PSEUDO-DYNAMIC ANALYSIS OF SLOPE

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ABSTRACT

Analysis of slope under seismic loading condition is very much important. Numerous methods have been developed like limit equilibrium method, finite element method, stress deformation analysis, etc. to analyse stability of slopes under seismic conditions. Although, different methods are available to analyse the stability of slopes under both the static and seismic conditions, but the complete solution for the seismic stability of slopes is still an important finding. In the present work an attempt is made here to analyse the stability of slope considering linear wedge failure under pseudo-dynamic condition using limit equilibrium method, which can be used for the design of slope. The results are plotted in non-dimensional charts.

Keywords: Pseudo-dynamic method, stability number

I. INTRODUCTION

Analysis of slopes under seismic condition is an important finding for researchers and engineers. Numerous methods have been developed like limit equilibrium method, finite element method, stress deformation analysis, etc. to analyse stability of slopes under seismic conditions. Classical works to analyse slope under static condition were carried out by researchers like Taylor (1937, 1948) for translation failure of slope on a planar failure surface, the ordinary method of slices by Fellenius (1936) and Bishop's modified method given by Bishop (1955) for circular and log spiral failure surfaces are well understood. Non-homogeneous anisotropic soils with non-circular failure surfaces was analysed by Morgenstern and Price (1965), Spencer (1967), Janbu (1973), Chowdhury (1978) and Zhu et al. (2003). Using pseudo-static model slopes has been further investigated by Terzaghi (1950), Newmark (1965), Seed (1966, 1968), Sarma (1975), Kramerand Smith (1997), Rathje and Bray (1999, 2000), Loukidis et al. (2003), Wartman et al. (2003, 2005). In this present work, a planer failure surface passing through the toe is assumed for a homogeneous soil and by using limiting equilibrium approachstability number of any generalized slope under seismic condition is determined. Acceleration coefficients both in the horizontal and vertical directions are considered in the analysis with a variation of parameters like slope angle, soil friction angle.

II. METHOD OF ANALYSIS

Consider a slope of c-ϕ soil of height H and slope angle i as shown in Fig.1. Seismic inertia forces acting on the slope are $Q_h(t)$ in the horizontal direction and $Q_v(t)$ in vertical direction. AB is the planar failure wedge inclined at an angle α with the horizontal. R is the reaction acting on failure wedge at an angle ϕ_m with the normal of the

failure wedge. C=c_mL is the mobilized cohesion acting along the face AB (length L) and $\phi_m$is the mobilised internal friction angle.
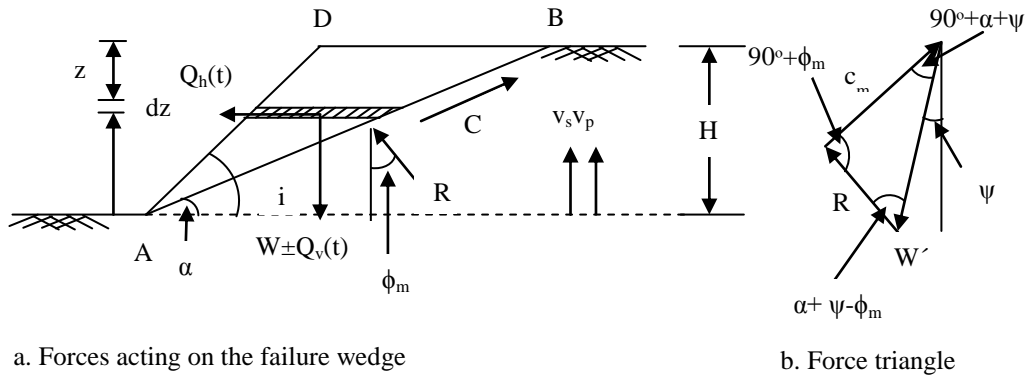

a. Forces acting on the failure wedge

b. Force triangle

Fig.1.Failure mechanism of the soil wedge

Weight of the wedge:

$$W = \frac{\gamma \times H^2 \left(\cot \alpha - \cot i \right)}{2}$$

(1)

The mass of a thin element of wedge at depth z of thickness dz is given by:

$$m(z) = \frac{\gamma}{g} \times \left(\cot \alpha - \cot i \right) \times \left(H - z \right) \times dz$$

(2)

For a sinusoidal base shaking subjected to both horizontal and vertical earthquake acceleration with amplitude k_hg and k_vg , the horizontal and vertical acceleration respectively at any depth z below the ground surface at time 't' can be expressed as:

$$a_h(z,t) = k_h \times \sin \omega \left(t - \frac{H - z}{V_s} \right)$$

(3) $$a_v(z,t) = k_v \times \sin \omega \left(t - \frac{H - z}{V_p} \right)$$

(4)Therefore, total horizontal inertia force and vertical inertia force acting on the failure wedge

are: $$Q_h(t) = \int_0^H m(z) \times a_h(z,t)$$

$$(5) \, Q_v(t) = \int_0^H m(z) \times a_v(z,t)$$

(6)

The resultant weight which makes an angle $\psi^{'}$ with vertical and is given as:

$$\psi^{'} = \tan^{-1} \frac{k_h}{(1 \pm k_v)}$$

(7)From Fig.1b using sine rule:

$$\frac{W^{'}}{\sin(90° + \phi_m)} = \frac{C_m}{\sin(\alpha + \psi^{'} - \phi_m)} = \frac{R}{\sin(90° - \alpha - \psi^{'})}$$

(8)Solving Eq.8 we get:

$$S_n = \frac{c_m}{\gamma H} = \frac{\sin\alpha \sin(\alpha + \varphi - \phi_m)(\cot\alpha - \cot i)}{2\cos\phi_m} \times \begin{bmatrix} 1 \pm \dfrac{\eta k_v}{H\pi^2}\left\{2\pi\cos\omega\varepsilon + \dfrac{\eta}{H}(\sin\omega\varepsilon - \sin\omega t)\right\} + \\[2mm] \dfrac{\lambda^2 k_h^2}{4\pi^4 H^2}\left\{2\pi\cos\omega\xi + \dfrac{\lambda}{H}(\sin\omega\xi - \sin\omega t)\right\}^2 + \\[2mm] \dfrac{\eta^2 k_v^2}{4\pi^4 H^2}\left\{2\pi\cos\omega\varepsilon + \dfrac{\eta}{H}(\sin\omega\varepsilon - \sin\omega t)\right\}^2 \end{bmatrix}^{\frac{1}{2}} \quad (9)$$

Where, $\varepsilon = t - \dfrac{H}{V_p}, \xi = t - \dfrac{H}{V_s}$

III. RESULT AND DISCUSSION

3.1 Stability Number

Stability number for different value of i and ϕ_m are presented in tabular form.

Table 1 Stability Number of slope under pseudo-dynamic condition at different inclination and soil friction angle

k_h=0, k_v=0				K_h=0.1, K_v=0			
i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$			

i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$	i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$
20°	0.011				20°	0.028	0.00387		
30°	0.031	0.01			30°	0.05	0.02	0.003	0.002
40°	0.053	0.02	0.007		40°	0.075	0.04	0.017	0.003
50°	0.077	0.05	0.023	0.006	50°	0.1	0.07	0.037	0.016
60°	0.105	0.07	0.045	0.023	60°	0.128	0.09	0.063	0.037
70°	0.135	0.1	0.072	0.046	70°	0.16	0.12	0.093	0.066
80°	0.169	0.13	0.105	0.077	80°	0.195	0.16	0.128	0.1
90°	0.21	0.17	0.144	0.116	90°	0.236	0.2	0.171	0.143
K_h=0.1, K_v=0.05				K_h=0.1, K_v=0.1					
i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$	i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$
10°	0.007737				10°	0.008215			
20°	0.027982	0.004116			20°	0.028141	0.004371		
30°	0.05146	0.020058	0.003055		30°	0.05247	0.020172	0.003244	
40°	0.076496	0.041947	0.016928	0.002686	40°	0.078458	0.04277	0.017024	0.002852
50°	0.103362	0.067263	0.038186	0.016055	50°	0.106365	0.068988	0.038936	0.016146
60°	0.132611	0.095808	0.064549	0.038178	60°	0.136757	0.098591	0.066205	0.038928
70°	0.16503	0.128066	0.095791	0.067237	70°	0.17045	0.13207	0.098575	0.068962
80°	0.201721	0.165006	0.132568	0.103308	80°	0.208586	0.170425	0.136713	0.106309
90°	0.244251	0.208157	0.176284	0.147554	90°	0.252795	0.215242	0.182073	0.152168
K_h=0.2, K_v=0				K_h=0.2, K_v=0.1					
i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$	i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$
10°	0.0288	0.0003			10°	0.033	0.0013		
20°	0.051	0.02	2E-04		20°	0.052	0.02	7E-04	
30°	0.075	0.04	0.011	1E-04	30°	0.076	0.04	0.013	6E-04
40°	0.1	0.06	0.031	0.01	40°	0.103	0.06	0.032	0.011
50°	0.126	0.09	0.056	0.03	50°	0.132	0.09	0.057	0.03
60°	0.155	0.12	0.084	0.056	60°	0.163	0.12	0.087	0.057
70°	0.187	0.15	0.117	0.088	70°	0.197	0.16	0.122	0.091
80°	0.223	0.19	0.155	0.126	80°	0.236	0.2	0.163	0.132
90°	0.264	0.23	0.199	0.172	90°	0.281	0.24	0.21	0.181
K_h=0.2, K_v=0.2									
i	$\phi=10°$	$\phi=20°$	$\phi=30°$	$\phi=40°$					
10°	0.0373	0.00325							
20°	0.053	0.02	0.002						
30°	0.078	0.04	0.015	0.001					
40°	0.106	0.06	0.033	0.013					
50°	0.137	0.09	0.058	0.032					
60°	0.17	0.13	0.09	0.058					
70°	0.207	0.16	0.127	0.094					
80°	0.249	0.21	0.17	0.137					
90°	0.297	0.26	0.221	0.19					

3.2 Parametric Study

Stability charts for different values of soil friction angle areplotted below
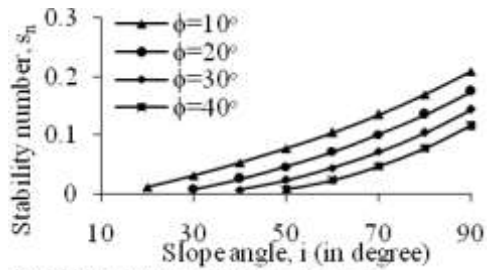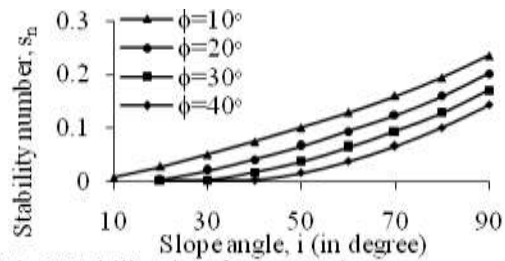

Fig.2.Stability chart for $k_h=0$, $k_v=0$

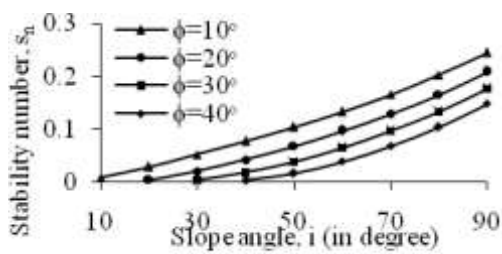Fig.3.Stability chart for $k_h=0.1$, $k_v=0$

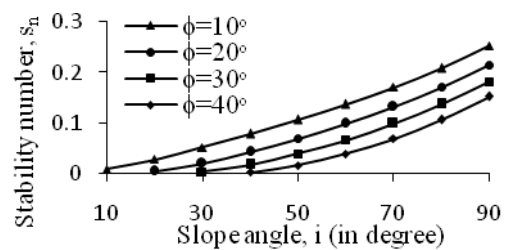Fig. 4. Stability chart for $k_h=0.1$, $k_v=0.05$

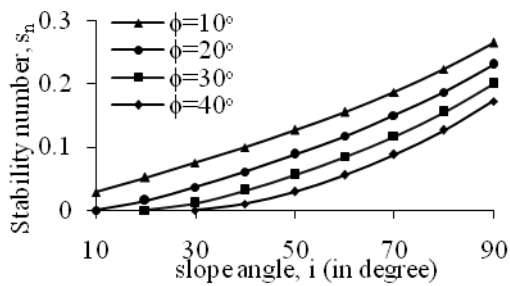Fig. 5. Stability chart for $k_h=0.1$, $k_v=0.1$

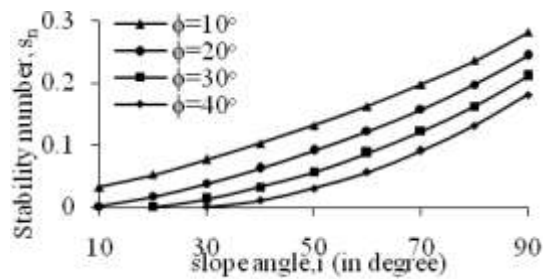Fig. 6. Stability chart for $k_h=0.2$, $k_v=0$
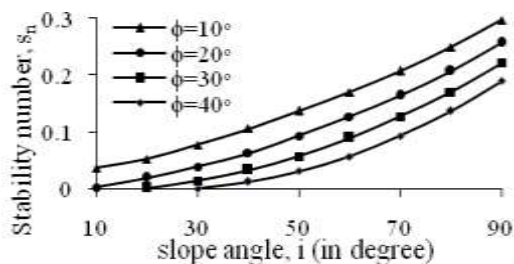

Fig. 7. Stability chart for $k_h=0.2$, $k_v=0.1$

Fig. 8. Stability chart for $k_h=0.2$, $k_v=0.2$

Stability charts for different horizontal and vertical acceleration coefficients are plotted below
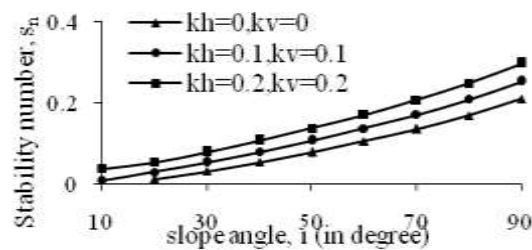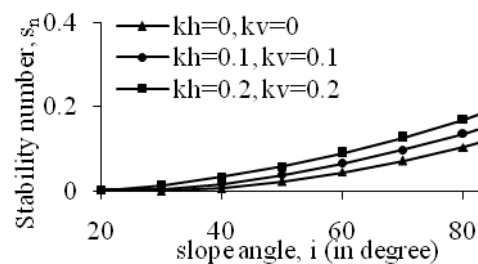
Fig. 9. Stability chart for ϕ=10°

Fig. 10. Stability chart for ϕ=30°

It has been observed that the value of stability number increases due to increase in k_h and k_v values.

IV. COMPARISON

Stability number obtained in the present study under pseudo-dynamic condition is slightly lesser than the values obtained by Ling et al. (1999) where they considered pseudo-static condition. The comparison is shown in Table 2.

Table 2 Comparison of present study with those obtained by Ling et al. (1999)

ϕ_m	i = 60°, k_h=0.1, k_v=0.05		i = 90°, k_h=0.1, k_v=0.05	
	Ling et al. (1999)	Present study	Ling et al. (1997)	Present study
20°	0.096613	0.095808	0.209887	0.208157
30°	0.065097	0.064549	0.177752	0.176284
40°	0.038512	0.038178	0.148787	0.147554

V. CONCLUSION

In this paper formulation is developed for stability number using pseudo-dynamic approach. Moreover, stability number and stability charts has been given using Pseudo-dynamic method for different values of k_h and k_v and linear interpolation is suggested for any intermediate value.

NOMENCLATURE

a_h , a_v Amplitude of horizontal and vertical seismic acceleration respectively

g   Acceleration due to gravity.

H	Height of the slope
k_h , k_v	Seismic acceleration coefficient in the horizontal and vertical direction respectively
S_n	Stability number
W	Weight of failure wedge
$Q_h(t)$, $Q_v(t)$	Horizontal and vertical inertia force due to seismic accelerations respectively
t, T	Time (seconds) and period (seconds) of lateral shaking
α	Angle of inclination of the failure surface with the vertical
ϕ	Friction angle of the backfill soil
γ	Unit weight of the soil
c_m	Mobilized cohesion
$λ = TV_s$	Wave length of shear wave
$η = TV_p$	Wave length of compression wave
W'	Resultant weight, $W' = \sqrt{Q_h(t)^2 + \{W \pm Q_v(t)\}^2}$

REFERENCES

[1]   Taylor, D.W. (1937). "Stability of earth slopes." Four. Boston Soc. Civil Engrs.,24(3), 197-246.

[2]   Taylor, D. W. (1948). Fundamentals of Soil Mechanics. Wiley, New York.

[3]   Fellenius, W. (1936). "Calculation of stability of earth dams." Proc. 2nd Congr. LargeDams, Washington, 4, 445-462.

[4]   Bishop, A.W. (1955). "The use of slip circle in the stability analysis of earth slopes."Geotechnique, 5(1), 7-17.

[5]   Morgenstern, N. R., and Price, V. E. (1965). "The analysis of the stability of general slip surfaces."Geotechnique, 15(1), 79-93.

[6]   Spencer, E. (1967). "A method of analysis of the stability of embankments assuming parallel interslice forces." Geotechnique, 17(1), 11-26.

[7]   Janbu, N. (1973). "Slope stability computations." R. C. Hirschfeld and S. Poulos,eds., John Wiley and Sons, New York, N. Y., 47-86.

[8]   Chowdhury, R. N. (1978). Slope Analysis. Elsevier, New York.

[9]   Zhu, D. Y., Lee, C. F., and Jiang, H. D. (2003) "Generalised framework of limit equilibrium methods for slope stability analysis." Geotechnique, 53(4), 377-395

[10]   Terzaghi, K. (1950). Mechanisms of Land Slides. Engineering Geology (Berkeley) Volume, Geological Society of America.

[11]   Newmark, N. (1965). "Effects of earthquakes on dams and embankments."Geotechnique, 15(2), 139-160. 22

[12]   Seed, H. B. (1966). "A method for the earthquake resistant design of earth dams." J.Soil Mech.

Found Div., ASCE, 92, SM1, 13-41.

[13]   Seed, H. B. (1968). "Landslides during earthquakes due to soil liquefaction."Terzaghi Lecture, J. Soil Mech. Found Div., ASCE, 94, SM5, 193-261.

[14]   Sarma, S. K. (1975). "Seismic stability of earth dams and embankments."Geotechnique, 25, 743-761.

[15]   Kramer, S. L., and Smith, M. W. (1997). "Modified newmark model for seismicdisplacements of compliant slopes". J. Geotech. andGeoenv. Engrg., ASCE, 123(7),

635–644.

[16]   H. I. Ling, Y. Mohri, T. Kawabata (1999). "Seismic analysis of sliding wedge: extended Francias-Culmann's analysis." Soil Dynamics and Earthquake Engineering 18 (1999) 387-393.

[17]   Deepankar Choudhury, SomdattaBasu and, Jonathan D Bray (2007) "Behaviour of Slopes under Static and Seismic Conditions by Limit Equilibrium Method", M.ASCE.

[18]   Dr. K. R. Arora, Soil mechanics and foundation engineering.

[19]   Dr. B. C. Punmia, A. K. Jain, A. K. Jain, Soil mechanics and foundations.

[20]   GopalRanjan, A S R Rao, Basic and applied soil mechanics.