

Seismic Stability Analysis of Reinforced Soil Wall Using Horizontal Slice Method

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Abstract The seismic stability of a reinforced soil wall has been analyzed using the horizontal slice method considering the pseudo-static seismic forces. The effect of various parameters including wall inclination angle, angle of internal friction of soil, horizontal seismic loading, cohesion of the backfill and surcharge loading has been examined. It is found that the stability of a reinforced soil wall is largely affected by the horizontal seismic forces. The tensile resistance which has to be mobilized by the reinforcement to maintain the stability of wall, increases with an increase in surcharge, horizontal seismic forces, whereas the same decreases with an increase in the cohesion of soil. The normalized geosynthetic tensile reinforcement force is used for comparing the effect of different parameters.

Keywords Reinforced soil wall · Pseudo-static analysis · Horizontal slices method · Internal stability · Geosynthetics

List of Symbols

Basic SI units are given in parentheses

a_h Amplitude of horizontal seismic acceleration (m/s²)

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b_i Length of base of slice (m)
 c Cohesion of soil (N/m²)
 F Factor of safety (dimensionless)
 H Height of wall (m)
 H_i Horizontal inter-slice force acting on top of i th slice per meter length of wall (N/m)
 K Normalized geosynthetic tensile reinforcement force (dimensionless)
 K_{St} Normalized geosynthetic tensile reinforcement force for static case ($k_h = 0.0$, dimensionless)
 K_{Dy} Normalized geosynthetic tensile reinforcement force for seismic case ($k_h > 0.0$, dimensionless)
 K_{Ratio} Ratio of K for seismic case to that for static case (dimensionless)
 k_h Horizontal seismic coefficient (dimensionless)
 l_i Length of horizontal border of slices (m)
 m Number of reinforcement layers (dimensionless)
 m_i Mass of elemental i th slice (kg)
 N Number of slices (dimensionless)
 N_i Normal force upon base of slice per meter length of wall (N/m)
 q_{hi} Horizontal inertia force due to seismic acceleration acting at layer i per meter length of wall (N/m)
 S_i Shear force upon base of slice per meter length of wall (N/m)
 W_i Weight of slice per meter length of wall (N/m)
 α_i Angle of the base of elemental slice (°)
 β Slope angle (°)
 γ Unit weight of soil (kN/m³)
 τ_f Failure shear stress (N/m²)
 τ_r Required shear stress (N/m²)
 ϕ Angle of internal friction of soil (°)

Introduction

Reinforced soil structures are extensively used due to its versatility, cost effectiveness and ease of construction. The reinforced earth technique is particularly useful for (i) congested urban areas having a scarcity of land and (ii) approach roads to bridges effectively where the unreinforced retaining walls may cost higher and require more ground space. Earlier the designs of retaining walls were based on Rankine's or Coulomb's earth pressure theories. Okabe [1] and Mononobe and Matuso [2] were the pioneers to extend Coulomb's theory to incorporate seismic conditions using pseudo-static approach for cohesionless soils. This method is known as Mononobe–Okabe method [3]. Shukla [4] extended the Mononobe–Okabe theory for soil having both cohesion and friction (c – ϕ soil). Bathurst and Cai [5] studied stability analysis of geosynthetic reinforced segmental walls under seismic loading conditions considering the pseudo-static approach. A parametric study of factor of safety related to internal, external and facing failure modes and forces for reinforced segmental retaining walls were reported. Ling et al. [6] proposed a design methodology for the geosynthetic reinforced soil structures subjected to seismic loading based on pseudo-static limit equilibrium analysis, and examined the influence of different slope angles, horizontal seismic acceleration and soil properties on the design of reinforced soil structures.

Shahgholi et al. [7] introduced the horizontal slice method (HSM) to determine the internal stability of reinforced soil walls. This study considered the vertical equilibrium of each slice and horizontal equilibrium of the whole wedge in order to derive an equation to determine the required tensile force that has to be carried by the geosynthetic reinforcement. Nimbalkar et al. [8] determined the internal stability of reinforced soil-walls by considering the pseudo-dynamic method. The backfill material was considered cohesionless and free draining. The effect of various parameters like vertical and horizontal seismic coefficients, angle of internal friction of soil on the stability of reinforced soil wall was studied.

Nouri et al. [9] analyzed the seismic stability of reinforced soil for determining the forces in the reinforcement by developing formulations depending upon the nature and number of the assumptions and equations. The effect of various parameters like wall inclination and angle of internal friction of soil, horizontal seismic coefficient on the required tensile strength and reinforcement length was studied. Nouri and Fakher [10] further extended the study to include a log-spiral failure surface in reinforced soil wall. The effect of vertical seismic coefficient was also included in the detailed parametric study. Nouri et al. [11] had investigated the influence of amplification and

magnitude of ground acceleration on the seismic stability of the reinforced-soil wall.

Shekarian and Ghanbari [12] used the HSM to determine the seismic earth pressure on rigid retaining walls for both unreinforced and reinforced walls using the pseudo-dynamic approach and assuming the failure surface to be linear. Reddy et al. [13] conducted a parametric study to examine the effect of different reinforcement and backfill parameters on the factor of safety of the reinforced soil wall subjected to the pseudo-static seismic loads.

A critical review of literature shows that a very few studies have been reported for the stability analysis of reinforced soil wall considering the effect of wall inclination [9–11]. However, no study seems to be available to quantify the effect of cohesion and surcharge on the stability of reinforced soil walls. In view of this observation, the present study is directed to conduct a seismic stability analysis of the reinforced soil-wall using the HSM and considering the pseudo static forces. Shahgholi et al. [7] considered the effect of angle of internal friction and horizontal seismic coefficient using HSM. The proposed analysis, which is based on the extension of the method suggested by Shahgholi et al. [7], considers the effect of wall inclination, cohesion of the backfill and surcharge loading in addition to the two original parameters as angle of internal friction and horizontal seismic coefficient. The parametric study has been carried out to investigate the effect of various parameters like wall inclination, angle of internal friction and cohesion of soil, horizontal seismic loading and surcharge on backfill.

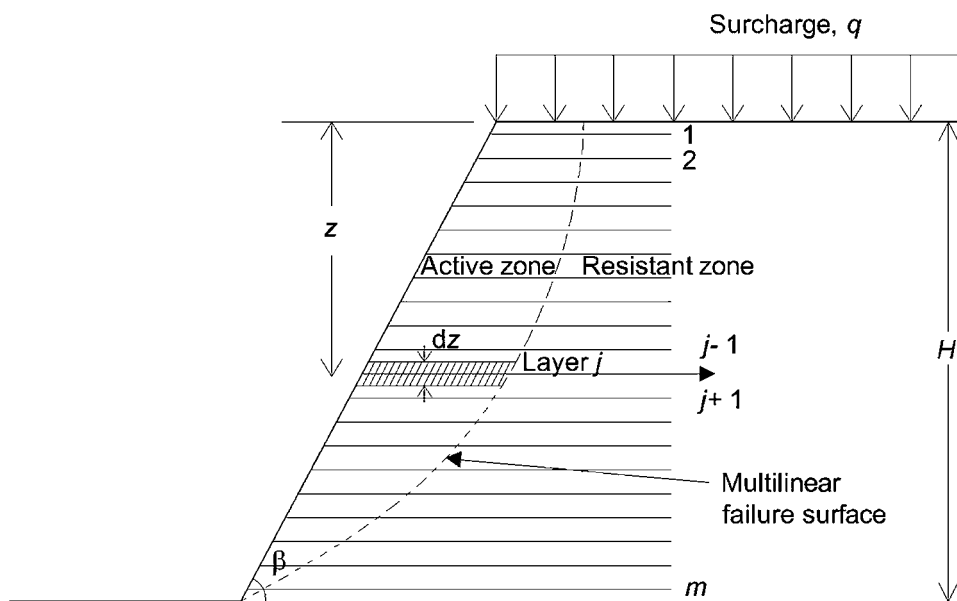
Method of Analysis

Figure 1 shows a geosynthetic reinforced wall of height H with a rigid sloping face inclined to the horizontal at an angle β . The backfill material is assumed to be homogeneous with total unit weight γ , angle of internal friction ϕ and cohesion c . A uniform surcharge loading of q per unit area is assumed to act on the top of the backfill. The reinforcement is laid horizontally. The tensile force in the geosynthetic layer is conservatively assumed to act horizontally [14].

The simplified formulation of HSM proposed by Shahgholi et al. [7] is used in the present analysis. A multi-linear failure surface is assumed and the failure wedge is divided into number of horizontal slices. The HSM overcomes the difficulties associated with the vertical slice method, especially with no interslice forces being developed by the action of reinforcement [7].

The following assumptions are made in the present study:

Fig. 1 Configuration of model reinforced soil wall considered in the analysis



- (1) The vertical stress on a backfill soil element is equal to the overburden pressure.
- (2) The soil below the toe of the wall is strong enough so that the failure surface does not pass below the toe of the slope.
- (3) The factor of safety (F) is defined as the ratio of the available shear resistance to the required shear resistance along the failure surface.
- (4) The factors of safety is equal for all the slices ($F = 1$).

Simplified Formulation

If n is the number of slices, then $2n + 1$ unknowns and $2n + 1$ equations are involved in the simplified formulation of the HSM. The unknowns are normal forces upon the base of each slice (n unknowns), shear forces upon the base of each slice (n unknowns) and the required tensile force (one unknown). The required equations are derived from the vertical equilibrium for each slice (n equations), S_i for each slice (n equations) and the horizontal equilibrium of the whole wedge (one equation).

The required tensile strength is determined from the pseudo-static method of analysis and the HSM [7] considering a multi linear failure surface.

For a thin elemental slice of thickness dz at a depth z , as shown in Figs. 1 and 2, the weight of the elemental i th slice is given by

$$W_i = \gamma dz \left(\frac{l_i + l_{i+1}}{2} \right). \tag{1}$$

Figure 2 shows the forces acting on single horizontal slice containing reinforcement. Only critical directions of q_{hi} acting on the elemental slice are shown in Fig. 2.

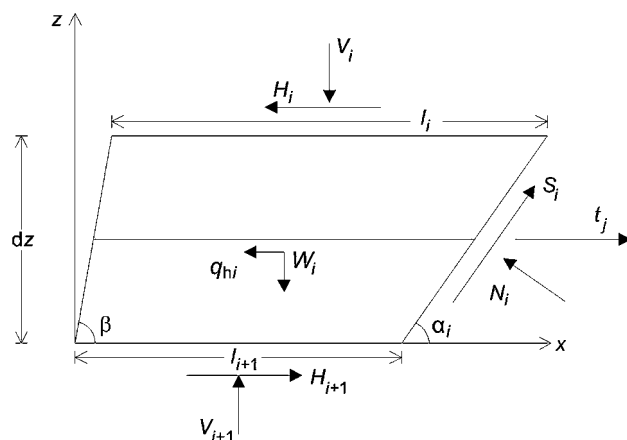


Fig. 2 Forces acting on the horizontal i th slice

The total horizontal inertia force q_{hi} acting on the i th slice can be expressed as

$$q_{hi} = m_i a_h = W_i k_h. \tag{2}$$

Considering the vertical force equilibrium for each slice,

$$V_{i+1} - V_i - W_i + S_i \sin \alpha_i + N_i \cos \alpha_i = 0, \tag{3}$$

where V_i and V_{i+1} are the vertical inter-slice forces calculated by the integration of overburden pressures on the horizontal border of the slice in a similar method to that used by Atkinson [15] and Shahgholi et al. [7].

The required shear stress,

$$\tau_r = \frac{\tau_f}{F}. \tag{4}$$

Thus, the shear force at the base of slice,

$$S_i = \frac{1}{F} (cb_i + N_i \tan \phi). \tag{5}$$

Considering the horizontal force equilibrium $\sum F_x = 0$ for the whole wedge (for m layers),

$$\sum_{j=1}^m t_j + \sum_{i=1}^n S_i \cos \alpha_i - \sum_{i=1}^n N_i \sin \alpha_i - \sum_{i=1}^n W_i K_h = 0. \quad (6)$$

Substituting for S_i from Eq. (5) into Eq. (3),

$$N_i = \frac{V_i - V_{i+1} + W_i - c_m b_i \sin \alpha_i}{\tan \phi_m \sin \alpha_i + \cos \alpha_i}, \quad (7)$$

where $c_m = \frac{c}{F}$ is the mobilized cohesion and ϕ_m is the mobilized angle of internal friction defined by $\tan \phi_m = \frac{\tan \phi}{F}$.

N_i is calculated using Eq. (7) and S_i is calculated using Eq. (5), and $\sum_{j=1}^m t_j$ is determined when the factor of safety is known.

The geosynthetic tensile reinforcement force $(\sum_{j=1}^m t_j)$ can be normalized to a parameter K [8] which is equivalent to the earth pressure coefficient:

$$K = \frac{\sum t_j}{0.5\gamma H^2}. \quad (8)$$

For i th slice under consideration, the linear failure surface making an angle α_i with the horizontal is selected such that the tensile force in geosynthetic reinforcement is maximum. To satisfy this condition, $\sum_{j=1}^m t_j$ is maximized with respect to the failure angle α_i . Maximization is done using generalized reduced gradient algorithm which is an inbuilt function in excel ‘SOLVER’ add-in. For each of the changing cells, the Solver evaluates the partial derivative of the objective function $\sum_{j=1}^m t_j$ with respect to the changing cells α_i using the finite-difference method. Solver reads the value of each changing cell α_i , in turn, modifies the value by a perturbation factor (the perturbation factor is 10^{-8} approximately), and writes the new value back to the worksheet cell. This iterative procedure is repeated till the function is maximized. This causes the spreadsheet to recalculate, producing a new value of the objective [16].

The outermost critical poly-linear failure surface obtained from the tieback analysis defines the active soil mass as shown in Fig. 1. It acts as the boundary into which the geosynthetics of required force are anchored so that an internally stable structure is produced. Each geosynthetic layer is extended into the stable backfill soil, so that the required geosynthetic tensile force t_j can be mobilized.

Convergence Study and Validation

Since, it is a numerical study, the results will be sensitive to discretization. In this case, the backfill is divided into n number of slices. Value of n has to be critically examined before performing the parametric study. Several trials were

taken with increasing number of slices ($\phi = 25^\circ, \beta = 70^\circ, c = 15 \text{ kPa}, q = 50 \text{ kPa}, k_h = 0.2$), and value of the total tensile force in the reinforcement $\sum t_j$ is used for comparison. The effect of number of slices is illustrated in Fig. 3. As seen from Fig. 3, it is observed that the results are almost constant when the number of slices is more than 20.

To examine the accuracy of the developed numerical procedure, the total tensile force in the reinforcement $\sum t_j$ computed from the present study for a standard case ($c = 0, q = 0, \beta = 90^\circ$) is compared with those presented in Shahgholi et al. [7] and those using ReSlope program [6, 14]. It is to be noted that results from the present study are exactly the same as Shahgholi et al. [7]. A marginal difference is observed with the results of ReSlope program. A comparison of these results is reported in Table 1.

Results and Discussion

Shear strength parameters (angle of internal friction and cohesion), inclination of wall, horizontal seismic acceleration and surcharge loading are the main factors which affect the behaviour of reinforced soil wall under the seismic loading. To understand the effect of the aforementioned parameters, a parametric study is conducted on a reinforced soil wall with properties as given in Table 2. A typical reinforced wall ($H = 5 \text{ m}$) is considered for the purpose of parametric study. To arrive at optimum number of slices, analyses were performed with increasing number of horizontal slices. After 20 slices, the response was varying marginally and converging towards a constant result. So in the present study, the backfill of the wall is divided into 20 horizontal slices with a geosynthetic reinforcement sheet assumed to be at the centre of each slice. Total number of slices and number of reinforcements are same in the present study and equal to n .

The vertical seismic acceleration coefficient has not been considered in this study as its effect is marginal when

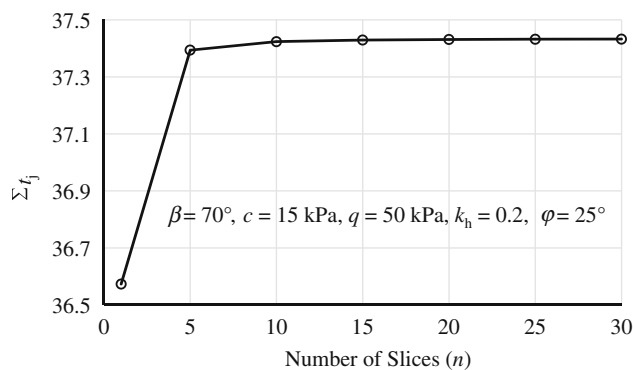


Fig. 3 Effect of number of slices on the total tensile force

Table 1 Comparison of Σt_j from present study ($c = 0, q = 0, \beta = 90^\circ$) with results of ReSlope

k_h	$\phi = 20^\circ$		$\phi = 25^\circ$		$\phi = 30^\circ$		$\phi = 35^\circ$	
	ReSlope	Present study	ReSlope	Present study	ReSlope	Present study	ReSlope	Present study
0	110	110	95	91	74	75	63	61
0.05	119	119	99	99	81	82	68	67
0.1	128	128	110	107	90	89	74	74
0.15	137	139	119	116	99	97	81	81
0.2	151	151	126	127	106	106	90	89

Table 2 Properties of soil and other parameters considered for the analysis

Parameters	Description	Values
γ	Unit weight of soil (kN/m ³)	18
H	Height of wall (m)	5
c	Cohesion of soil (kN/m ²)	0, 5, 10, 15
ϕ	Angle of internal friction of soil (°)	15, 20, 25, 30, 35
β	Inclination of wall (°)	90, 85, 80, 75, 70
q	Surcharge load (kN/m ²)	0, 25, 50
k_h	Horizontal seismic acceleration coefficient	0.0, 0.05, 0.10, 0.15, 0.20
m	Number of geosynthetic layer	20
n	Number of slices	20

compared to the horizontal seismic coefficient [17]. The results obtained from the study are presented graphically and described below.

For a typical case ($\phi = 25^\circ, \beta = 90^\circ, q = 50 \text{ kPa}, c = 0, k_h = 0.2$), the variation in geosynthetic tensile reinforcement force t_j along the height of the wall is presented in Table 3. It is to be noted that the geosynthetic tensile reinforcement force t_j increases with depth. The failure surface inclination α_i to the horizontal for each slice and the required length of reinforcement are also documented in Table 3. The failure surface inclination angle is around 50° at the base which gradually increases to 57° near the top surface. It can be seen that the anchor length (after the failure surface appears) as well as the total length of reinforcement decreases with depth.

Effect of Angle of Internal Friction of Soil (ϕ) on Normalised Reinforcement Force (K)

As the value of ϕ increases, the required K to maintain stability of the wall decreases. Figure 4 shows a plot between the K and ϕ at different values of k_h . From Fig. 4, it is noted that the value of K increases with the increase in k_h . For increase in ϕ from 15° to 25° , the value of K decreases by 29.8 and 31.1 % for $k_h = 0.2$ and 0.0 (static case), respectively. In Fig. 4, it can be observed that for a reinforced wall, the required geosynthetic strength to maintain the stability of wall is lower in the static condition when

compared to that in the seismic condition. But, the effect of angle of internal friction of soil on the stability of wall in the static condition is marginally more pronounced when compared to that of a wall subjected to seismic loading.

The variation of K with ϕ at different q is represented in Fig. 5. It is also observed that, for $q = 50 \text{ kN/m}^2$, the value of K decreases by 29.8 and 51.8 % when ϕ increases from 15° to 25° and 15° to 35° , respectively. For $q = 0 \text{ kN/m}^2$ (no surcharge load on backfill), the value of K decreases by 30.4 and 51.2 % when ϕ increases from 15° to 25° and 15° to 35° , respectively.

It can be concluded that the effect of change of angle of internal friction of soil is almost the same for a backfill with surcharge or no surcharge.

Figure 6 represents the change of the K with ϕ at different cohesion values of backfill. It is also observed that, for $c = 0 \text{ kN/m}^2$ (for a cohesion less soil), the value of K decreases by 29.8 and 51.8 % when ϕ increases from 15° to 25° and 15° to 35° , respectively. For $c = 15 \text{ kN/m}^2$, the value of K decreases by 36.7 and 62.5 % when ϕ increases from 15° to 25° and 15° to 35° , respectively. So, it can be concluded that the effect of change of angle of internal friction of soil on the stability of wall is more significant for a cohesive frictional soil backfill, when compared to that for a cohesionless backfill.

The change in K with ϕ at different values of β is depicted in Fig. 7. It is observed that for a vertical wall ($\beta = 90^\circ$), the value of K decreases by 29.8 and 51.8 % when the value of ϕ changes from 15° to 25° and 15° to 35° ,

Table 3 Typical variation in t_j ($\phi = 25^\circ$, $\beta = 90^\circ$, $q = 50$ kPa, $c = 0$, $k_h = 0.2$)

Slice numbers	Distance from top (m)	Distance from bottom	α_i	Distance of reinforcement from top (m)	t_j (kN/m)	Anchor length (m)	Total length (m)
1	0	5	57.08	0.125	8.64	0.222	3.93
2	0.25	4.75	56.32	0.375	8.95	0.211	3.76
3	0.5	4.5	55.66	0.625	9.24	0.202	3.58
4	0.75	4.25	55.06	0.875	9.53	0.194	3.40
5	1	4	54.54	1.125	9.82	0.187	3.22
6	1.25	3.75	54.06	1.375	10.09	0.181	3.03
7	1.5	3.5	53.63	1.625	10.37	0.175	2.84
8	1.75	3.25	53.24	1.875	10.63	0.170	2.65
9	2	3	52.89	2.125	10.90	0.166	2.46
10	2.25	2.75	52.56	2.375	11.16	0.161	2.26
11	2.5	2.5	52.27	2.625	11.42	0.157	2.07
12	2.75	2.25	51.99	2.875	11.67	0.154	1.87
13	3	2	51.74	3.125	11.92	0.150	1.67
14	3.25	1.75	51.50	3.375	12.17	0.147	1.47
15	3.5	1.5	51.28	3.625	12.42	0.144	1.27
16	3.75	1.25	51.08	3.875	12.66	0.142	1.06
17	4	1	50.88	4.125	12.90	0.139	0.86
18	4.25	0.75	50.70	4.375	13.14	0.137	0.65
19	4.5	0.5	50.54	4.625	13.38	0.135	0.44
20	4.75	0.25	50.38	4.875	13.62	0.133	0.24
					$\Sigma t_j = 224.63$		

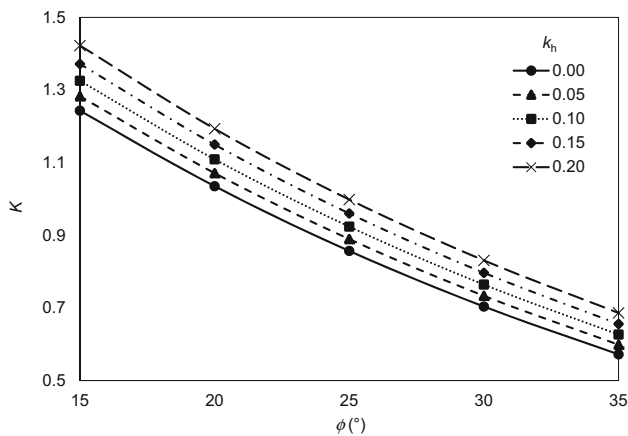


Fig. 4 Variation of K with ϕ for different values of k_h ($c = 0$, $q = 50$ kN/m² and $\beta = 90^\circ$)

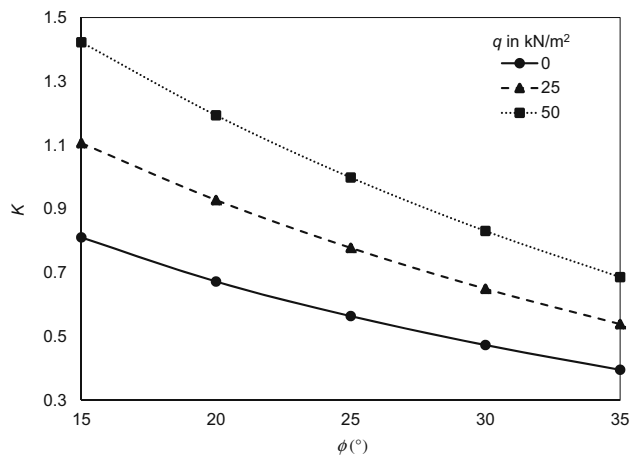


Fig. 5 Variation of K with ϕ for different values of q ($c = 0$, $k_h = 0.2$ and $\beta = 90^\circ$)

respectively. For an inclined wall (with wall inclination $\beta = 70^\circ$), the value of K decreases by 39.6 and 64.8 % when the value of ϕ changed from 15° to 25° and 15° to 35° , respectively. Hence, it can be concluded that the effect of variation of angle of internal friction of soil is more critical for an inclined wall when compared to a vertical wall.

Effect of Horizontal Seismic Acceleration Coefficient k_h

A detailed study is conducted to examine the effect of horizontal seismic acceleration coefficient. In general it is observed that as the horizontal seismic acceleration coef-

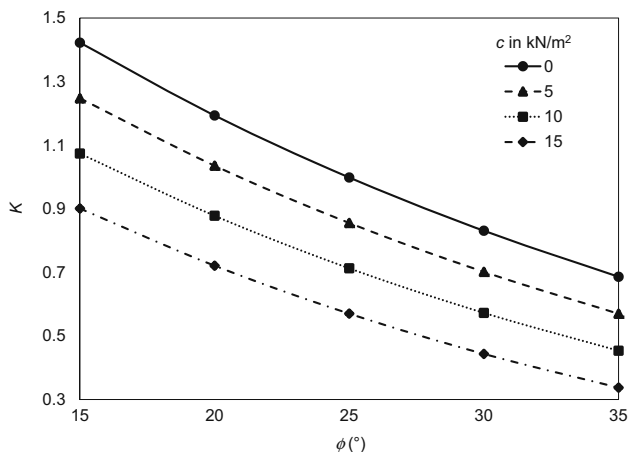


Fig. 6 Variation of K with ϕ for different values of c ($k_h = 0.2$, $q = 50 \text{ kN/m}^2$ and $\beta = 90^\circ$)

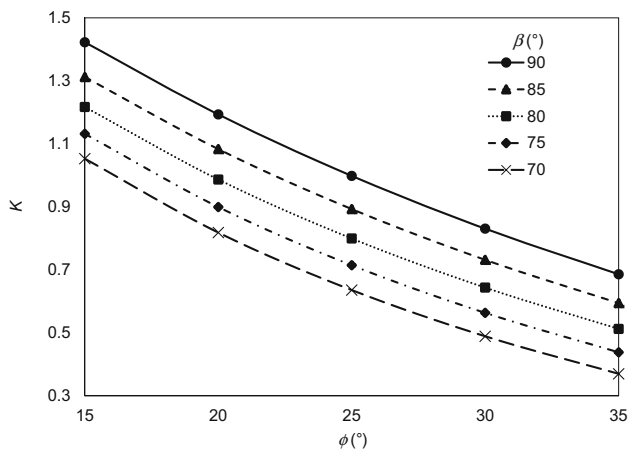


Fig. 7 Variation of K with ϕ for different values of β ($c = 0$, $k_h = 0.2$, and $q = 50 \text{ kN/m}^2$)

efficient increases, the required tensile reinforcement force to maintain the stability of the wall also increases. To quantify the effect of horizontal seismic acceleration coefficient, the ratio (K_{Ratio}) of required tensile reinforcement force K_{Dy} (for non-zero value of k_h) to that for the static condition K_{St} ($k_h = 0.0$) is considered ($K_{Ratio} = K_{Dy}/K_{St}$). The effect of surcharge, cohesion and wall inclination has been quantified in terms of K_{Ratio} and the same is summarized in Table 4. It can be observed that as the backfill surcharge loading increases, the required normalised geosynthetic tensile reinforcement force (K) to maintain stability of the wall increases. It can be also perceived that when $q = 0 \text{ kN/m}^2$, the value of K increases by about 17.3 and 39 % when k_h is increased from 0.00 to 0.10 and 0.00 to 0.20, respectively. Whereas for the same wall subjected to a surcharge loading of 50 kN/m^2 , the value of K increases by about 7.8 and 16.5 % when the value of k_h changes from 0.00 to 0.10 and 0.00 to 0.20,

respectively. Hence it can be concluded that the effect of change of horizontal seismic acceleration is critical for a wall with no surcharge as compared to the wall with surcharge.

Table 4 also documents the change in K_{Ratio} with horizontal seismic acceleration coefficient k_h at different values of cohesion of backfill. In a broader sense it is observed that the absolute value of K is decreasing with an increase in the cohesion of the backfill, however the values of K_{Ratio} increase with the cohesion of backfill. For a cohesionless backfill, as the value of k_h increases from 0.00 to 0.10, the value of K increases by about 7.8 %, while its value increases by about 16.5 % when the value of k_h increases from 0.00 to 0.20. However, for a cohesive frictional soil with cohesion of 15 kN/m^2 , an increase in the value of k_h from 0.00 to 0.10, the value of K increases by about 15.3 %. The increase in K is order of 32 % when the value of k_h increases from 0.00 to 0.20. So, it can be concluded that the effect of horizontal seismic coefficient on the stability of wall is more significant for a cohesive-frictional soil backfill, when compared to that for a cohesionless backfill.

Similarly from the observed variation in K_{Ratio} with k_h at different values of wall angle β , it can be concluded that the value of K_{Ratio} decreases with an increase in the wall angle β . It can be found that for a vertical wall ($\beta = 90^\circ$), there is 7.8 % increase in the value of K when the value of k_h changes from 0.00 to 0.10 and 16.5 % increase when k_h changes from 0.00 to 0.20. For an inclined wall ($\beta = 70^\circ$), the increase in the value of K are 14 and 30.1 % when the value of k_h changes from 0.00 to 0.10 and 0.00 to 0.20, respectively. Hence, it can be concluded that the effect of variation of horizontal seismic coefficient is more critical for an inclined wall when compared to a vertical wall.

Effect of Cohesion c

In the case of a vertical wall, the variation of K with cohesion c at different backfill surcharge loads is represented in Fig. 8. It can be noted that as the backfill surcharge loading increases, the required value of K to maintain stability of the wall increases. It can be also perceived that for a vertical wall subjected to the horizontal seismic loading (for $k_h = 0.2$), the value of K decreases by about 26.3 % when c increase from 0 to 5 kN/m^2 and its value decreases by about 52.1 % when the value of c changes from 0 to 10 kN/m^2 when there is no surcharge loading on backfill. Whereas in case of surcharge loading of 50 kN/m^2 , the value of K decreases by about 14.3 % when the value of c varies from 0 to 5 kN/m^2 and its value decreases by about 28.6 % when the value of c varies from 0 to 10 kN/m^2 . Hence, it can be concluded that the effect of change of cohesion is critical for a wall with no surcharge as compared to the wall with surcharge.

Table 4 Variation in K_{Ratio} with respect to surcharge, cohesion and wall inclination

Cohesion c (kPa)	Surcharge q (kPa)	Inclination β ($^\circ$)	K_{St} $k_h = 0.0$	K_{Ratio} $k_h = 0.05$	K_{Ratio} $k_h = 0.1$	K_{Ratio} $k_h = 0.15$	K_{Ratio} $k_h = 0.2$
0	0	90	0.4059	1.0823	1.1734	1.2749	1.3896
0	25	90	0.6313	1.0521	1.1078	1.1676	1.2320
0	50	90	0.8568	1.0381	1.0782	1.1205	1.1652
0	50	90	0.8568	1.0381	1.0782	1.1205	1.1652
5	50	90	0.7152	1.0456	1.0933	1.1433	1.1959
10	50	90	0.5737	1.0567	1.1158	1.1776	1.2423
15	50	90	0.4321	1.0751	1.1533	1.2347	1.3195
0	50	90	0.8568	1.0381	1.0782	1.1205	1.1652
0	50	85	0.7479	1.0445	1.0915	1.1413	1.1940
0	50	80	0.6525	1.0514	1.1060	1.1638	1.2255
0	50	75	0.5670	1.0591	1.1220	1.1890	1.2607
0	50	70	0.4887	1.0679	1.1404	1.2180	1.3013

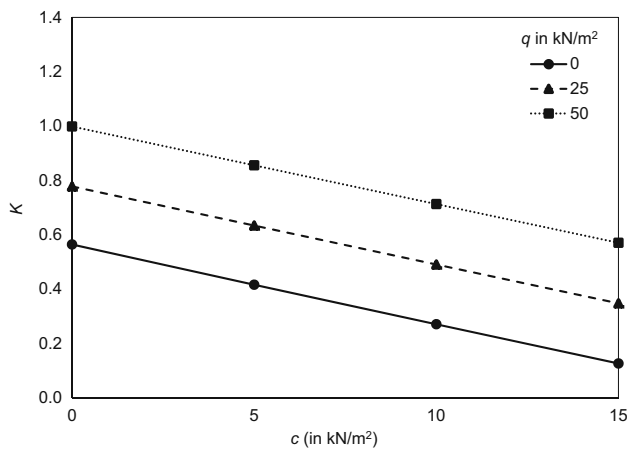


Fig. 8 Variation of K with c for different values of q ($k_h = 0.2$, $\phi = 25^\circ$, and $\beta = 90^\circ$)

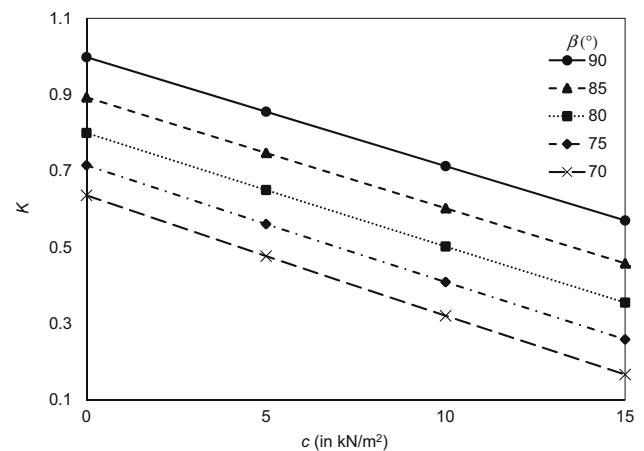


Fig. 9 Variation of K with c for different values of β ($k_h = 0.2$, $\phi = 25^\circ$, and $q = 50 \text{ kN/m}^2$)

The variation in K with cohesion at different values of β is shown in Fig. 9 with $\phi = 25^\circ$, $k_h = 0.2$ and $q = 50 \text{ kN/m}^2$. It can be found that for a vertical wall ($\beta = 90^\circ$), there is 14.3 % decrease in the value of K when the value of c changes from 0 to 5 kN/m^2 and 28.6 % decrease when c changes from 0 to 10 kN/m^2 . For an inclined wall with wall inclination $\beta = 70^\circ$, the decrease in the value of K are 25 and 49.6 % when the value of c changes from 0 to 5 and 0 to 10 kN/m^2 , respectively. So, it can be concluded that the effect of variation of cohesion of soil is more critical for an inclined wall when compared to a vertical wall.

Effect of Surcharge q

The effect of surcharge loading on K for different values of β is highlighted in Fig. 10. It can be found that for a

vertical wall ($\beta = 90^\circ$), there is 37.9 % increase in the value of K when the value of q changes from 0 to 25 kN/m^2 and 77 % increase when q changes from 0 to 50 kN/m^2 . For an inclined wall (with wall inclination $\beta = 70^\circ$), the increase in the value of K are 28.2 and 58.7 % when the value of q changes from 0 to 25 kN/m^2 and 50 kN/m^2 , respectively. So, it can be concluded that the effect of variation of surcharge is more critical for a vertical wall as compared to an inclined wall.

Design Example

It is possible to estimate the value of total tensile force in reinforcement Σt_j using the results presented in Table 4 (for $\phi = 25^\circ$). For example, if it is required to calculate the total tensile force in reinforcement Σt_j for a special case as:

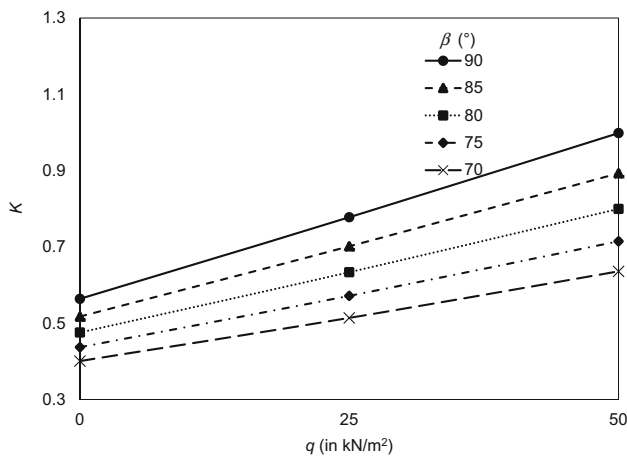


Fig. 10 Variation of K with q for different values of β ($c = 0$, $\phi = 25^\circ$, and $k_h = 0.2$)

$q = 37.5$ kPa, $c = 0$, $k_h = 0.2$ using Table 4, then it can be computed as follows:

- K_{St} for $c = 0$, $k_h = 0.0$ and $q = 25$ kPa is 0.6313. Corresponding K_{Ratio} for $k_h = 0.2$ is 1.2320. Then K_{Dy} for $k_h = 0.2$ is $K_{Dy} = 0.6313 \times 1.2320 = 0.777762$.
- Similarly, K_{St} for $c = 0$, $k_h = 0.0$ and $q = 50$ kPa is 0.8568. Corresponding K_{Ratio} for $k_h = 0.2$ is 1.1652. Then K_{Dy} for $k_h = 0.2$ is $K_{Dy} = 0.8568 \times 1.1652 = 0.998343$.
- For the case of $q = 37.5$ kPa, the average value of K_{Dy} for surcharge 25 and 50 kPa is $(0.777762 + 0.998343) / 2 = 0.888052$. The total tensile force in reinforcement Σt_j is given by equation

$$\sum t_j = K_{Dy} \times (0.5\gamma H^2) = 0.888052 \times 0.5 \times 18 \times 5^2 = 199.812 \text{ kN/m.}$$

The value of the total tensile force in reinforcement Σt_j using numerical procedure is 199.734 kN/m for the case of $q = 37.5$ kPa, $c = 0$, $k_h = 0.2$. These two values are in a close agreement. It can be concluded that Table 4 can be used to evaluate to estimate the total tensile force in reinforcement Σt_j provided $\phi = 25^\circ$ and $H = 5$ m.

Conclusions

Based on the results and discussion presented in the previous section, the following conclusions are made:

- (1) In case of both static and seismic conditions, the increase in angle of internal friction of soil increases the stability of the reinforced wall. Consequently, for a given safety factor, the tensile forces that are developed by the reinforcement to maintain the

stability decreases with an increase in angle of internal friction of the soil.

- (2) An increase of cohesion in the backfill soil mass increases the stability of reinforced soil wall.
- (3) It is observed that as the horizontal seismic acceleration coefficient increases, the required tensile reinforcement force to maintain the stability of the wall increases.
- (4) The stability of reinforced wall is largely affected by the presence of surcharge on the backfill soil for both static and seismic loading conditions. This was noticed from the increase of required tensile reinforcement force with an increase in the magnitude of the surcharge to maintain the stability of wall for a given factor of safety.
- (5) For a given backfill soil and loading conditions, the inclined retaining wall is found to be more stable than the vertical wall.

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