SEGMENTAL BLOCK RETAINING WALLS WITH COMBINATION GEOGRID AND ANCHOR REINFORCEMENTS

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ABSTRACT: Reinforced segmental block retaining walls can be constructed where the reinforced fill zone is constrained by using a combination of geogrid reinforcement and anchor reinforcement. The paper presents the design theory for reinforced soil walls where the reinforced fill zone is constrained and where anchors are used to dissipate the reinforcement residual tensile stresses beyond the reinforced fill zone. A case study is also presented where this technique was successfully used.

INTRODUCTION

Geogrid reinforced segmental block retaining walls have become an accepted practice for economical retaining wall construction. For this retaining wall system the segmental blocks act as the wall facing, and provide some stability, with the majority of the stability provided by the geogrid reinforced fill behind the segmental block facing. For stability, the geogrid reinforcement has to have adequate design strength, be located at specific vertical spacings in the reinforced fill, and extend an adequate distance into the reinforced fill. While geogrid reinforcement design strengths and vertical spacings are easy to attain, the required reinforcement lengths into the reinforced fill may be difficult to attain if there are outcrops of heavily overconsolidated soils, soft rocks or hard rocks in close vicinity to the retaining wall. In these instances it can be impractical to excavate the heavily overconsolidated soil or rock and thus, some additional means must be found to fully dissipate the tensile stresses induced in the reinforcement over its truncated length.

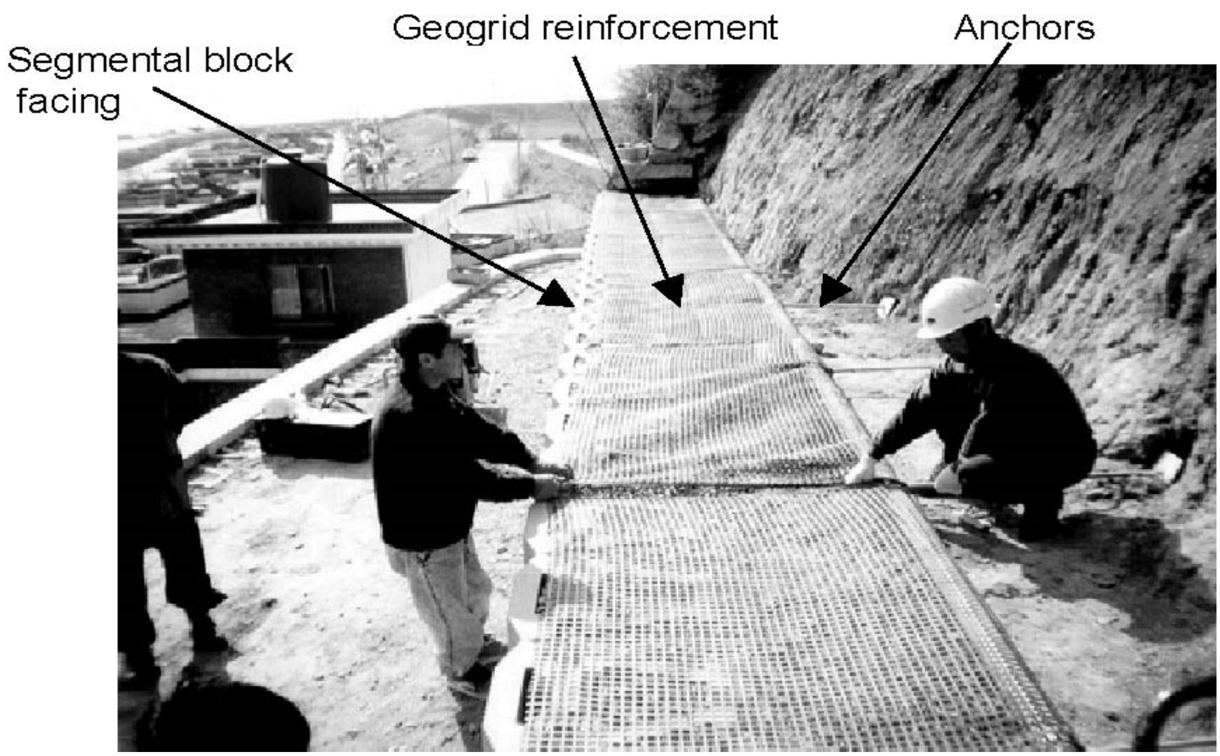

Figure 1. Geogrid reinforced segmental block wall with a constrained reinforced fill zone

One technique is to anchor the geogrid reinforcement into the heavily overconsolidated soil or rock zone at the rear of the reinforced fill zone, Figure 1. The anchors act to fully dissipate any residual tensile stresses at the rear of the geogrid reinforcement into the heavily overconsolidated soil or rock zone. A comparison between a conventional geogrid reinforced segmental block wall and one with a combination of geogrid and anchor reinforcements is shown in Figure 2. For the conventional reinforced segmental block wall the extent of the reinforced fill zone is adequate to account for local and external stability requirements. However, the segmental wall with the constrained reinforced fill zone cannot generate adequate internal stability in terms of geogrid reinforcement bond because of the truncated reinforcement lengths. By attaching anchors to the geogrid reinforcements and inserting them into the heavily overconsolidated soil or rock zone residual tensile stresses in the geogrid reinforcement can be fully dissipated, and the retaining wall can achieve the required stability.

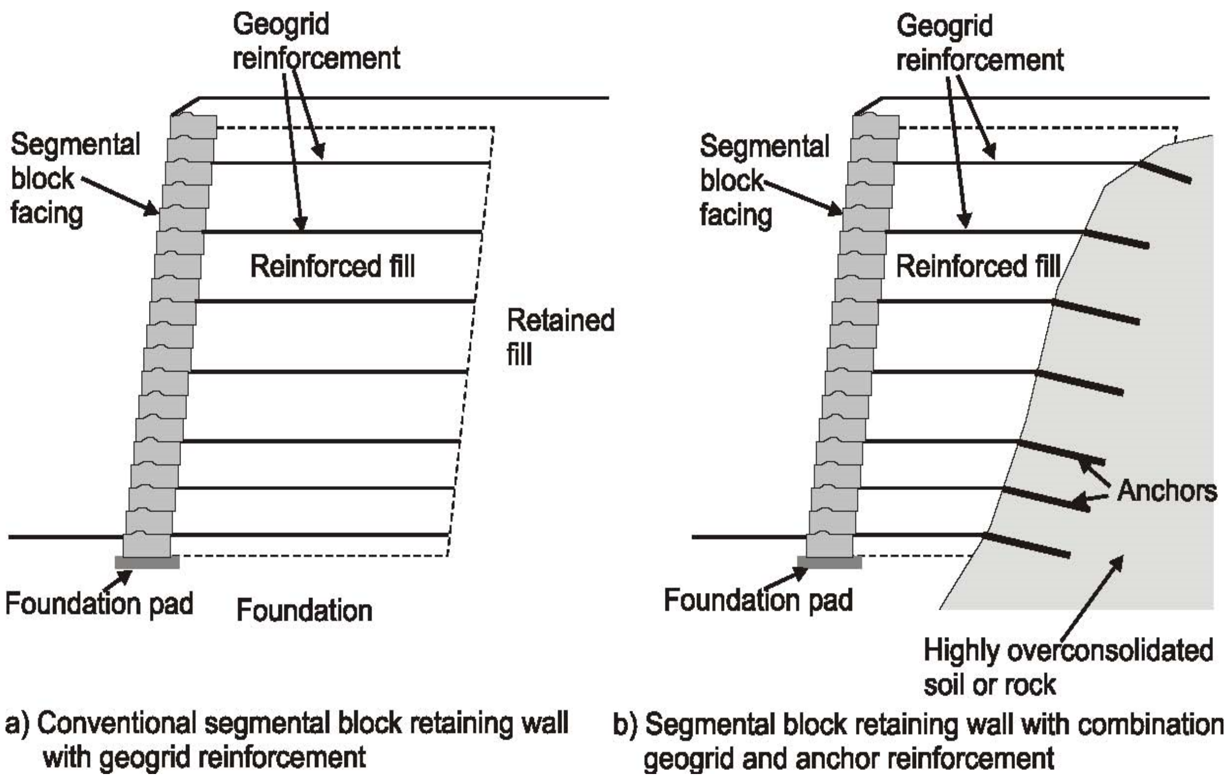

Figure 2. Conventional geogrid reinforced segmental block wall and one with a combination of geogrid and anchor reinforcements

The general representation of the constrained reinforced soil retaining wall problem is shown in Figure 3. Layers of geogrid reinforcement lying within the reinforced fill zone transfer tensile stresses from the vicinity of the wall face, of height *H*, and dissipate these within the reinforced fill, of base width L_b. The extent of the reinforced fill is constrained by a rigid zone, inclined at an angle α to the horizontal, at the rear of the reinforced fill zone. If the tensile stresses in the geogrid reinforcements cannot dissipate before reaching the rigid zone then anchors attached to the geogrid reinforcements are inserted into the rigid zone to provide the required residual tensile stress dissipation. Depending on the geometry, anchors may, or may not, be required for the full height of the wall.

EARTH PRESSURES ACTING ON THE REAR OF THE SEGMENTAL BLOCK WALL FACING

The vertical stresses acting within the reinforced fill are due to the self weight of the reinforced fill plus surcharge, plus other additional vertical stresses due to external vertical loads acting on the reinforced fill. Since the rigid zone at the rear of the reinforced fill zone does not impart stresses onto the reinforced fill (unlike retained soil) there is no influence from this zone on the vertical stresses in the reinforced fill zone. Further, while the interface of the rigid zone may be considered an unyielding surface, the interface at the wall face is a yielding surface due to the reinforcement connections with the wall face generally being made with extensible geogrids. Consequently, the presence of this yielding surface at the wall face prevents any arching occurring within the constrained zone of the reinforced fill. Thus, the vertical stresses acting within the reinforced fill zone can be considered uniform across the full width of the reinforced fill, and are due to the full self weight of the reinforced fill plus any surcharge and external vertical load effects.

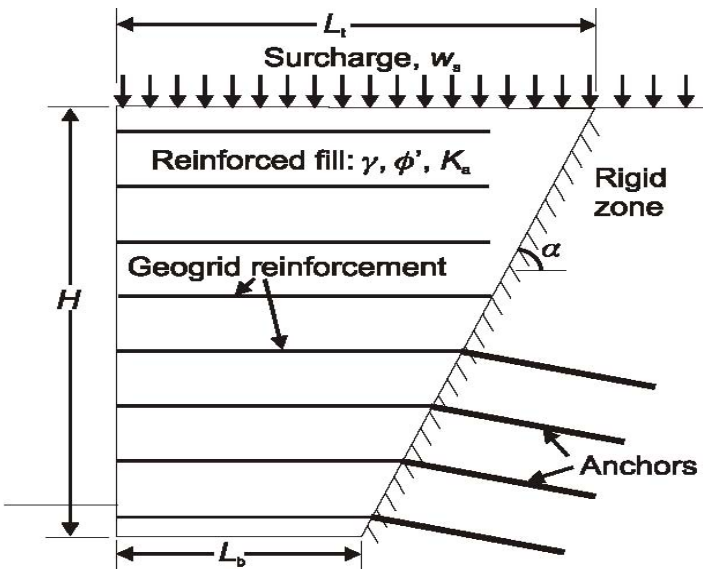

Figure 3. General representation of the reinforced soil retaining wall problem with constrained reinforced fill zone

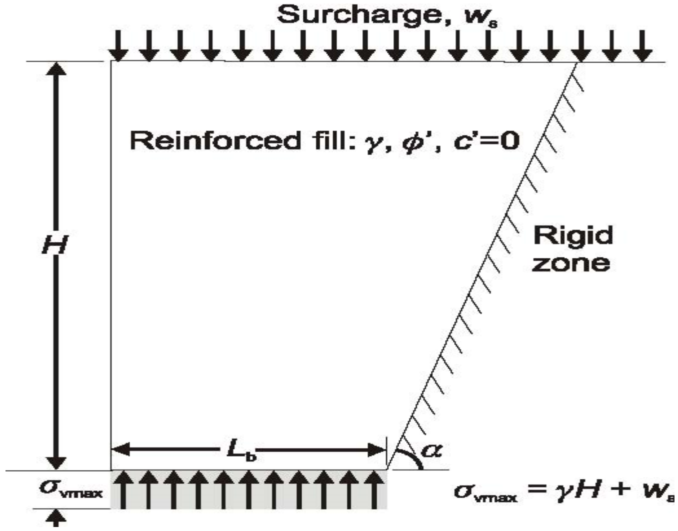

Figure 4. Maximum vertical stress at the base of the constrained reinforced soil wall

The maximum vertical stress acting across the base of the reinforced soil wall is shown in Figure 4. For the reasons state above, the maximum vertical stress is uniform across the width of the reinforced fill zone and is equal to the full self weight of the reinforced fill plus any surcharge or other vertical external loading effects.

In determining the gross horizontal equilibrium force acting on the wall face the constrained nature of the reinforced fill must be considered. Within the constrained reinforced fill zone the active failure wedge may not be able to develop fully because of the relative close proximity of the rigid zone at the rear of the reinforced fill. The general geometry of the problem is shown in Figure 5a where the extent of the reinforced fill zone is described in terms of H, the wall height, and L_t, the width of the top of the fill zone. While the boundary of the rigid zone is shown as vertical in Figure 5a, in fact, the following analysis is also applicable where the rigid zone boundary is inclined (i.e, at angle α in Figure 4).

The forces acting on the fill zone are shown in Figure 5b. The destabilizing force is due to the weight of the fill, W, within the potential failure surface. For simplicity, it is assumed that the equilibrium force acting on the rear of the wall face is horizontal and the wall face is vertical. Thus, the equations for equilibrium are:

If $L_t/H < cot\left(45^{\circ} + \varphi'/2\right)$;

$$\frac{P_h}{0.5\gamma.\ {}^{2}} = K = \frac{L_t}{H}\left(2 - \frac{L_t}{H}tan\theta\right)\frac{sin\theta - tan\varphi' cos\theta}{cos\theta + tan\varphi' sin\theta} \qquad \text{(1a)}$$

If $L_t/H \geq cot\left(45^{\circ} + \varphi'/2\right)$

$$\frac{P_h}{0.5\gamma.\ {}^{2}} = K = cot^{2}\left(45^{\circ} + \varphi'/2\right) = K_a \qquad \text{(1b)}$$

where, the above variables are described in Figure 5b, and K is the horizontal force coefficient acting on the rear of the wall facing.

Figure 5c shows values of K for various L_t/H ratios for a fill internal friction angle $\varphi' = 30°$. For Lt/H ratios greater than 0.5 the full active wedge can develop wholly within the fill zone and hence the value of $K = Ka = cot^2(45° + \varphi'/2)$, Equation 1b. For L_t/H ratios less than 0.5 the full active wedge cannot develop wholly within the fill zone and hence the value of K decreases for decreasing ratios of L_t/H. The value of K for a constant wedge angle $\theta = 45°+\varphi'/2$ is shown plotted in Figure 5c for various values of L_t/H. However, this value of K is not the maximum value that can be attained for a specific L_t/H ratio. The reason for this is that the constrained fill forces the critical failure surface to a greater wedge angle θ than what would normally be the case under full active conditions. This results in a higher K value than when the wedge angle $\theta = 45°+\varphi'/2$ is used. This maximum horizontal force coefficient K_{max} is shown plotted in Figure 5c for fill internal friction angle $\varphi' = 30°$. There is a significant difference between this K_{max} value and the K value determined from a constant wedge angle $\theta = 45°+\varphi'/2$ value especially at low values of L_t/H.

Figure 5d shows the plot of the wedge angle θ that yields the maximum horizontal force coefficient K_{max} for a fill with internal friction angle $\varphi' = 30°$. For L_t/H ratios greater than 0.5 where the full active wedge can develop within the fill the wedge angle is $\theta = 45°+\varphi'/2$. For L_t/H ratios less than 0.5 the wedge angle θ coincides closely, but not exactly, with the juncture of the fill and the rigid zone at the top of the wall.

Figure 5e shows the calculated maximum horizontal force coefficients K_{max} for fill types in the range $\varphi' = 25°$ *to* $45°$. Figure 5f shows the wedge angle θ that may be used in Equation 1a to calculate the maximum horizontal force coefficients K_{max} for fill types in the range $\varphi' = 25°$ *to* $45°$.

Differentiating the gross horizontal force P_h with respect to depth at the wall face yields the horizontal stress distribution acting on the rear of the wall face in Figure 6.

For $0 \leq z \leq L_t tan(45° + \varphi'/2)$

$$\sigma_h = \cot^2(45° + \phi'/2)(\gamma z + w_s) \tag{2a}$$

For $L_t tan(45° + \varphi'/2) \leq z \leq H$

$$\sigma_h = \frac{L_t}{z}\left(2 - \frac{L_t}{z} tan\theta\right)\left(\frac{sin\theta - tan\varphi' cos\theta}{cos\theta + tan\varphi' sin\theta}\right)(\gamma z + w_s) \tag{2b}$$

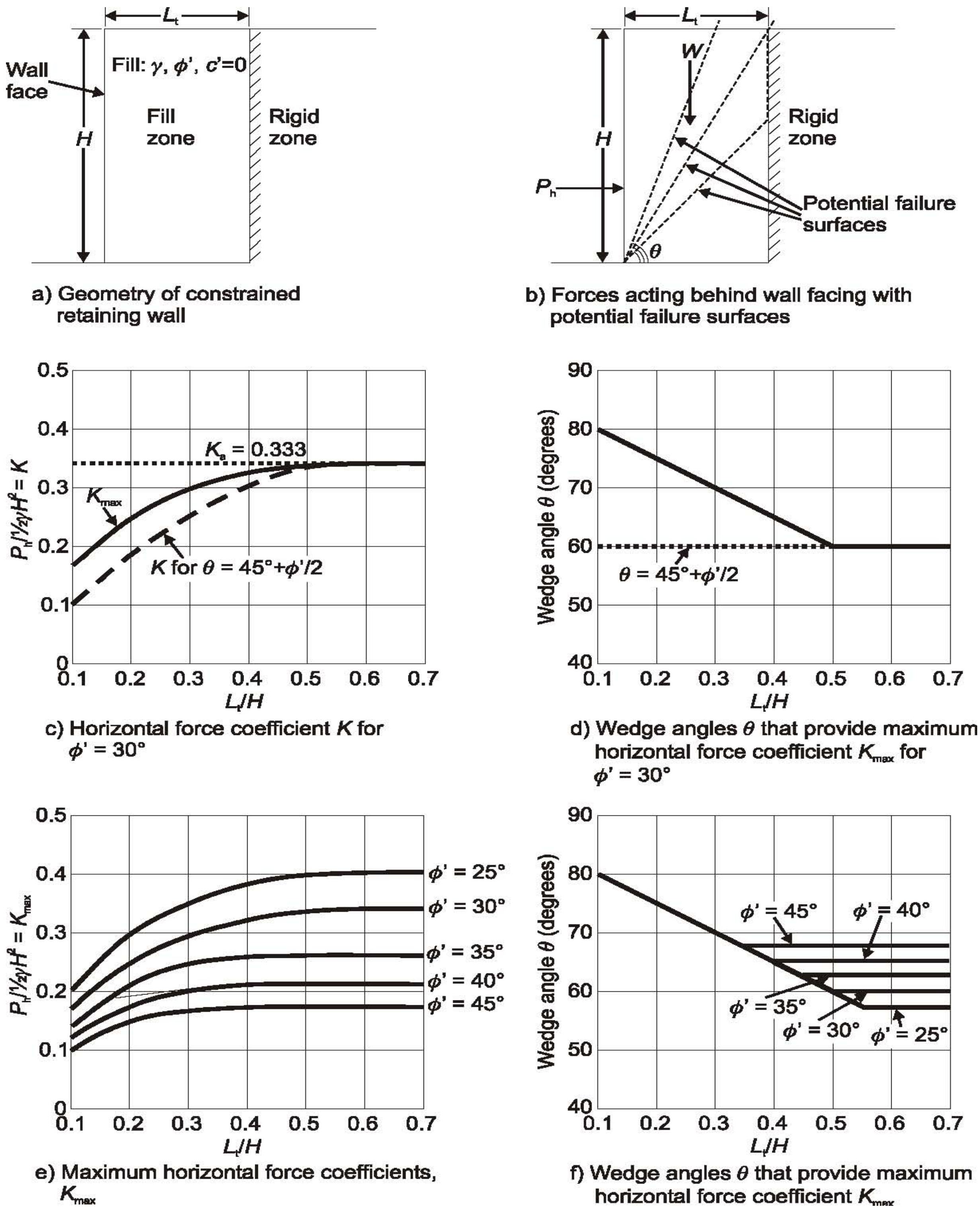

Figure 5. Horizontal force coefficient acting on rear of wall facing due to weight of reinforced fill

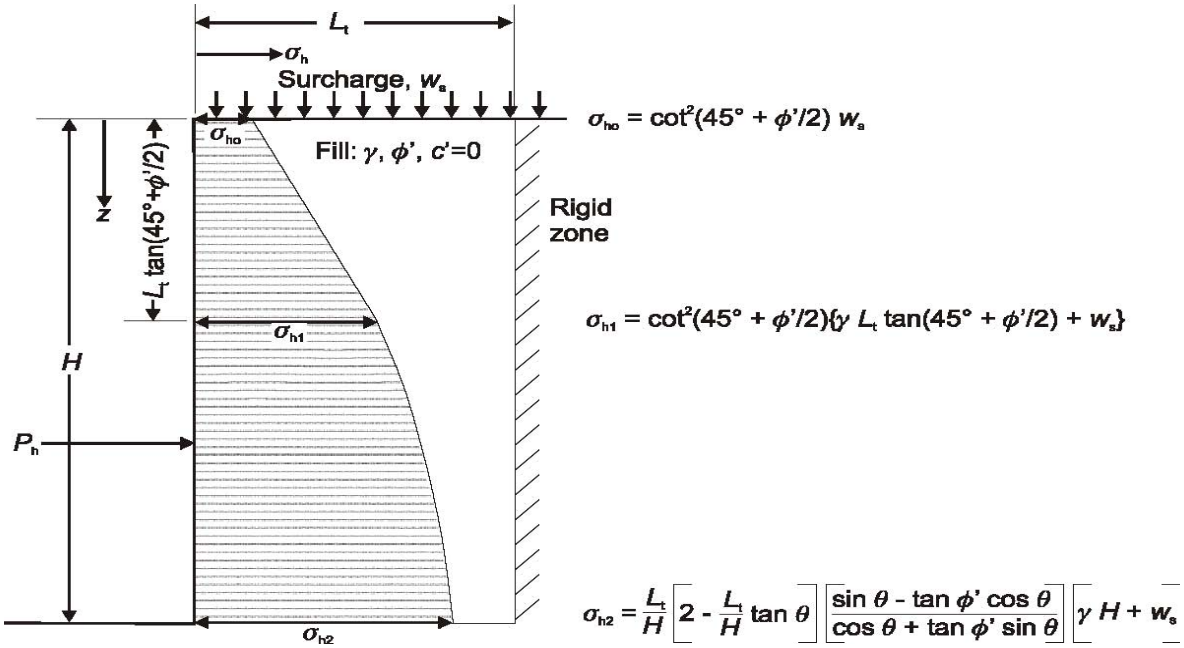

Figure 6. Horizontal stress distribution acting on rear of wall facing due to self weight and surcharge and the gross horizontal force P_h is;

$$P_h = 0.5\frac{L_t}{z}\left(2 - \frac{L_t}{z}tan\theta\right)\left(\frac{sin\theta - tan\varphi' cos\theta}{cos\theta + tan\varphi' sin\theta}\right)z(\gamma z + 2w_s) \qquad (2c)$$

where, θ is the appropriate critical wedge angle obtained from Figure 5f.

In the region between σ_{ho} and σ_{h1} in Figure 6 the horizontal stress distribution increases linearly with depth while in the region between σ_{h1} and σ_{h2} the horizontal stress distribution increases logarithmically with depth.

TENSIONS IN THE REINFORCEMENTS AND ANCHORS

Tensions are generated in the reinforcements and anchors which provide internal stability for the reinforced soil wall. Figure 7a shows the stress regime at level *j* in the wall. A vertical stress σ_{vj} is generated due to the weight of the fill plus surcharge and other external loads. This vertical stress imparts a horizontal stress on the rear of the wall facing which is resisted by the generation of a tension force in the reinforcement, T_{maxj}. If this tension force is not dissipated on reaching the end of the reinforcement at the boundary with the rigid zone then the residual tension is transferred to the anchor, P_{anj}, for dissipation within the rigid zone.

The location of maximum tension within each reinforcement layer in the reinforced soil wall coincides with the juncture between the active and passive zones in the reinforced fill zone defined by the wedge angle θ, Figure 7b.

The maximum tension generated in the reinforcement, T_{maxj}, at level *j* in the wall is;

$$T_{maxj} = \sigma_{hj} S_{vj} \qquad (3)$$

where, σ_{hj} is the horizontal stress on the rear of the face of the wall at level j, and S_{vj} is the vertical spacing between the reinforcement layers at level j in the wall.

Figure 8 sets out the assumed tension distribution along the length of the geogrid reinforcement at level j in the wall. For conservatism and ease of design it has been assumed that the tension developed in the reinforcement at the wall face is equal to the maximum tension generated, i.e. $T_{facej} = T_{maxj}$.

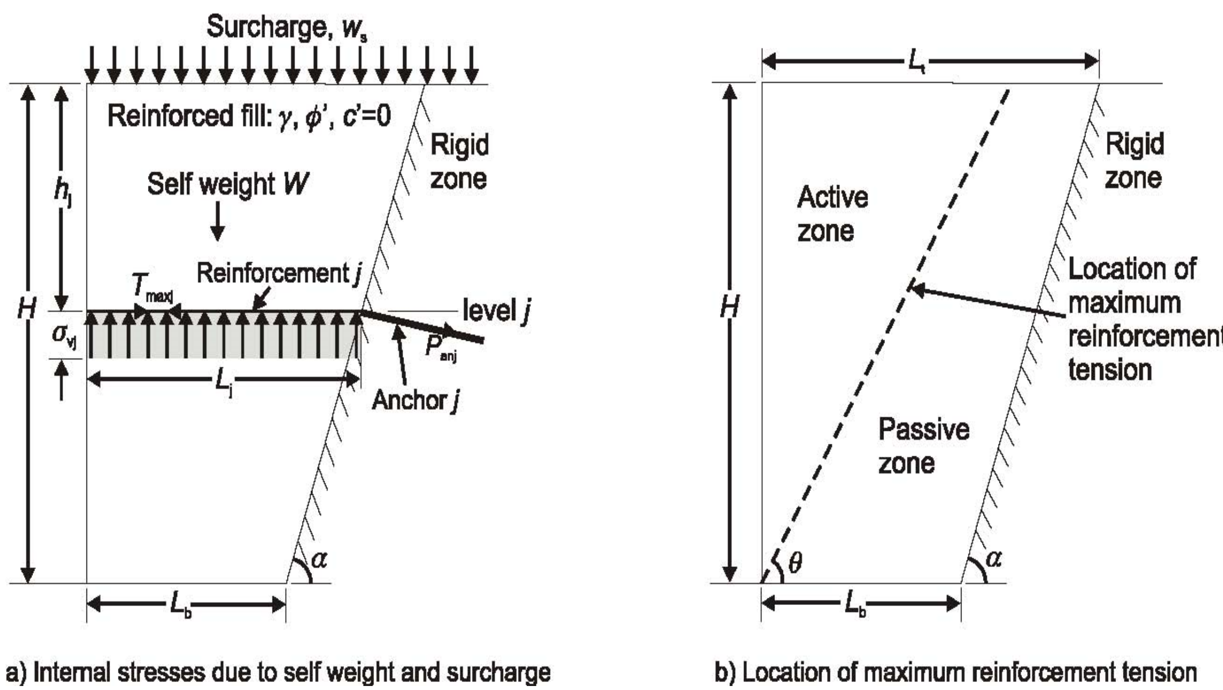

Figure 7. Internal stresses and location of maximum reinforcement tension

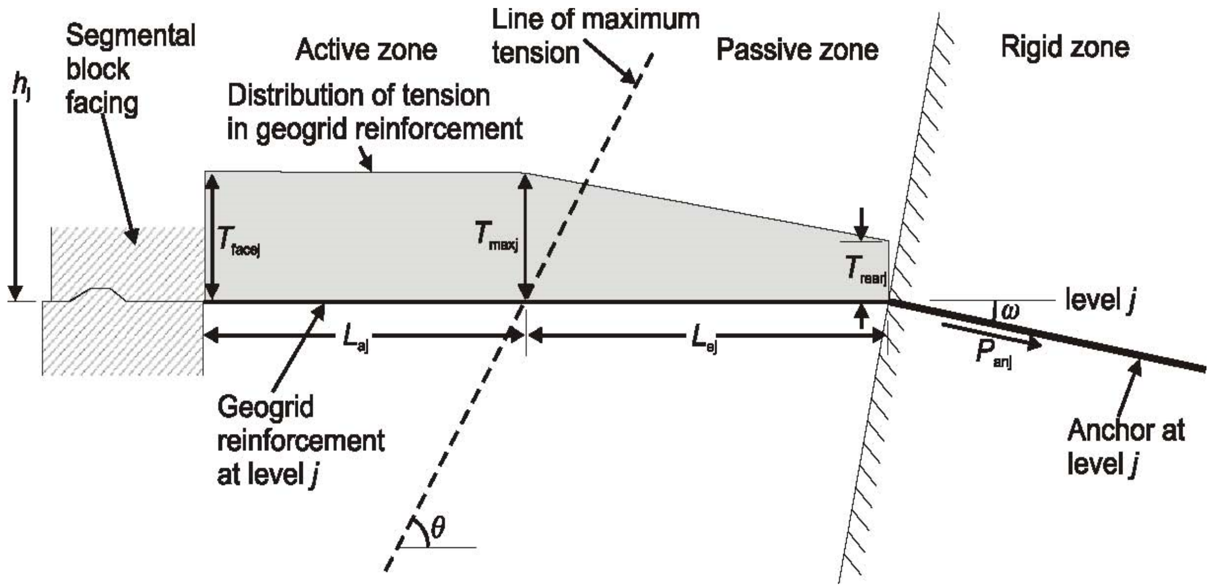

Figure 8. Distribution of tension along a layer of reinforcement

If the line of maximum tension occurs at some distance from the rigid zone boundary, Figure 8, then the tension generated in the reinforcement can dissipate within the passive zone of the reinforced fill. The rate of tension dissipation within the passive zone is shown in Figure 9. It is governed by the overburden stress $\gamma h+w_s$, and the reinforcement/fill bond $a'tan\varphi'$.

The tension retained in the reinforcement at the rear of the passive zone, T_{rearj}, at level j in the wall is;

$$T_{rearj} = T_{max\,j} - 2(\gamma h_j + w_s)a' \tan\phi' L_{ej} \tag{4}$$

where, T_{maxj} is the maximum tension occurring in the reinforcement at the juncture of the active and passive zones at level j in the wall, γ is the unit weight of the reinforced fill, h_j is the height of the wall above level j, w_s is the surcharge on top of the wall, a' is the reinforcement/fill bond coefficient, φ' is the angle of internal friction of the reinforced fill, and L_{ej} is the reinforcement length in the passive zone at level j in the wall.

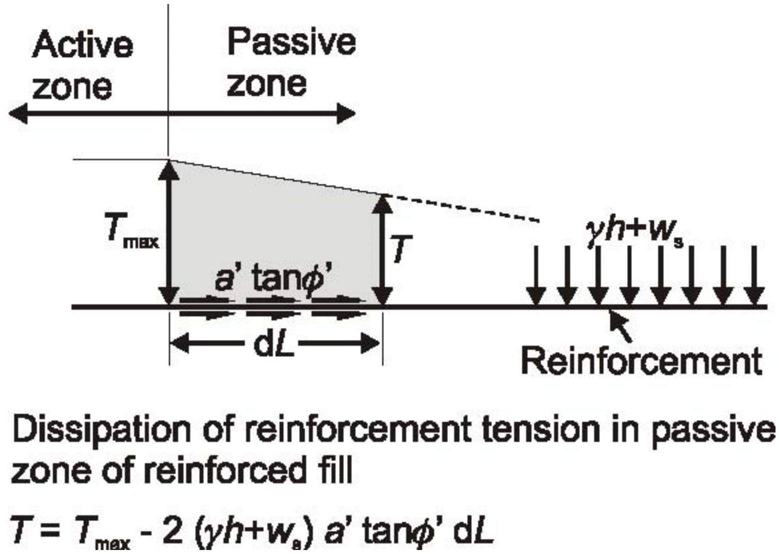

Figure 9. Assumed linear profile of reinforcement tension dissipation in the passive zone of the reinforced fill

If the tension in the reinforcement has not been fully dissipated on reaching the boundary of the rigid zone then anchors will be required to dissipate this residual tension within the rigid zone. The magnitude of the working load in each anchor, P_{anj}, will be;

$$P_{anj} = T_{rearj} S_{hj} \sec\omega \qquad (5)$$

where, P_{anj} is the working load in the anchor at level j in the wall, T_{rearj} is the tension in the reinforcement at the rear of the passive zone at level j in the wall, S_{hj} is the horizontal spacing between adjacent anchors at level j in the wall, and ω is the angle of inclination to the horizontal of the anchor at level j in the wall.

TYPES OF ANCHORS USED

The anchors used to connect the reinforcements into the rigid zone are relatively low capacity, non-stressed anchors or soil nails. These can be of the form of bonded passive anchors and nails, rotational dead-man tieback anchors, or driven nails. Consideration needs to be given to the required design life of the anchors because they must perform for the full design life of the reinforced soil wall. In some cases this design life may be as long as 120 years.

The connection between the anchor and the geogrid reinforcement is normally effected by a circular steel bar or pipe. This type of connection requires the use of flexible geogrids with negligible bending resistance to ensure stress concentrations do not occur at the connection.

REINFORCED SEGMENTAL BLOCK RETAINING WALL AT PA JU, SOUTH KOREA

The landscaping and earthworks for an apartment complex at Pa Ju, North of Seoul, South Korea required the construction of several reinforced segmental block retaining walls. At one location a 130 m long retaining wall was required, ranging in height from 3 m to 8 m.

The area where this wall was to be constructed consisted of partially decomposed quartzite. In order to minimize risk to an adjacent building the owner decided not to excavate to construct a conventional geogrid reinforced segmental block wall. Instead a combination geogrid and anchor reinforced segmental block wall was constructed.

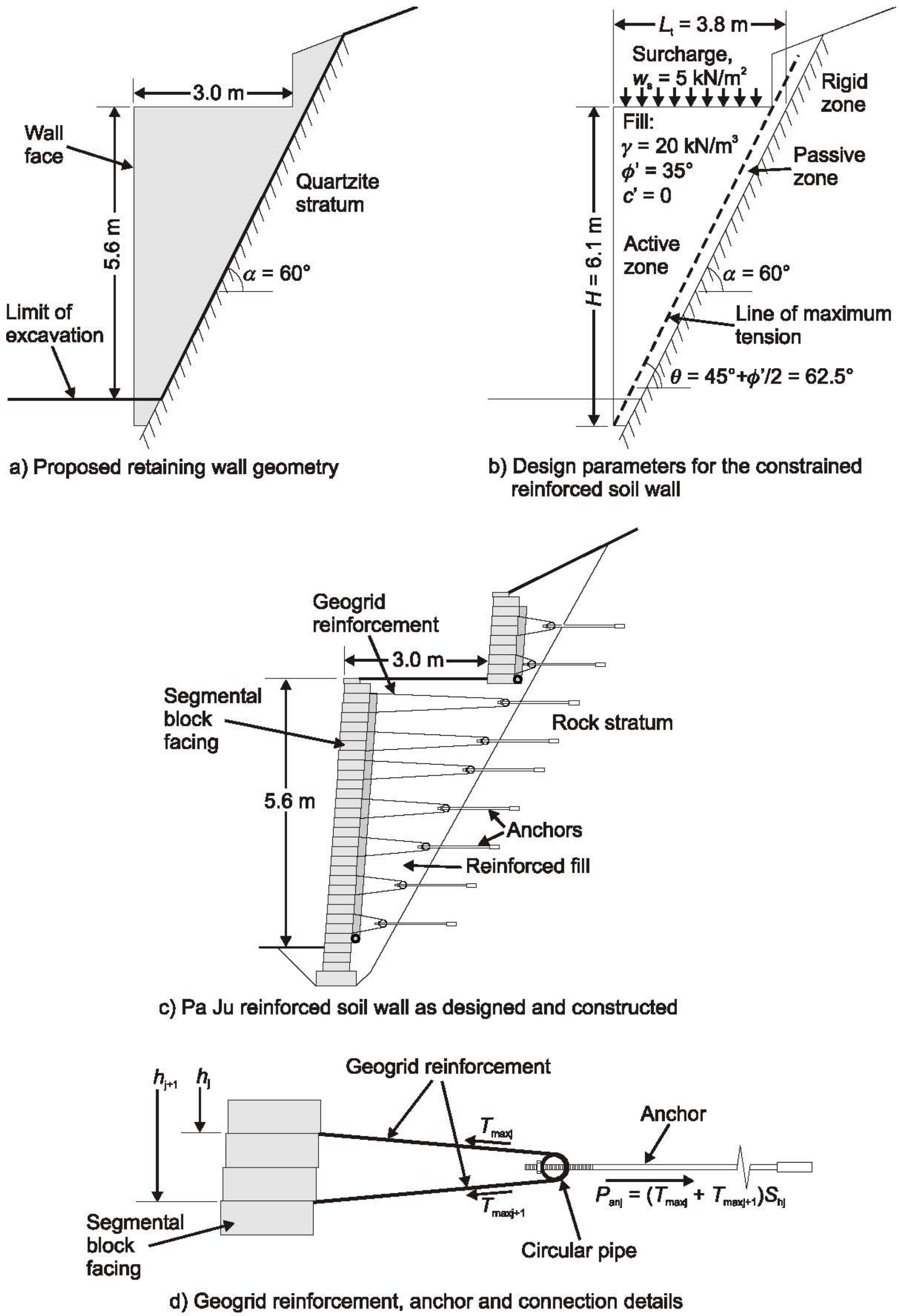

Figure 10. Details of Pa Ju reinforced segmental block retaining wall

Figure 10a shows the proposed retaining wall geometry in relation to the location of the quartzite stratum whose surface was inclined approximately 60° to the horizontal. The design called for a wall of exposed height 5.6 m with a crest width of 3.0 m.

Figure 10b shows the various design parameters used for the constrained reinforced soil wall. The ratio $L_t/H = 3.8/6.1 = 0.62$ gives a critical wedge angle $\theta = 45°+\varphi'/2 = 62.5°$ (Figure 5f). Because of the wall geometry the full active wedge can develop within the reinforced fill zone, and the line of maximum reinforcement tension is shown plotted in Figure 10b. The passive zone within the reinforced fill is very limited and corresponds approximately with the boundary of the rigid zone (the quartzite stratum). Consequently, for the design of the wall it was assumed that the maximum tension developed in the reinforcement occurs at the boundary of the rigid zone, i.e. at the face of the quartzite stratum.

The segmental wall facing consisted of Anchor Vertica Pro blocks. The horizontal stress distribution acting against the rear of the wall facing was determined in accordance with Equation 2, and the appropriate geogrid vertical spacing was determined on the basis of geogrid design strength and the connection capacity between the geogrid reinforcement and the segmental block facing. For this wall it was planned to economize on the number of anchors by utilizing one row of anchors for every two layers of geogrid reinforcement, Figures 10c and 10d. The original design called for 260 Mantaray MR2 earth anchors to be installed into the partially decomposed quartzite stratum. These were to be installed by pre-auguring into the partially decomposed quartzite back slope prior to anchor insertion. The detailed layout of the reinforced segmental block wall is shown in Figure 10c.

The connection between the anchors and the geogrid reinforcement was effected by the use of a 75 mm diameter galvanized steel pipe, Figure 10d. Miragrid geogrids were used as the geogrid reinforcement as these exhibited the required long-term design strengths and had the required level of bending flexibility which enabled them to pass easily around the galvanized steel pipe without attracting additional tensile stress.

During construction it was found that the quartzite stratum was harder than originally anticipated. Consequently, drilled rock bolts had to be substituted for the majority of the Mantaray earth anchors. A total of 208 rock bolts were used in addition to 52 Mantaray MR2 earth anchors.

CONCLUSIONS

The paper provides an analytical model for reinforced soil walls where the reinforcements consist of a combination of geogrids and anchors (or nails). The technique is particularly suited to situations where a rigid zone constrains the extent of the reinforced fill zone. In this case residual tensions in the geogrid reinforcements within the reinforced fill zone are transferred into the rigid zone by means of anchors or nails. A major consideration is that all components must fulfill the design life requirements of the retaining wall.

The geogrid and anchor reinforced segmental block wall at Pa Ju, South Korea has demonstrated that this combination reinforcement technique can be used successfully.

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