Application of Vertical Reinforcement for Performance Enhancement of Reinforced Soil Under Seismic Loading

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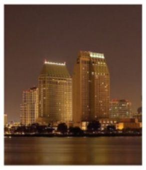

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APPLICATION OF VERTICAL REINFORCEMENT FOR PERFORMANCE ENHANCEMENT OF REINFORCED SOIL UNDER SEISMIC LOADING

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ABSTRACT

Reinforced soils have been widely used in geotechnical structures as a result of their satisfactory performance and cost effectiveness. A number of investigations have been carried out to find out the seismic deformation modes of reinforced soil walls with conventional horizontal inclusions. This study puts forward a new concept of soil reinforcement, applying vertical reinforcement together with conventional horizontal reinforcement. A key difference between the general practice and after insertion of vertical reinforcement is that the latter not only provides passive resistance against shearing due to making intact layers but also increases the strength and stability of the reinforced soil. The concept of soil reinforcement behaviour and its positive effects are analysed under static and seismic loads. The vertical reinforcement can be implemented by stitching horizontal reinforcing layers to each other. For this purpose, different techniques can be applied. Two practical and possible methods, proposed by the authors, are presented. Employing this technology can promote numerous benefits to the current industry of soil reinforcement.

INTRODUCTION

Reinforced soils have been widely used in different variety and range of applications. Many studies, examined the reinforcement of soil, have mainly focused on soil reinforcing with conventional horizontal inclusions (Zhang et al., 2008). Most of the recent papers have been published on reinforced soil foundations (e.g. Alamshahi and Hataf 2009; Latha and Somwanshi 2009) and retaining walls (e.g. Huang and Luo, 2009, Won and Kim, 2007) under static loading. Likewise, some studies carried out considering seismic loading are El-Emam and Bathurst, 2007; Jahanandish and Keshavarz, 2005; Sabermahani et al., 2009 and Shekarian et al., 2008. A few studies were carried out to investigate the strength of soil reinforced with multi-layer horizontal-vertical orthogonal elements (Zhang et al., 2006; Zhang et al., 2008) and pullout response for cellular reinforcement (Khedkar and Mandal, 2009; Wesseloo et al., 2009). It can be noted that these studies have only considered the strength in the case of static loading.

Khedkar and Mandal (2009) have proposed a three-dimensional cellular reinforcement for reinforced soil applications. Their experimental study as well as the finite element analysis for pullout response of cellular reinforcements under low normal pressures has indicated a better performance of cellular reinforcement over the planar one. Yet, in order to incorporate the advantage of up-coming cellular type of reinforcement in reinforced soil retaining wall, a seismic study of cellular reinforcement for its pullout characteristics under working surcharge pressures is required.

A three-dimensional cellular reinforcement can be used in place of the horizontally placed, in the conventional, two-dimensional reinforcement in the reinforced soil retaining walls (Khedkar and Mandal, 2009). Longitudinal members are connected perpendicular to transverse members of equal height. Various materials such as steel, polypropylene, high density polyethylene, etc. can be used in the manufacture of such reinforcement. The addition of reinforcement in the form of height over two dimensional reinforcements makes the cellular reinforcement stiffer, allowing low modulus materials for the manufacturing purposes. The raised height of traverse member the increased height of transverse member provides good bearing resistance in the pullout situations of cellular reinforcement depending upon the longitudinal spacing.

Zhang et al. (2006) have suggested three-dimensional reinforcing elements for reinforced soil applications. They demonstrated the adequacy and enhanced performance of the three-dimensional reinforcement over the planar reinforcement based on an array of triaxial test results. However, testing procedures under plane strain conditions are preferred, which is the most practical mothod in case of reinforced soil walls. Based on experimental results, Zhang et al. (2008) analysed the interaction of reinforcing elements and conducted a comparison between shear strength of the soil reinforced with horizontal reinforcements and orthogonal inclusions. A strength model was developed using the limit equilibrium theory and compared with the results of triaxial tests for soil reinforced with multi-layer orthogonal inclusions. The results of analytical solution were in and stability of the reinforced soil, but the latter mainly enlarges frictional resistance. However, these studies of partial inclusion of vertical elements do not consider the dynamic loading and reinforced soil walls performance under such loading.

Hoe and Dov (2005) studied the post earthquake assessment of the soil reinforcement of Ta Kung Wall, Ji-Ji Earthquake. The internal stability is evaluated by using tieback analysis, assuming a log-spiral mechanism so that the length and strength of reinforcements are determined. The stability is achieved by anchoring potential failure soil mass into stable backfill. In the external stability evaluation, direct sliding and compound failure are considered. For geosynthetic force, the most critical condition was for vertical acceleration that acts downward, whereas the most critical failure surfaces were for vertical acceleration that acts upward. Their study showed that the Ta Kung wall would have been stable against failure in the absence of vertical acceleration. However, with the vertical acceleration, the wall was no longer stable against failure.

This paper presents a new concept of soil reinforcement using vertical reinforcement designed for the connection of two conventional horizontal reinforcements. The primary difference between the general practice and the insertion of vertical reinforcement is that the latter not only provides passive resistances against shearing but also makes all the layers to remain intact increasing the strength and stability of the reinforced soil. The idea of asserting the effect of vertical acceleration by strengthening soil-reinforcement interactions installing vertical reinforcements or inclined reinforcements is discoursed. The concept of soil reinforcement behaviour and its positive effects are analysed for static and dynamic loading. Moreover, the change in reinforced soil behaviour with inclusion of vertical reinforcement and its constructive role to address the most possible modes of failure are discusses.

CONCEPTUAL DESIGN OF REINFORCED SOIL INCORPORATING VERTICAL REINFORCEMENT

The main concept of this type of soil reinforcement is associated with combining vertical reinforcement with the conventional horizontal reinforcement based on the design requirements. For this system, similar to normal reinforcement, the selected granular material is compacted over horizontal reinforcement up to a given height and then another layer of horizontal reinforcement is laid down. Afterwards, the proposed vertical reinforcements are inserted vertically or in an angle with vertical as per design requirement. A typical soil structure reinforced with 3D reinforcing elements for in situ applications is shown in Fig. 1.

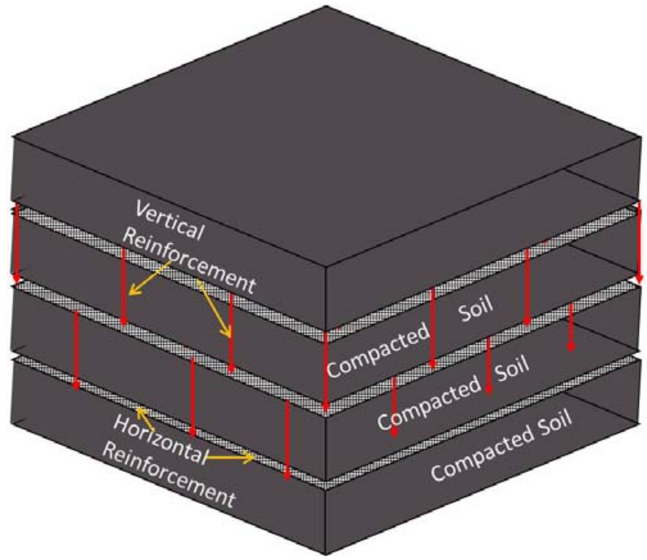
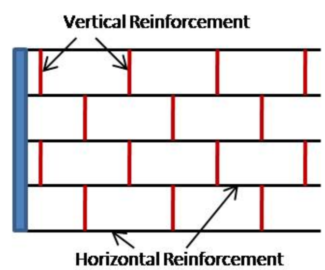
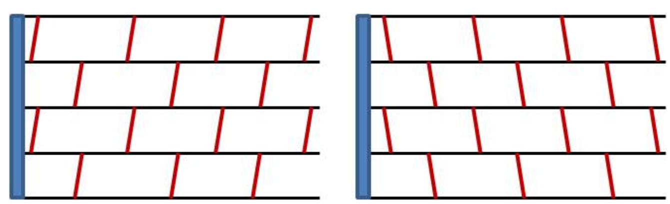


Fig. 1. Typical reinforced soil using vertical reinforcement

The cross sectional configurations of vertical reinforcing elements are shown in Fig.2.



(a)



(b) (c)

Fig. 2. Front section of typical vertical reinforcement: (a) Horizontal and vertical reinforcement; (b) and (c) Horizontal and inclined reinforcement

OVERVIEW OF FAILURE MODES

The modes of failure for design of reinforced soil walls can be divided into three categories in current guidelines and specifications. They are: external, internal and facing elements failures. The external stability considers the reinforced soil mass as a rigid body subject to lateral earth pressures from backfill soil and supplement loads. In design, such instability in the design of walls consists of base sliding, overturning, bearing capacity, excessive settlement, and global (deep seated) failures. The bearing capacity and settlement failure modes depend on each other. The wall settlements can be limited if they are designed properly considering the bearing capacity and eccentricity failure modes. The internal stability considers the position and strength of reinforcement within the reinforced soil mass. It comprises of tensile over-stressed, pullout and internal sliding failures of reinforcements. Required reinforcement length, position, and strength are determined such that the wall design will satisfy all the failure modes with minimum safety factors given in the specifications. The facing elements failure, considered as a local stability criterion in design, is related to connections of the reinforcement and facing units, column shear failure and toppling.

Further instability modes should also be considered in certain conditions, such as modes of failure governed by seismic or cyclic loading. Sabermahani et al. (2009) investigated two main seismic deformation modes, overturning (maximum displacement at top) and bulging (maximum displacement at mid-height) of the facing, together with an additional base sliding mode that occurred simultaneously to the other two modes.

Overturning Mode

In overturning mode, the top of the wall faced the maximum lateral displacement, causing the outwards movement of reinforced zone rotating similar to a rigid block. The outwards movement was responsible for the formation of the gap in front of the backfill which caused development of parallel multi-line failure surfaces. The maximum ground surface settlement over the wall was observed on the top of the gap. Moreover, multi-line failure surfaces were occurred at the rear part of the reinforced zone in the backfill but no internal failure was noticed in the reinforced zone. Figure 3 shows the details of the failure mechanism and the deformation mode in the reinforced zone and backfill. This figure depicts a stepped-shape formation on the ground surface caused by a discontinuity appeared in the ground settlement profile. Furthermore, the ends of the reinforcement layers were settled downward due to the movement of drag-down force behind the reinforced zone.

This behaviour is based on some limit equilibrium assumption as it is not exactly similar to the overturning or base sliding of a rigid body. The rotation of the reinforced zone around the toe of the wall, was presumed as rigid, dragged the first layer of reinforcement to excavate the foundation soil layer.

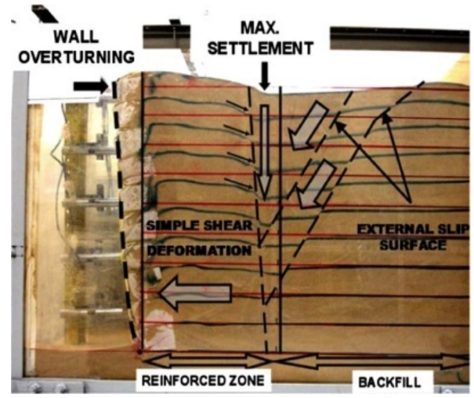


Fig. 3. Details of overturning mode and failure mechanism (Sabermahani et al. 2009)

Bulging Mode

Generally, the failure mechanism in walls with bulging deformation mode was not similar to the overturning mode, where no external failure surface was observed, but an internal single failure surface was noticed in the reinforced zone whereas, the maximum displacement occurred in the middle of the wall facing.

Figure 4 describes the details of the slip surface position, the internal failure mechanism, and the maximum settlement in the bulging mode. As there was an absence of large lateral movement of the reinforced body, uniform small settlement in the backfill was demonstrated in ground surface profile. The face bulging in convex shape caused the concave shaped settlement profile that developed the maximum settlement at the mid of the reinforced zone. This behaviour was considered as a flexible medium and this flexibility of the reinforced zone facilitated the wall toe not to excavate the foundation layer.

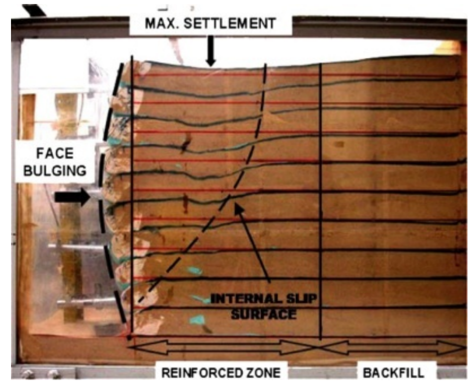


Fig. 4. Details of bulging mode and failure mechanism (Sabermahani et al. 2009)

IMPROVING REINFORCED SOIL BEHAVIOUR

The soil reinforcement function is a mechanical improvement of soil performance, which achieves by supporting tensile forces in two ways: (a) to reduce the shear force that has to be carried by the soil, and (b) to enhance the available sharing resistance in the soil by increasing the normal stress acting on the potential shear surfaces.

Soil shearing resistance stems from frictional contact among soil particles subject to the effective compressive stress. Deformation in the soil causes tensile or compressive stresses to be developed in the reinforcement. The magnitude of stresses depends on the reinforcement inclination in the direction of tensile or compressive stresses in the soil. As reported by Jewell (1996), the mobilised reinforced force, ultimately limited by the available bond, acts to alter the force equilibrium in the soil mass.

Shear deformation in the soil will cause tensile force Tr, to be mobilised in the reinforcement, and provide two additional components of resistance in the slope (Fig. 5a). The tangential component of the reinforcement force, $Tr\mathrm{Sin}\theta$ directly resists the disturbing shear force in the soil, while the normal component of the force, $Tr\cos\theta$, mobilises additional frictional shearing resistance, $Tr\cos\theta\tan\phi$.

Vertical reinforcement adds two more components: $Tvr\cos\theta$ resisting the disturbing shear force, and $Tvr\mathrm{Sin}\theta$, normal component of the force, provides extra frictional shearing resistance $Tvr\mathrm{Sin}\theta\tan\phi$ as shown in Fig. 5b. In addition to this, it will cage the soils in different units alongside layered by horizontal reinforcement and produce intact effect in soil mass.

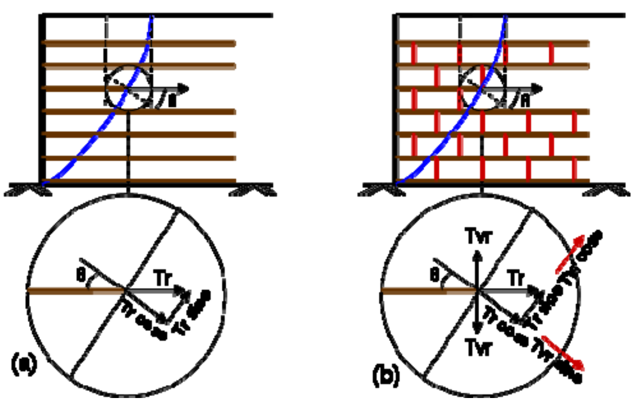


Fig. 5. Effect of reinforcement on equilibrium allowing for: a) horizontal reinforcement, b) horizontal and vertical reinforcement

The shear resistance of soil is given by $\tau = \sigma_1 \tan\phi$ where σ_1 denotes the vertical stress, τ is the shear stress, and A is the area of the soil shear surface. Shearing force increases due to horizontal reinforcement is given as:

$$\tau = \sigma_1 \tan\phi + (T_r/A)(\sin\theta + \cos\theta\tan\phi) \qquad (1)$$

Equation (1) further modified increasing shear force due to vertical reinforcement as:

$$\tau = \sigma_1 \tan\phi + (T_r/A)(\sin\theta + \cos\theta\tan\phi) + (T_{vr}/A_{vr})(\cos\theta + \sin\theta\tan\phi) \qquad (2)$$

In addition, for an inclined reinforcement from the vertical, as shown in Fig. 6, the additional shear strength provided by inclined reinforcement can be estimated by the following equation:

$$\tau = \sigma_1 \tan\phi + (T_r/A)(\sin\theta + \cos\theta\tan\phi) + (T_{vr}/A_{vr})(\cos(90-\varphi) + \sin(90-\varphi)\tan\phi) \qquad (3)$$

where A_{vr} is the area of the soil shear surface, ϕ is the angle of shear distortion and is expressed as $\tan^{-1}[1/(m+(\tan i)^{-1})]$, where i is the initial angle of inclination with respect to shear surface; m is shear distortion ratio $(m = x/z)$. The optimal orientation for roots to provide the additional shear strength will be finalised in further research. However, it will be around in an inclination between 40◦ and 70◦ rather than in a vertical orientation like in inclined root-soil interaction.

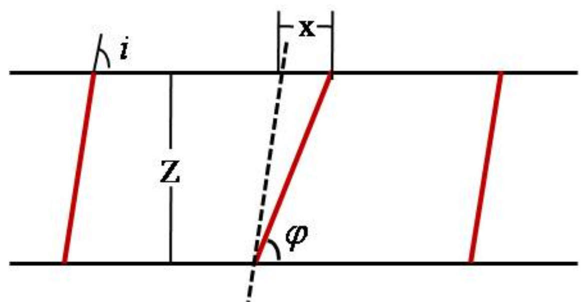


Fig. 6. Inclined-Horizontal Reinforcement Model

Further, the vertical component of inclined reinforcement provides tensile force, while its horizontal component along with horizontal reinforcement provides the horizontal tensile force during seismic loading.

ENHANCEMENT AGAINST MODES OF FAILURE

The advantage from proposed reinforcement mechanisms against following failures: bearing capacity, tensile over-stress and pullout are analysed in this section.

Bearing capacity of foundation

Many experimental and analytical studies have been performed to investigate the behaviour of reinforced soil foundation (RSF) for different soil types. The method of superposition can be used to include the contribution of reinforcement (Sharma et al., 2009) and the bearing capacity including increased bearing capacity can be given as:

$$q_{u(R)} = q_{u(UR)} + \sum_{i=1}^{N} \frac{4T_i[u+(i-1)h]}{B^2} \qquad (4)$$

where, T_i is the tensile force in the i^{th} reinforcement layer, u is the depth of reinforcement location, $q_{u(UR)}$ is the bearing capacity of unreinforced soil foundation depending on the friction angle of soil.

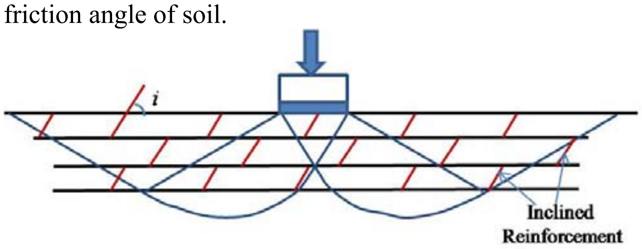


Fig. 7. Failure mode within reinforced zone of soil foundation

Vertical reinforcement enhances the tensile strength and provides bending effects, which ultimately increases the bearing capacity with some modification in equation (4) as follows:

$$q_{u(R)} = q_{u(UR)} + \sum_{i=1}^{N} \frac{4(T_i + \Delta T_{vi})[u + (i-1)h]}{B^2} + f(I) \qquad (5)$$

where, ΔT_{vi} is the increased tensile force due to the vertical reinforcement and $f(I)$ is bending effect on reinforced soil as a function of the moment of inertia.

Tensile over-stress

Rupture failure occurs in the reinforcement after the tensile force in the reinforcement becomes larger than the reinforcement strength. In current design, rupture failure is being checked and vertical spacing between reinforcement layers should be decreased if reinforcement strength is less than the tensile force or reinforcement with higher allowable tensile strength should be selected.

The failure envelop of unreinforced $c-\phi$ soil is given by:

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi} \qquad (6)$$

Where c is the cohesion and N_ϕ can be defined as $N_\phi = \tan^2(45+\phi/2)$. This envelop modifies if we used horizontal reinforcement, which is given by

$$\sigma_1 = (\sigma_3 + R_T/S_z)N_\phi \qquad (7)$$

where R_T is the tensile strength of reinforcing materials per unit length, S_z is the spacing of reinforcements. Comparing equations (6) and (7) gives:

$$c = R_T/2S_z\sqrt{N_\phi} \qquad (8)$$

Therefore, failure envelop of reinforced and unreinforced soils are parallel and exhibit the same angle of internal friction. The additional strength develops in the form of anisotropic apparent cohesion.

Vertical reinforcement unites the soil mass vertically as well as horizontally. First, tensile stresses are produced in the horizontal reinforcement and a corresponding compression in the soil element. Then, it extends tensile stresses in vertical reinforcement before the slippage between the soil and horizontal reinforcement. These additional stresses enlarge the apparent anisotropic cohesion and the friction angle, which is an additional strength to the soil over tensile stress. Hence, they reduce the chance of rupture failure in the reinforcement arrangement.

Pullout Failure

Pullout failure occurs when tensile force in reinforcement exceeds the friction force between the reinforcement and soil. A number of numerical and experimental tests have been carried out to identify the pullout strength behaviour for different types of reinforcement (e.g. Saran 2005).

For the slippage failure, the failure envelop is given by:

$$\sigma_1 = \sigma_3 N_\phi' \qquad (9)$$

$$\text{where}, \quad N'_\phi = N_\phi/1-(2b_r.f^*/S_z)N_\phi \qquad (10)$$

As $N'_\phi > N_\phi$, then, $\phi_R > \phi$. The parameter N_ϕ can be expressed as: $N_\phi = \tan^2(45+\phi_R/2)$ and f^* represents the coefficient of internal friction between the reinforcing material and soil.

As a result, in the case of failure of reinforcement due to slippage, the failure envelop of reinforced sand will also pass through the origin, and it indicates an increase of angle of internal friction. Application of vertical reinforcement increases the total area of reinforcement, which enlarges the internal friction angle, and ultimately enhances soil strength performance.

Overturning mode under dynamic loading

In the overturning mode, the reinforced zone moved outwards like a rigid block with internal deformation in a simple shear manner. As Sabermahani et al. (2009) reported that multi-line failure surfaces were formed in the unreinforced backfill, since there was no failure in the reinforced zone. A stepped-shape settlement profile formed on the ground surface because of maximum settlement behind the reinforced zone.

Vertical/inclined reinforcement ties each layer to another so that such a multi-line failure overcomes. This extra reinforcement reduces internal sliding mobilising its tensile strength before slippage of one layer over another. Moreover, the vertical component of inclined reinforcement can decrease the wall settlement as shown in Fig. 3.

Bulging mode under dynamic loading

According to Sabermahani et al (2009) in the bulging mode, walls behave more flexibly and a single failure surface formed at the reinforced zone. The convex shape deformation of the facing causes a concave settlement profile on the ground surface with a maximum value at the middle of the reinforced zone.

The caging effect will obtain by using vertical reinforcement along with the horizontal one. Each layer ties with another and will behave as one which reduces the total force at the back of the facing panel. Therefore, the chance of bulging failure reduces with vertical reinforcement.

IMPROVED RESPONSE TO DYNAMIC LOADING

The application of reinforced-soil has increased worldwide as a result of their reasonable seismic performance and cost effectiveness. Recent investigations show that formation of overturning mode due to maximum displacement at the top and bulging mode due to the maximum displacement at the mid-height due to seismic loading. These effects of such modes of failure can be provided using vertical reinforcement.

Active Earth Force in Seismic Loading

The magnitude of the dynamic force increment due to shaking is evaluated using the Mononobe-Okakabe approach (Mononobe 1924, 1929; Mononobe and Matsuo 1929; Okabe 1924) in the external seismic stability analysis which is an extension to the conventional Coulomb sliding wedge theory integrating the effects of lateral inertia forces on the retained soil mass. As in the static case, the soil at the rear is considered to be in limit equilibrium exerting horizontal force onto the reinforced soil block. Considering the soil block is a monolithic unit, this external force is simply applied to the reinforced block and conditions regarding external stability are calculated. In internal design, the sideways inertial force applied on the potentially sliding block (active zone) is estimated from the seismic coefficient (k_h). Under static forces and this additional dynamic force, pullout and rupture are checked.

In earthquake engineering practice, Mononobe-Okabe approach is common, in which the Coulomb wedge analysis is extended to include horizontal and vertical inertial forces due to ground shaking. The geometry and force diagram associated with this method is shown in Fig. 8. The backfill retained by the wall is assumed to be in an active mode of failure under self weight and inertial forces due to ground acceleration. Both the retaining structure and the retained backfill act as rigid bodies with the maximum shear stress along the potential sliding surface.

Dynamic earth pressures on earth retaining structures are a complex problem of soil-structure interaction and suggest that peak dynamic stresses should be of main concern in design (Whitman 1991). The Mononobe-Okabe approach fails to represent the actual dynamic behaviour, but it is a scheme to relate dynamic earth pressures to a possible state of failure.

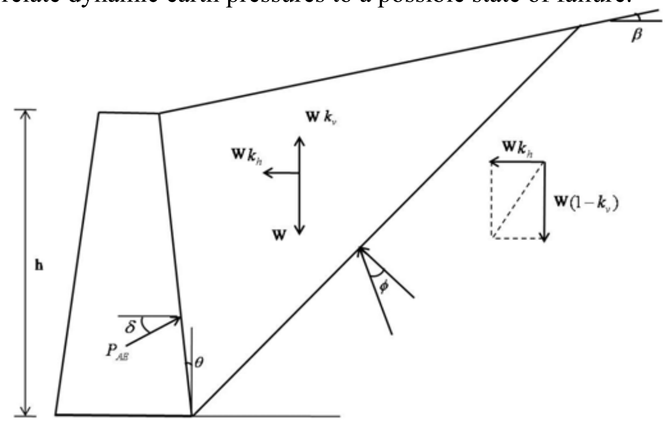


Fig. 8. Mononobe-Okabe approach

Magnitude of the dynamic earth force is correlated to the static earth pressure by a coefficient k_h which is based on the maximum ground acceleration. In Mononobe-Okabe method the total active earth force is calculated as;

$$P_{AE} = \frac{1}{2}\gamma H^2[(1 \pm k_v)K_{AE}] \tag{11}$$

where, γ is unit weight of the retained soil, and H height of the wall and K_{AE} is the total earth pressure coefficient. The vertical acceleration coefficient k_v will have a plus sign when acting downward and a minus sign when acting upward. In most cases the vertical ground acceleration is taken as acting upward reducing the total active earth pressure; while in some cases, it is ignored completely. Following formulations will use the convention where vertical ground acceleration is acting upward, utilizing the form of the above formula with the negative sign. The total earth pressure coefficient for a cohesionless dry backfill can be calculated using the formula;

$$K_{AE} = \frac{\cos^2(\phi - \psi - \theta)/\cos\psi\cos^2\theta\cos(\psi + \theta + \delta)}{\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \psi - \beta)}{\cos(\beta - \theta)\cos(\psi + \theta + \delta)}}\right]^2} \tag{12}$$

where,
ϕ = the friction angle of the retained soil
δ = the mobilised interface friction angle between the back of the wall facing and the backfill soil (or the mobilised interface friction angle between back of the reinforced soil zone and the retained soil in case the reinforced earth wall system is treated as a monolithic structure)
θ = the inclination angle of the inside face of the wall with the vertical (or batter angle of the back of MSE wall)
β = the back-slope angle
ψ = the seismic inertia angle given by

$$\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$$

The horizontal component of total active thrust is:

$$P_{AE-HOR} = \frac{1}{2}\gamma H^2[(1-k_v)K_{AE}]\cos(\delta+\theta) \quad (13)$$

Parameters k_h and k_v are horizontal and vertical seismic coefficients, respectively. These parameters are expressed as a fraction of the gravitational acceleration, g.

Dynamic behavior parameters of reinforced-soil walls

During vibration of soil layers due to an earthquake, the stress-strain hysteresis loop may be obtained based on non-linear elastic curve as shown in Fig. 9.

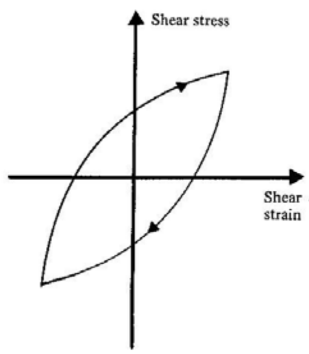


Fig. 9. Shear stress-strain characteristics of soil

In earthquake related problems, the level of shear strain has considerable effects on the shear moduli and damping ratio of soils. As the magnitude of shear strain increases, the value of shear modulus, G, of a soil decreases (Fig. 10), and the damping ratio increases. According to Fig. 10, it can be inferred that the value of the maximum shear modulus, G_{max}, is for a very small strain (i.e. measurement of field wave velocity).

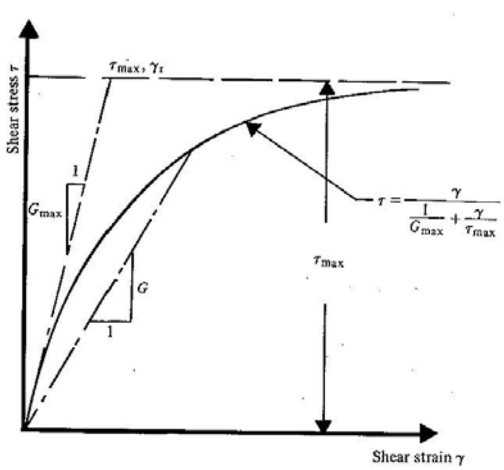


Fig. 10. Nature of variation of shear modulus with strain

Estimation of Shear Modulus: Based on several experimental observations, Hardin and Drnevich (1972) proposed a generalised method according to that the variation of shear stress against strain of all soils can be approximated by a hyperbolic relation (Fig 10).

$$\tau = \frac{\gamma}{1/G_{max} + \gamma/\tau_{max}} \quad (14)$$

where τ is the shear stress, γ is the shear strain, $G_{max} = \tau_{max}/\gamma_r$, γ_r is the reference strain and τ_{max} is the maximum shear stress at failure.

Estimation of Damping Ratio: Hardin and Drnevich (1972) presented a relationship between the damping ratio and shear modulus as:

$$D = D_{max}(1 - G/G_{max}) \quad (15)$$

where D_{max} is the maximum damping ratio.

After inclusion of vertical reinforcement over conventional reinforcement, the value of the maximum shear stress τ_{max} raises as the apparent cohesion and the friction angle of soil mass increases. This increased value of τ_{max} can boost the value of the shear stress, τ in dynamic loading. In addition, intactness of soil mass by the connection of reinforcing layers enlarges the damping ratio according to the Equation 5.

CONSTRUCTION PROCESS

The main objective of the vertical reinforcement is to stitch each horizontal reinforcing layer to another. As this proposed technique is a new concept, the detailed process of construction can be another major challenge.

For this purpose, approximately a 10-20 mm diameter high tensile rod/pipe with a cone tip can be used similar to a cone penetration test. As the spacing of reinforcement is generally not more than 500mm, the maximum insertion depth for this equipment can be around one meter. Thus, it does not need substantial load for insertion and pull out of the rod. The detailed features of this equipment can be designed after consulting with manufacturers. For this purpose, different techniques are presented below.

Push-Pull Method

Firstly, a horizontal reinforcement is laid down, which is covered by a compacted soil layer. Then, a fibre cord with a V-shape flexible tip, which opens as an umbrella beneath the first layer, is inserted just below the lower reinforcement with the help of a metal hose as shown in Fig. 11. When the fibre cord is pulled out, the V-shape tip will be opened as an umbrella and then, this will be tied with the upper layer of horizontal reinforcement.

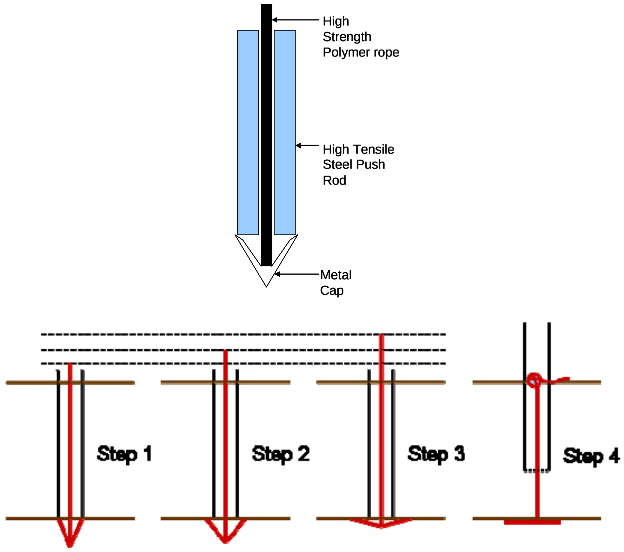
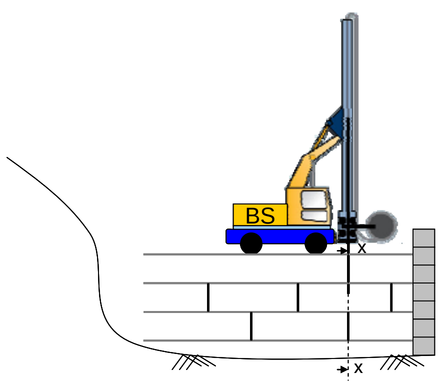
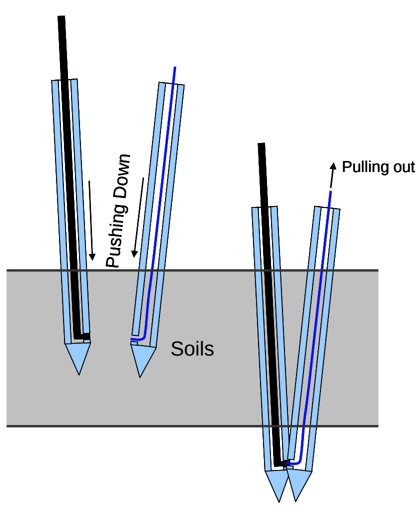


Fig. 11. Construction steps for push-pull method



(a)



(b)

Fig. 12. Sewing mechanism: (a) Sewing mechanism mounted on truck, (b) Insertion process

<u>Sewing method</u>

Sewing mechanism mounted truck (shown in Fig. 12a) can be more useful for large scale work. In this method, two inserting rods stitch two layers of reinforcements; the inserter rod is ejected down for insertion of the vertical reinforcement and the puller rod is pullout as shown in Fig 12b. The two rods are inserted with a V-angle downwards depending on the spacing of the reinforcement. After penetrating lower horizontal reinforcement, these rods come in touch with each other so that the puller rod comes out bringing the vertical reinforcement up. This phenomenon will fasten the HR. Similar to the previous method; the vertical reinforcement is inserted to tie up two horizontal reinforcements.

CONCLUSIONS

This paper summarises an innovative concept for the improvement of reinforced-soil performance. Vertical reinforcement not only ties each layer to another but also encounter some components of tensile forces and increases frictional resistance rising anisotropic cohesion and frictional angle. The theoretical analysis anticipates strength upgrading against bearing capacity, tensile over-stress and pullout failures in static loading and overturning and bulging modes in seismic loading. The major component of seismic force will resist by the combined effect of both reinforcement, while its components will resist by its corresponding direction reinforcement. Vertical reinforcement attempts to create block action against seismic force, which is very important measure to protect structures from the earthquake or other dynamic loading. This technology has been largely driven by economics to get the optimum benefit of soil reinforcement. Consequently, employing this concept can bring substantial benefits to the current soil reinforcement industry.

As the proposed technique is in its initial stages, the findings acquired from these theoretical analyses necessitate further validation based on in situ performance. Further investigation based on large scale laboratory testing and numerical modeling would be required to extend the results for field applications. Additional research on reinforced soil behaviour incorporating vertical reinforcement using rigorous numerical modeling and experimental testing is carrying out by the authors and the results will be disseminated in the follow up papers.

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