Stability Analysis of Pile Group Subjected to Lateral Loading using Pseudo Dynamic Method

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Abstract— Pile foundations, classified under Deep foundations are used when shallow foundations are deemed unsuitable owing to soil or site conditions. Pile foundation system find wide applications in case of expansive soils such as black cotton soil and collapsible soils such as loess or when there is a layer of weak soil near the surface that cannot support the load of the superstructure. Pile foundations are subjected to significant amount of lateral loads besides vertical loads due to seismic forces. Most of the seismic analyses are complicated and require numerical analysis. Furthermore they are aimed at determining stability by considering limiting value of lateral displacement. Very limited seismic studies are conducted considering the pile group as a block. Even in the seismic studies conducted, pseudo-static approach or numerical methods are employed. The objective of this study is to analyze the response of pile group as a block in cohesion less soils subjected to seismic horizontal and vertical forces using pseudo–dynamic approach and compare the results with the existing pseudo-static method. The results of the study help us to determine the relative stability of the pile block against overturning for the avoidance of failure of the foundation system. It is observed that although the stability factor decreases with increase in the value of k_v for a given value of k_h. For the horizontal acceleration coefficient (k_h), the effect is not as pronounced as in the case of horizontal acceleration coefficient. The variation of stability factor is negligible. Relative stability increases with the increase in the value of ϕ. For higher values of ϕ there is an increase in shearing resistance of soil. Increase in density has a negative effect on the stability of the pile block. However as the graph suggest that for all other parameters remaining same, the stability factor is higher in case of pseudo-dynamic method as compared to the pseudo-static for any given value of density. The effect of arching is reflected in pseudo-dynamic analysis where the stability factor becomes constant after a depth of about 10m.

Key words: Pile Block, Lateral Loads, Pseudo-Dynamic Analysis, Stability Factor, Seismic Acceleration Coefficients

I. Introduction

Deep foundations are used when shallow foundations are deemed unsuitable owing to soil or site conditions. Pile foundations are classified under deep foundations. A pile is a slender member made of steel, concrete or wood. It is either driven into soil or formed in-situ by excavating a hole and filling it (with concrete). These are mostly used for the foundations in case of expansive soils such as black cotton soil and collapsible soils such as loess. Pile foundations are used in the cases when there is a layer of weak soil at the surface. This layer cannot support the weight of the building, so the loads of the building have to be transmitted to the underlying layer of stronger soil or rock stratum.

Pile foundations can also be employed in the scenario where a superstructure imposes heavy, concentrated loads, such as in a high rise structure, bridge, or water tank. Pile foundations are capable of taking higher loads than spread footings.

A. *Classification of Pile Foundation*

1) *Based on the Function:*

1) End bearing pile
2) Friction pile
3) 3, Tension or uplift pile
4) Compaction pile
5) Fender pile and dolphins
6) Anchor pile
7) Batter pile

Two of these are of particular importance vis-à-vis the load transfer mechanism of the pile and are described here.

2) *End Bearing Piles*

In end bearing piles, the bottom end of the pile rests on a layer of especially strong soil or rock. The load of the building is transferred through the pile onto the strong layer. In this mechanism, this pile acts like a column. The key principle is that the bottom end rests on the surface which is the intersection of a weak and strong layer. The load therefore bypasses the weak layer and is safely transferred to the strong layer.

3) *Friction Piles*

The load transfer mechanism in friction piles is different. The pile transfers the load of the superstructure to the soil across the entire length of the pile, by friction. In other words, the entire surface of the pile, which is cylindrical in shape, works to transfer the forces to the soil. In a friction pile, the amount of load a pile can support is directly proportional to its length except for the fact beyond a particular depth the arching effect comes into picture.

4) *Based on materials and composition:*

1) Concrete pile
2) Timber pile
3) Steel pile
4) Composite pile

B. *Necessity of Pile Groups*

pile foundations generally consists of a group of piles instead of a single pile, where they perform the dual function of reinforcing the soil, and also of carrying the applied load down to deeper, stronger soil strata. Failure of the pile group may occur either by the failure of individual pile or as failure of the overall block of soil. The supporting Capacity of group of vertically loaded piles can however be considerably less than the sum of the capacities the individual piles comprises the group. Group action in piled foundation could result in failure or excessive settlement even though loading tests made on a single pile have indicated satisfactory capacity in all cases the elastic and consolidation settlements of the group are greater than those of single pile carrying the same working load as that on each pile within the group. This is because the zone of soil or rock which is stressed by the entire group extended to a much greater width and depth than the zone beneath the single pile.

Pile groups are used when l) Column load is heavy 2) Method of installation of piles is by driving.

1) Classification of Pile Groups

Pile groups are classified as:

1) Free standing pile group
2) Piled foundation Free standing pile group: These are used where the foundation soil is expansive in Nature pile cap does not transfer any of the column loads directly to foundation soil. Piled foundation: As pile cap is made to rest on the ground surface, it helps in transfer of part of column load directly to foundation soil these are use in expansive soils.

2) Pile Spacing and Pile Arrangement

In certain types of soil, especially in sensitive clays, the capacity of individual Piles within the closely spaced group may be lower than for equivalent isolated pile. However, because of its insignificant effect, this may be ignored in design. Instead the main worry has been that the block capacity of the group may be less than the sum of the individual piles capacities. As a thumb rule, if spacing is more than 2 - 3 pile diameters, then block failure is most unlikely it is vital importance that pile group in friction and cohesive soil arranged that even distribution of load in greater area is achieved. Large concentration of piles under the center of the pile cap should be avoided. This could lead to load concentration resulting in local settlement and failure in the pile cap. Varying length of piles in the same pile group may have similar effect for pile load up to 300kN the minimum distance to the pile cap should be 100mm. For higher than 300kN, this distance should be more than 150 mm. In general, the following formula may be used in pile spacing:

As per IS CODE End-bearing

S = 2.5 d Friction piles: S = 3.0 d Piles in loose sand: S = 2.0 d where

d = assumed pile diameter

s = pile centre to centre distance (spacing)

II. METHODOLOGY

A. Basic Introduction to the Pseudo-dynamic Approach

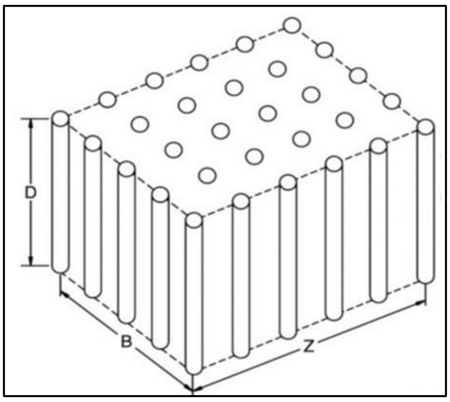

Fig. 3.1: Schematic representation of a typical pile block

A typical pile block has been demonstrated in the figure. For the present study a pile block consisting of 25 piles (5 piles each in 5 rows) is considered. The diameter of pile is 300 mm and spacing of 2d is considered.

This approach considers the failure of the pile group as a composite block. This results in the formation of active and passive failure wedge on either side of the block. The seismic forces acting on the failure wedges have been computed and stability analysis with respect to moment has been conducted using pseudo-static and pseudo-dynamic approach. As the piles are floating the failure mechanism is considered in a manner that the pile block rotatesabout the centre of the pile cap in the vertical plain.The failure surfaces are assumed to be planar, an assumption which holds good for lower value of wall friction.

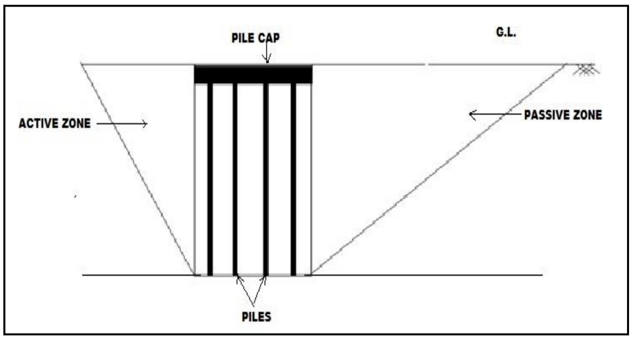

Fig. 3.2: Schematic representation of the development of active and passive wedges in dry condition using a linear rupture surface

The pseudo-dynamic approach considers finite shear wave velocity within the backfill material as proposed by Steedman and Zeng (1990). The soil medium is considered to be homogeneous isotropic and elastic throughout the depth of the pile. It is assumed that the shear modulus (G) is constant with the depth of pile block throughout the depth. But both the phase and the magnitude of accelerations are varying along the depth of the wall. The magnitude are varying linearly along the depth of the wall

The primary and the shear wave velocity depend on medium, which in this case is the soil and can be determined as:

$$V_p = \sqrt{\frac{G(2-2\nu)}{\rho(1-2\nu)}} \quad \text{(Eq.3.1)}$$

υ =Poisson's ratio

$$V_s = \sqrt{\frac{G}{\rho}} \quad \text{(Eq.3.2)}$$

Das (1993) suggested that $V_p/V_s = 1.87$, which is accepted by most researchers. Further as suggested by Kramer (1996),$T=2\pi/\omega=4H/V_s$, whereω = the angular frequency is considered in the analysis.

It must be noted that in the present analysis it is assumed that the rupture surface in the active zone extends to the ground surface. The Coulomb Theory of earth pressure assumes that the surface of sliding or failure is a plane. This assumption introduces an error as it over-estimates the value of passive earth pressure. For the active case the error introduced is small. If the angle of wall friction is low the failure surface is almost plane. However, if is high, the passive failure plane deviates considerably from Coulomb's assumption, which predicts unrealistically high passive pressures. Terzaghi explains that the curved failure surface is a result of an increase in the downward tangential force, which ultimately results in the increase in vertical pressure close to the wall.

In this analysis, similar method has been adopted for pile group behaving as a block in cohesionless soils considering a planar failure wedge in both active and passive wedge with angles that the wedge makes with the horizontal as α and α', respectively (as shown in Fig.1). If the base of the wall is subjected to harmonic horizontal seismic acceleration of amplitude $\alpha_h g$, where g is the acceleration due to gravity and harmonic vertical seismic acceleration of amplitudeαv g, the acceleration at any depth z and time t, below the top of the wall can be expressed as,

$$a_h(z,t) = \left\{1 + \frac{f_a - 1}{H}(H - z)\right\}\alpha_h \sin\omega\left(t - \frac{H - z}{V_s}\right) \quad \text{(Eq.3.3)}$$

where

H = height of the total height of the pile.

and

$$a_v(z,t) = \left\{1 + \frac{f_a - 1}{H}(H - z)\right\}\alpha_v \sin\omega(t - \frac{H - z}{V_p}) \quad \text{(Eq.3.4)}$$

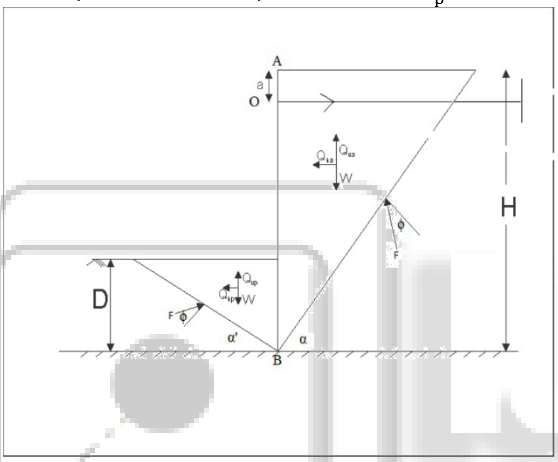

Fig. 3.3: Schematic representation of the static and seismic forces acting on active and passive wedges in dry condition using a linear rupture surface

III. PSEUDODYNAMIC ANALYSIS USING PLANAR FAILURE SURFACE

Consider a wall AB of height H .The height of dredge line above the base of the wall isd. As discussed earlier the system will have an active earth pressure distribution behind the retaining wall and passive earth pressure distribution on the upstream side (as shown in Fig.2). The anchor is fixed at point O at a depth of 'a' below the ground surface.

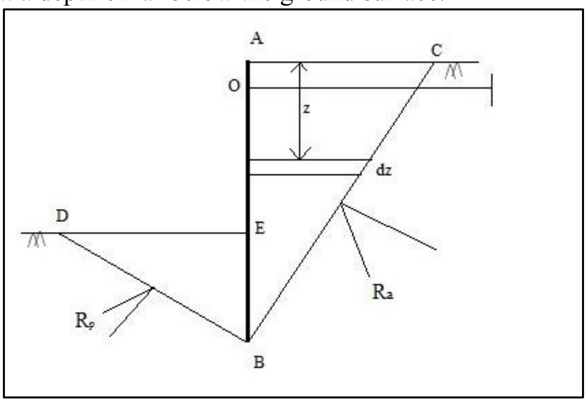

Fig. 3.4: Vertical strip in active zone at depth z.

A. *Analysis of Active Wedge ABC*

The active wedge consists of seismic horizontal and seismic vertical force,the force due to the weight of the wedge and a reaction force. The resultant of these forces is the seismic active earth pressure.At any depth z below the ground surface consider a vertical strip of thickness dz.The mass of the strip will be

$$m = \frac{\gamma(H - z)}{g\tan\alpha}dz \quad \text{(Eq.3.5)}$$

Horizontal seismic force in the active wedge

$$Q_{ha} = \int_0^H m * a_h \quad \text{(Eq.3.6)}$$

As already mentioned

$$a_h(z,t) = \left\{1 + \frac{f_a - 1}{H}(H - z)\right\}\alpha_h \sin\omega\left(t - \frac{H - z}{V_s}\right) \quad \text{(Eq.3.7)}$$

Horizontal force in the active wedge

$$Q_{ha} = \int_0^H \{1 + \frac{f_a - 1}{H}(H - z)\alpha_h \sin\omega(t - \frac{H - z}{V_s})\frac{\gamma(H - z)}{g\tan\alpha}dz \quad \text{(Eq. 3.8)}$$

$$Q_{ha} = \frac{k_1}{4\pi^2}[2\pi\lambda H\cos\omega\tau + \lambda^2(\sin\omega\tau - \sin\omega t)] + \frac{Fk_1}{8\pi^3}[4\pi^2\lambda H^2\cos\omega\tau + 4\pi\lambda^2 H\sin\omega\tau + \lambda^3(\cos\omega t - \cos\omega\tau)] \quad \text{(Eq.3.9)}$$

where

$$k_1 = \frac{\gamma\alpha_h}{g\tan\alpha}$$

$$F = \frac{f_a - 1}{H}$$

$$\tau = t - H/Vs$$

ω =angular frequency = 2π/T

Moment due to Horizontal seismic force in active wedge (This force acts at a depth (z-a) from O)

$$M_{Qha} = \int_0^H \{1 + \frac{f_a - 1}{H}(H - z)\alpha_h \sin\omega\left(t - \frac{H - z}{V_s}\right)\frac{\gamma(H - z)}{g\tan\alpha} z\,dz \quad \text{(Eq3.10)}$$

$$M_{Qha1} = \frac{k_1}{8\pi^3}[-2\pi\lambda^2(H\sin\omega t + H\sin\omega\tau) + 2\lambda^3(\cos\omega\tau - \cos\omega t)] \quad \text{(Eq.3.11)}$$

$$M_{Qha2} = \frac{Fk_1}{16\pi^3}[\ 4\pi^2\lambda^2 H^2\sin\omega\tau + 4\pi\lambda^3\{2H\cos\omega\tau + H\cos\omega t\} + 6\lambda^4(\sin\omega\tau - \sin\omega t)] \quad \text{(Eq.3.12)}$$

$$M_{Qha} = M_{Qha1} + M_{qha2} \quad \text{(Eq.3.13)}$$

The vertical seismic force in the active wedge

$$Q_{va} = \int_0^H m * a_v \quad \text{(Eq.3.14)}$$

This on substitution of values becomes:

$$Q_{va} = \int_0^H\{1 + \frac{f_a - 1}{H}(H - z)\alpha_v \sin\omega(t - \frac{H - z}{V_p})\frac{\gamma(H - z)}{g\tan\alpha}dz \quad \text{(Eq.3.15)}$$

$$Q_{va} = \frac{k_2}{4\pi^2}[2\pi\eta H\cos\omega\psi + \eta^2(\sin\omega\psi - \sin\omega t)] + \frac{Fk_2}{8\pi^3}[4\pi^2\eta H^2\cos\omega\psi + 4\pi\eta^2 H\sin\omega\psi + \eta^3(\cos\omega t - \cos\omega\psi)] \quad \text{(Eq.3.16)}$$

where

$$\eta = TV_P$$

$$\psi = t - {}^{H}/_{V_p}$$

$$k_2 = \frac{\gamma\alpha_v}{g\tan\alpha}$$

The moment produced due to vertical acceleration in the active wedge is computed to be:

$$M_{Qva}=\frac{K_2}{16\pi^3\tan\alpha}\ [4\pi^2\eta\ H^2\cos\omega\psi\ +4\pi\eta^2\ H\sin\omega\psi\ -2\eta^3\ \{\cos\omega\psi\ -\cos\omega t\}\ ] +\frac{FK_2}{32\pi^4\tan\alpha}[\ 8\pi^3\eta H^3\cos\omega\psi\ +12\pi^2\eta^2)H^2\sin\omega\psi - 12\pi\eta^3 H\cos\omega\psi\ +\ 6\eta^4(\sin\omega t-\sin\omega\psi)\ ] \quad \text{(Eq.3.17)}$$

B. Weight of the Wedge

$$W\ =\frac{1}{2}\gamma \quad \text{(Eq.3.18)}$$

Moment due to weight

$$M_W=\frac{1}{6}\gamma\frac{H^3}{\tan^2\alpha} \quad \text{(Eq.3.19)}$$

Reaction force

$$R_a=\frac{W-Q_v-Q_h\tan\delta}{\sin(\alpha-\phi)\tan(\delta)+\cos(\alpha-\phi)} \quad \text{(Eq.3.20)}$$

Moments due to reaction force

$$M_{Rh}=R_a\sin(\alpha-\phi)\left(\frac{2}{3}H-a\right) \quad \text{(Eq.3.21)}$$

$$M_{Rv}\ =\frac{R_a\cos(\alpha-\phi)H\cot\alpha}{3} \quad \text{(Eq. 3.22)}$$

C. Analysis of the Passive Wedge

The passive wedge ABD along BD makes an angleα' with the horizontal. The height of dredge line above the base is taken as'd'. The Horizontal seismic force can be written as:

$$Q_{hp}=\int_{H-d}^{H}\left\{1+\frac{f_a-1}{H}(H-z)\right\}\alpha_h\sin\omega(t-\frac{H-z}{V_s})\frac{\gamma(H-z)}{g\tan\alpha'}dz \quad \text{(Eq.3.23)}$$

$$Q_{hp}=\frac{k_1'}{4\pi^2}[2\pi\lambda d\cos\omega\tau'+\lambda^2(\sin\omega\tau'-\sin\omega t)]+\frac{Fk_1'}{8\pi^3}[4\pi^2\lambda d^2\cos\omega\tau'+4\pi\lambda^2 d\sin\omega\tau'+2\lambda^3(\sin\omega\tau'-\sin\omega t)] \quad \text{(Eq.3.24)}$$

Moment due to Horizontal seismic force in passive wedge is as under:

The limit of integration is H-d to H

$$M_{Qhp}=\int_0^H\{1+\frac{f_a-1}{H}(H-z)\alpha_h\sin\omega(t-\frac{H-z}{V_s})\frac{\gamma(H-z)}{g\tan\alpha'}z\ dz \quad \text{(Eq.3.25)}$$

$$M_{Qhp1}=\frac{k_1'}{8\pi^3}[\quad -2\pi\lambda^2(H\sin\omega t+H\sin\omega\tau')+2\lambda^3(\cos\omega\tau'-\cos\omega t)] \quad \text{(Eq.3.26)}$$

$$M_{Qhp2}\ =\frac{Fk_1'}{16\pi^4}[4\pi^2\lambda^2 H^2\sin\omega\tau'+4\pi\lambda^3(H\cos\omega t+2H\cos\omega\tau')+6\lambda^4(\sin\omega\tau'-\sin\omega t)] \quad \text{(Eq.3.27)}$$

$$M_{Qhp}=M_{Qhp1}+M_{Qhp2} \quad \text{(Eq.3.28)}$$

where

$$\tau'=t-D/V_s$$

$$k_1'=\frac{\gamma\alpha_h}{g\tan\alpha'}$$

Vertical seismic force in active wedge

$$Q_{vp}=\int_0^H\{1+\frac{f_a-1}{H}(H-z)\alpha_v\sin\omega(t-\frac{H-z}{V_p})\frac{\gamma(H-z)}{g\tan\alpha'}dz \quad \text{(Eq.3.29)}$$

$$Q_{vp}=\frac{k_2'}{4\pi^2}[2\pi\eta d\cos\omega\psi'+\eta^2(\sin\omega\psi'-\sin\omega t)]+\frac{Fk_2'}{8\pi^3}[4\pi^2\eta d^2\cos\omega\psi'+4\pi\eta^2 d\sin\omega\psi'+2\eta^3(\cos\omega t-\cos\omega\psi')] \quad \text{(Eq.3.30)}$$

The moment due to this force

$$M_{Qvp}=\int_{H-D}^{H}\{1+\frac{f_a-1}{H}(H-z)\}\alpha_v\sin\omega(t-\frac{H-z}{V_p})\frac{\gamma(H-z)}{g\tan\alpha'}\frac{H-z}{2\tan\alpha'}dz \quad \text{(Eq.3.31)}$$

$$M_{Qvp1}=\frac{k_2'}{16\pi^3\tan\alpha'}[4\pi^2\eta d^2\cos\omega\psi'+4\pi\eta^2 d\sin\omega\psi'+2\eta^3(\cos\omega t-\cos\omega\psi')] \quad \text{(Eq.3.32)}$$

$$M_{Qvp2}=\frac{Fk_2'}{32\pi^4\tan\alpha'}[8\pi^3\eta d^3\cos\omega\psi'-12\pi^2\eta^2 d^2\sin\omega\psi'-12\pi\eta^3 d\cos\omega\psi'+6\eta^4(\sin\omega t-\sin\omega\psi')] \quad \text{(Eq.3.33)}$$

$$M_{Qvp}=M_{Qvp1}+M_{Qvp2} \quad \text{(Eq.3.34)}$$

where

$$\psi'=t-{}^{d}/_{V_P}$$

Weight of passive wedge

$$W=\frac{1}{2}\gamma\frac{d^2}{\tan\alpha'} \quad \text{(Eq.3.35)}$$

$$M_{Wp}=\frac{1}{6}\gamma\frac{d^3}{\tan^2\alpha'} \quad \text{(Eq.3.36)}$$

$$R_p=\frac{W-Q_{vp}-Q_{hp}\tan\delta}{\cos(\alpha'+\phi)-\sin(\alpha'+\phi)\tan\delta} \quad \text{(Eq.3.37)}$$

$$M_{Rph}=R_P\sin(\alpha'+\phi)(H-\frac{d}{3}-a) \quad \text{(Eq.3.38)}$$

$$M_{Rpv}=R_p\cos(\alpha'+\phi)\frac{d}{3}\cot\alpha' \quad \text{(Eq.3.39)}$$

D. Weight of the block

The weight of block itself is computed as

$$W_{block}=H[\gamma_{soil}A_pN+\gamma_{conc}(A-A_p)] \quad \text{(Eq.3.40)}$$

Where

A= Area of pile Group

A_p=Area of a single pile

N=no. of piles

H=height of piles

γ_{soil}=unit weight of soils

γ_{conc}=unit weight of concrete

The weight is assumed to be acting along the centre of the pile. The moment due to the weight of the block is considered as a resisting moment.

E. Factor of Safety

The factor of safety is computed considering the most critical case, i.e. when the chances of failure are most likely. For this the following combination is considered.

1) Driving Moments:

1) $M_{Q_{ha}}$
2) M_{W_p}
3) $M_{R_{ha}}$
4) $M_{Q_{hp}}$
5) $M_{Q_{vp}}$
6) $M_{R_{hp}}$

Stabilizing Moments:

1) $M_{Q_{va}}$
2) $M_{R_{hp}}$
3) M_{W_a}
4) M_{vp}
5) M_{W_b}

The factor of safety for overturning about the anchor is thereby computed as:

$$F_S = \frac{M_{Q_{va}}+M_{R_{hp}}+M_{W_a}+M_{vp}+M_{W_b}}{M_{Q_{ha}}+M_{W_p}+M_{R_{ha}}+M_{Q_{hp}}+M_{Q_{vp}}+M_{R_{hp}}} \quad \text{(Eq. 3.41)}$$

The stability factor is then computed as:

$$\text{Stability Factor} = \frac{F_S(\text{for the given parameters})}{F_S(\text{when no lateral load is applied})} \quad \text{(Eq.3.42)}$$

IV. RESULT & DISCUSSION

To avoid shear fluidization the combination of k_h and $k_v$should be carefully chosen in accordance with the given condition: ϕ ≥ tan ⁻(kh/1-kv). Results are presented in graphical form with respect to depth (d) of the wall.

- In the parametric study, required depth of the sheet piles is determined for a factor of safety one. However, the obtained depth can be increased by 20% - 30% to provide factor of safety more than one.
- Variation of parameters considered is as:

ϕ (in degrees) =20, 25, 30, 35, 40;
δ = 0, 0.25 ϕ, 0.5 ϕ, 0.75 ϕ, ϕ;
k_h =0, 0.1, 0.2, 0.3, 0.4, 0.5;
k_v= 0, 0.5 k_h, k_h;
f_a =1, 1.2, 1.4;
V_s =100m/s;
V_p =187m/s.

- As suggested by various researchers, H/ λ =0.3 and H/ η=0.17.
- In the planar failure surface, the results have to be optimized for α and α’.
- The comparison in this analysis is based on a parameter called as stability factor. Stability factor is a relative stability as compared to the case when no lateral load is applied for a standard dimension considered in this analysis.
- The value of vertical acceleration coefficient is taken half of the horizontal seismic acceleration value where not specified.
- The depth of piles is taken as 10m for the standard case.

A. *Effect of k_h*

Figure 7 shows the effect of k_h on the required depth of the sheep piles in dry condition. It is observed that the depth of wall increases with increase in the value of k_h.The incremenet is linear for planar failure wedge case. The result clearly reveals that linear failure wedge underestimates the depth of pile as it over-estimates the passive resistance and underestimates the active resistance. It is also important that values are comparable for lower values of k_h, but as the value increases the difference between the values obtained from planer and curved failure surface increases. Figure 8 the effect of k_h on the required depth of the sheep piles in submergerd condition. Similar behavior is observed in case of submerged condition. Figure 9 shows the effect of kv on the depth of the depth of the sheet pile. It is observed that, the variation in k_v also results in affecting the seismic stability. However, the effect of k_h is more significant as compared to the effect of k_v on the required depth of the sheet pile. No difference is observed between the required depth obtained from pseudo-static planar failure surface and pseudo-dynamic planar failure surface as k_v/k_h value changes. It is observed that there is an increase in depth requirement by about 28% in PDS, 40% in PSC and 46% in PDC analysis on increasing k_h from 0 to 0.2.

Value of K_h	Pseudo-static Method	Pseudo-dynamic method
0	1	1.000
0.1	0.978	0.923
0.2	0.894	0.857
0.3	0.748	0.803
0.4	0.587	0.761

.Table 1: Values of Stabilty Factor for different values of k_h

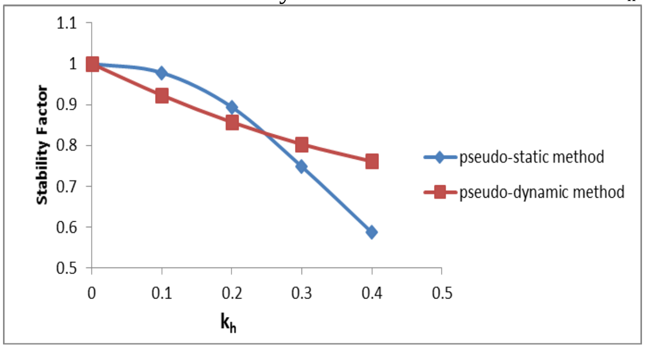

Fig. 4: Plot of Stability factor vs horizontal acceleration coefficient (k_h)

The effect of vertical acceleration coefficient is tabulated as below. The value of horizontal acceleration coefficient is taken as 0.3.

It can be clearly observed that the relative stability of the pile block decreases with increase in the value of horizontal acceleration coefficient. The stability factor decreases by about 11% when the k_h increases to 0.2 in case of pseudo-static method. Correspondingly the decrease in case of pseudo-dynamic approach is slightly higher (14%). However as the seismic acceleration increases (to 0.4) the pseudo-static method shows a considerable decline in the stability (about 41%) as compared to pseudo-dynamic method (24%). Thus for higher values of k_h the pseudo-static method gives a lower value of stability factor.

B. *Effect of k_v*

K_v	Pseudo-static Method	Pseudo-dynamic method
0	0.758	0.842
0.1	0.756	0.813
0.2	0.737	0.786
0.3	0.687	0.743

.Table 2: Values of Stability Factor for different values of k_v

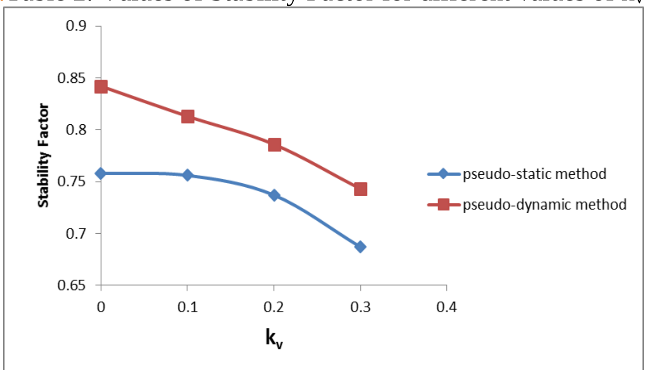

Fig. 5: Plot of Stability factor vs vertical acceleration coefficient (k_v)

Thus it can be observed that although the stability factor decreases with increase in the value of kv for a given value of kh the effect is not as pronounced as in the case of horizontal acceleration coefficient. The variation of stability factor is negligible as kv increases from 0 to 0.1 (about 0.3%). It decreases to about 2.8% for Kv=.2 and about 9.4 for kv=0.3. For the pseudo-dynamic case the decrease follows a similar trend. The stability factor decreases by 3.4%, 6.7% and 11.8% for an increase in kv from to 0 to 0.1,0.2 and 0.3 respectively.

C. *Effect of ϕ*

Figure 10 shows the effect of ϕ on the stability factor. The results clearly suggest that the relative stabilty increases with the increase in the value of ϕ. This can be attributed to the increase in shearing resistance of soil. This comparison is made for a constant value of δ to study the effect due to change in friction angle only. It is observed that on increasing ϕ from 20 to 30 degrees, the stability factor increases by about 20.2 % in pseudo-static analysis while the corresponding change in PD is 17% and that in PSC is 41%.A further increase in the value of ϕ to 40 deg results in an increase of about 34.2% in pseudo-static method, while the corresponding changes in pseudo-dynamic method is about 59%.

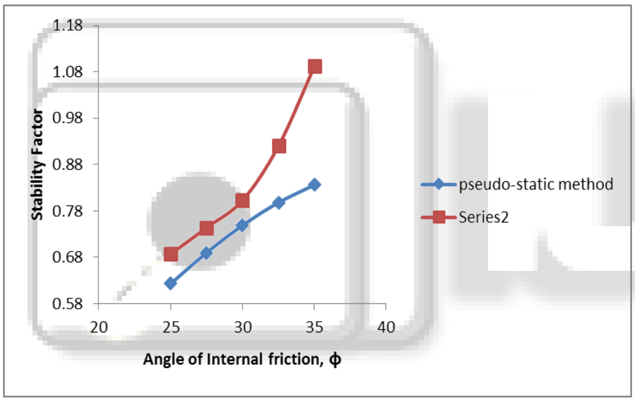

Fig. 6: Plot of Stability factor vs angle of internal friction (φ)

φ	Pseudo-static Method	Pseudo-dynamic method
25	0.623	0.687
27.5	0.690	0.744
30	0.749	0.803
32.5	0.798	0.920
35	0.836	1.092

Table 3: Values of Stability Factor for different values of angle of internal friction

D. *Effect of Density*

The seismic horizontal force acting on the pile block is a function of density of soil and therefore increase in density has a negative effect on the stability of the pile block. Thus for both pseudo-static and pseudo-dynamic method the stability factor decreases with decrease in density of the soil. However as the graph suggest that for all other parameters remaining same, the stability factor is higher in case of pseudo-dynamic method as compared to the pseudo-static for any given value of density.

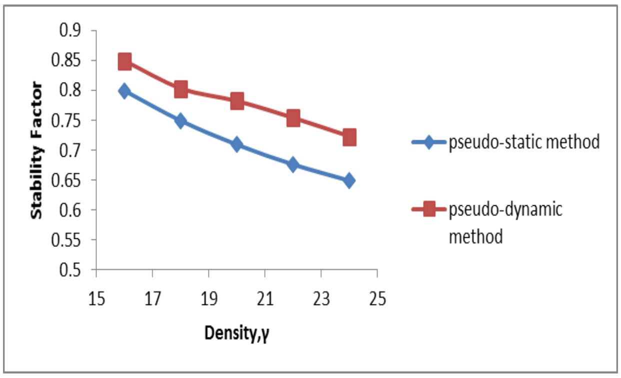

Fig. 7: Plot of Stability factor vs density

γ	Pseudo-static Method	Pseudo-dynamic method
16	0.799	0.849
18	0.749	0.803
20	0.709	0.782
22	0.676	0.754
24	0.649	0.723

Table 4: Values of Stability Factor for different values of density

E. *Effect of Depth*

DEPTH	Pseudo-static Method	Pseudo-dynamic method
8	1.340	1.457
9	0.939	1.023
10	0.749	0.803
11	0.613	0.806
12	0.512	0.80

Table 5: Values of Stability Factor for different values of depth of pile

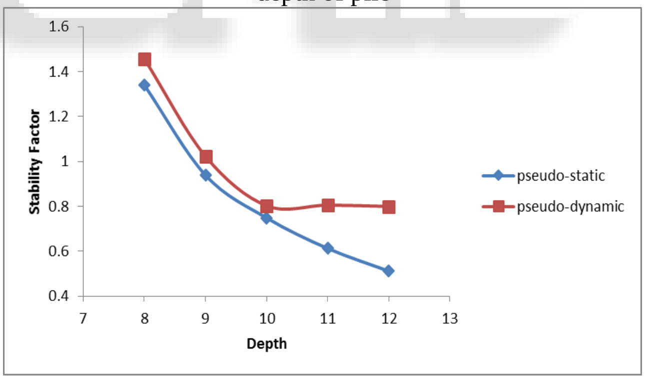

Fig. 8: Stability Factor vs Depth

The key observation in analyzing the effect of depth of pile is that arching effect is reflected in pseudo-dynamic analysis where the stability factor becomes constant after a depth of about 10m.

V. Conclusion

In the present study the effect of seismic forces on a pile group was analyzed using pseudo-static and pseudo-dynamic method where the block behaviour of pile is considered. The failure surfaces were considered to be planar. The following observations were made.

There is a clear trend in decrease in the relative stability of the pile block with increase in the value of horizontal acceleration coefficient. The stability factor decreases by about 11% when the k_h increases to 0.2 in case of pseudo-static method. Correspondingly the decrease in case of pseudo-dynamic approach is slightly higher (14%). However as the seismic acceleration increases (to 0.4) the pseudo-static method shows a considerable decline in the stability (about 41%) as compared to pseudo-dynamic method (24%). Thus for higher values of k_h the pseudo-static method gives a lower value of stability factor leading to uneconomic design.

Thus it can be observed that although the stability factor decreases with increase in the value of k_v for a given value of k_h the effect is not as pronounced as in the case of horizontal acceleration coefficient. The variation of stability factor is negligible.

Relative stability increases with the increase in the value of ϕ. For higher values of ϕ there is an increase in shearing resistance of soil. It can be interpreted that in the pseudo-the pseudo-static method gives a lesser value of factor of safety. Thus there is a possibility that analysis using pseudo-static method leads to uneconomical design. On the other hand for lower values of $k_h$the pseudo-static method leads to unsafe design

Increase in density has a negative effect on the stability of the pile block. Thus for both pseudo-static and pseudo-dynamic method the stability factor decreases with decrease in density of the soil. However as the graph suggest that for all other parameters remaining same, the stability factor is higher in case of pseudo-dynamic method as compared to the pseudo-static for any given value of density. However it must be considered that for loose soils, where the water table is relatively higher the susceptibility to liquefaction will come into picture.

The key observation in analyzing the effect of depth of pile is that arching effect is reflected in pseudo-dynamic analysis where the stability factor becomes constant after a depth of about 10m.

It must be emphasized, however that there exists a scope of improvement in the proposed model. In the present work the reaction forces are assumed to be acting at 1/3 of height of the wedge. This assumption holds good for the values considered in the work as suggested by researchers. A more comprehensive study can be done by incorporating the effects of sinusoidal variations of seismic waves on the point of location of these reaction forces so that the model can be used for a wide range of parameters.

An improvement in the degree of accuracy in determination of the results can be achieved by using optimization software. Furthermore this analysis can be extended to the seismic stability of anchors themselves, so that the seismic stability of the system in entirety can be analyzed.

There is also the scope of incorporating the effects of pile cap resistance which has not been considered in the current analysis.

The results should be interpeted with caution as these are not the actual factor of safety but relative to the case when no lateral loads. Therfore the analysis first need to be condutcted for static case to determine the actual factor of safety.

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