerable slope. The slope inclinations 1:2 and 1:2.25 found relatively stable under seismic excitation. Occurrence of progressive failure due to formation of shear band may be the case with most vulnerable slope. The PGA amplification factor and maximum horizontal displacement were evaluated for all the cases and found that these factors increase with height of slope.

The slope stability is limited to seismic load only, however, it can be evaluated for various building configuration loads under seismic condition also. This study is limited to 2D cases only, but better results can be predicted with 3D analysis. The impact of shear band behavior of slope may be restricted to the residual strength of material used. This aspect can be extended for some more material model also.

A complete list of abbreviations is shown in *Appendix I.*

6. Conclusion and future work

It was observed that deep yield zone for slope inclination 1:1.75 and 1:2.25 causes the tension failure in the slope. The continuous yield zone along the failure surface was not observed. The slope instability is found in almost all the cases considered. The slope with an inclination of 1:2 has not collapsed completely due to local failure. Slope 1:1.25 found relatively stable. The inclination of 1:1.75 has found complete collapse due to formation of shear band. Development of shear band occurred along the slope with application of seismic load, but a delay of complete development of shear band in slope due to inertial effects. PGA amplification factor increases with an increase in the slope. In case of higher inclination, displacement of point increases with an increase in the height of the point from toe, but the rate of increment of displacement decreases. The creation of shear bands is thought to be an important. The transmitting boundary may affect the behavior of localized strain and its propagation in slope mass.

This aspect may be applied to the boundary of the slope for safer and economical modelling while the structure on slope, underground structures and geostructures etc. The study was limited to deformation behavior for free slopes only, however, it may be extended for placing structure on various positions of the crest and face of the slope. Impact of seismic excitation on the stability of buildings which will be built on this type of slopes will alter the behavior of buildings and soil, this can be studied.

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Conflicts of interest

The authors have no conflicts of interest to declare.

Author's contribution statement

Vijay Kumar: Concept and formulation, method of analysis, Writing – original draft, Analysis and Interpretation of results. **Sunita Kumari:** Supervision, final correction, investigation on challenges and Draft manuscript preparation.

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Appendix I

S. No.	Abbreviation	Description
1	2D	2-Dimensional
2	3D	3-Dimensional
3	CC	Cam-Clay
4	CSL	Critical State Line
5	DSSI	Dynamic Soil-Structure Interaction
6	FEM	Finite Element Method
7	MCC	Modified Cam-Clay
8	PGA	Peak Ground Acceleration