Effects of Hydrostatic Pressure on Limit State Design of Reinforced Soil Wall Internal Stability

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Abstract: Current design of reinforced soil walls in Malaysia generally does not consider action due to water (e.g. hydrostatic pressure, seepage forces) because free-draining granular backfill material is used for the wall. However, numerous investigations of reinforced soil wall failures revealed that low quality backfill materials coupled with a build up of pore pressure behind the wall are responsible for the collapses. This paper demonstrates the importance of hydrostatic pressure on the internal stability of soil reinforcing systems. From the analysis of a design example, it was observed that when hydrostatic pressure builds up, it increases the tensile forces in the reinforcements and decreases the soil-reinforcement friction. In view of this, the effects of hydrostatic pressure on reinforced soil wall internal stability should be carefully assessed in the design analysis.

1 INTRODUCTION

Reinforced soil (RS) walls have been used in a variety of applications since their first appearance in France in the early 1960s. Although many reinforced soil walls have been safely constructed and are performing well to date, there are many areas that need in depth studies in order to better understand their mechanical behaviour under more aggressive and harsh environments. Thus, the design methodology for reinforced soil structures is in a state of transition. In addition, reinforced soil wall applications in aquatic sites such as lakefronts, seawalls and storm water runoff channels require additional specialised engineering considerations as the current technology is normally used in dry conditions.

Many reinforced soil wall failures are caused by poor drainage, leading to the building up of pore water (hydrostatic) pressure, which in turn generates a destabilising force that has not been considered in the standard design methodologies. As such, in a high intensity rainfall region like Malaysia and Southeast Asia, it is advisable to carefully assess the potential build up of hydrostatic pressure in reinforced soil walls, particularly when less permeable cohesive frictional fill materials are used in high walls where water needs a longer time to discharge from the wall.

This paper focuses on a study of hydrostatic pressure effects on soil reinforcing systems which is generally not carried out in the current limit state design computations. A design example is carried out to demonstrate the effects of hydrostatic pressure on the internal stability of walls. Findings of the study and recommendations are presented and discussed.

2 CURRENT DESIGN APPROACHES AND PRACTICES

2.1 *Limit State Design*

The philosophy of limit state design is to ensure that none of the ultimate limit states (i.e. collapse or major damage) or the serviceability limit states (i.e. deformations in excess of acceptable limits, other forms of distress or minor damage) occurs during the service life of the reinforced soil wall. All potential modes of failure must be considered, including internal and external stability. Ultimate limit states are deemed to be reached when, for a specific mode of failure, the disturbing forces are equal to or exceed the restoring forces. The disturbing forces originate from self weight, groundwater and live loads, whereas the restoring forces are derived from the soil and the reinforcement. Practice in soil reinforcement is to design against the ultimate limit state and check for the serviceability limit state.

Current design approaches of reinforced soil walls (e.g. BS 8006, FHWA, FMOT) generally do not explicitly consider actions due to water in the backfill materials in the computation of disturbing forces, as well as restoring forces, due to the use of high permeability granular material as backfill. However, the above assumption is not always safe and special consideration should be given to amend the design equations where it is anticipated that building up of hydrostatic pore water pressures will affect the strength of the fill materials due to reduction in effective stress.

2.2 *Backfill Materials*

Generally, the selection of backfill material for reinforced soil walls must satisfy the mechanical criterion of soil-reinforcement friction. Most of the specifications indicate that the backfill material used should be a frictional fill material or cohesive frictional fill material free from organic or other deleterious material. The majority of the walls constructed in Malaysia use high permeability frictional fill materials (e.g. sand, quarry dust, crusher run). Sometimes cohesive frictional fill material may be allowed in certain sections of anchored earth systems with careful design.

For practical reasons, the criterion of friction is commonly replaced by criterion concerning the particle size distribution (grading) of the backfill materials. The predominant factor is the percentages in weight of fine particles. Having said that, fines (material passing a 63 μm test sieve) of 15 % are usually permitted in most of the specifications, thus they do not satisfy the requirements of a good free draining material, as the gravity drainage is too slow according to the general guide presented by Courtesy Moretrench American Corporation (Powers, 1981) (see Fig. 1).

Therefore, build up of hydrostatic pressure within the reinforced soil mass can occur should backfill material of poor draining be used or the wall is very high with poor internal drainage thus causing water to take a longer time to discharge fully from the wall.

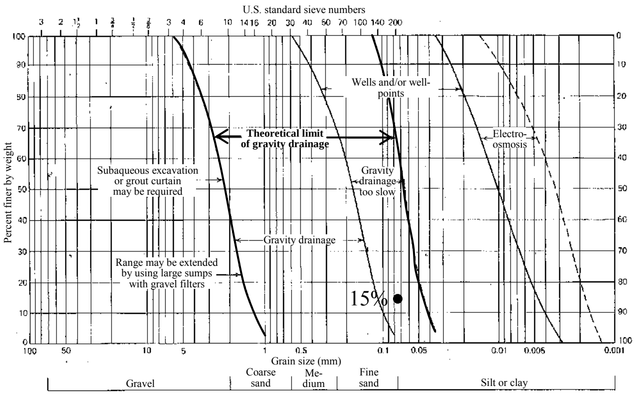

Fig. 1 Gradation of soil particles on gravity drainage (Powers, 1981).

3 BEHAVIOUR OF SOIL-REINFORCEMENT INTERACTION DUE TO WATER

The actions of water, or more generally related to water, modify the material characteristics as follows:-

(a) the phenomenon of uplift according to Archimedes' Principle

(b) the reduction in soil-reinforcement friction

3.1 *Phenomenon of Soil Buoyancy*

The water that accumulates behind the wall exerts full hydrostatic pressure but the earth effective overburden pressure is reduced because of the buoyancy effect of water on the weight of the submerged soil. The addition of disturbing pressure from hydrostatic origin is directly transferred to the reinforcements. Therefore, water can add excessive lateral pressure behind a wall that can ultimately cause the wall to fail even at low heights if not properly designed.

3.2 *Variation of Soil-reinforcement Friction Capacity*

The law of friction mobilization relates to the normal stress exerted on the reinforcement. However, the normal stress exerted on the reinforcement is generally unknown and therefore an apparent friction coefficient, f* has been introduced by the relationship:

$$f^* = \frac{T_{max}}{\sigma_v'} \tag{1}$$

where T_{max} is the maximum shear stress which can be mobilized on the reinforcement faces and σ_v' is the average effective vertical stress resulting from the overburden stress above the considered reinforcement. From this equation, it is clear that buoyancy effect on the soil will reduce the effective vertical stress, resulting in decreasing soil-reinforcement friction capacity.

4 DESIGN EXAMPLE

Consider a simple reinforced soil wall to retain a 10m high fill-as an example. To conform to the design situation imposed in BS 8002: Section 3.2.2.2, the retained surcharge will be 10kPa on top and behind the reinforced fill. It will be assumed that the pre-existing groundwater table was high, so the phreatic surfaces in both the short term, when water rises faster than discharge capacity and in the long term will be taken at the various levels of $0.25H_{wall}$, $0.5H_{wall}$, $0.75H_{wall}$ and $1.0H_{wall}$ from the bottom of wall at the retained side for parametric study. The objective of this paper is to set out some simple design calculations considering hydrostatic pressure. Fig. 2 shows the design cross-section.

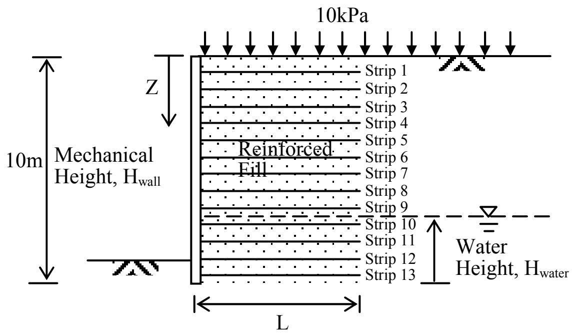

Fig. 2 Design cross-section.

Table 1 summarizes the engineering properties of the typical soils adopted in this design example. The internal friction angle at a density corresponding to the as-compacted state is approximately 36° and the apparent cohesion is disregarded as the current design guideline requires. The wall is assumed to be situated in a residual soil stratum with an internal friction angle of 30° and cohesion of 5kPa.

Table 1 Engineering properties of the soils.

Material	γ_b (kN/m^3)	c' (kN/m^2)	ϕ_p' (°)
Granular Reinforced fill for RS wall	20	0	36
Backfill behind wall (e.g. residual soil)	18	5	30

Limit equilibrium internal stability analyses were conducted based on the BS 8006: 1995 design approach to check for rupture, adherence (for reinforced earth systems) and pull-out (for anchored earth systems) failures. The coherent gravity method was adopted as the wall is reinforced by inextensible reinforcement (e.g. steel). For parametric study, an in-house computer spreadsheet which can incorporate hydrostatic pressure in the wall was used. This program was developed based on the BS 8006 design approach. These analyses were aimed at gaining insights into the effects of hydrostatic pressure on the internal stability of reinforced soil structure.

The partial factors were applied to each component of load based on load combination B in Table 17 of BS 8006, which is the most critical. In this combination, the effects due to the weight of soil behind the wall structure are multiplied by 1.5. However, the effects of hydrostatic thrust are not multiplied by a load factor (FMOT, 1980). This combination considers the maximum overturning loads together with minimum self mass of structure and superimposed traffic load. This combination normally dictates the reinforcement requirement for pull-out resistance and is normally the worst case for sliding along the base.

For simplicity, a uniform distribution of identical reinforcing elements length (L) of $0.7H_{wall}$ (i.e. 7m) was used throughout the height of the wall. For reinforced earth systems, a typical reinforcement strip size of 50mm x 4mm available in the Malaysian market is adopted for the analysis. Corrosion allowance of 0.5mm per face as recommended by BS 8006 is considered in addition to galvanizing of the steel reinforcement. Both smooth reinforcement and highly adherent (H.A.) reinforcement were considered (see Figure 3).

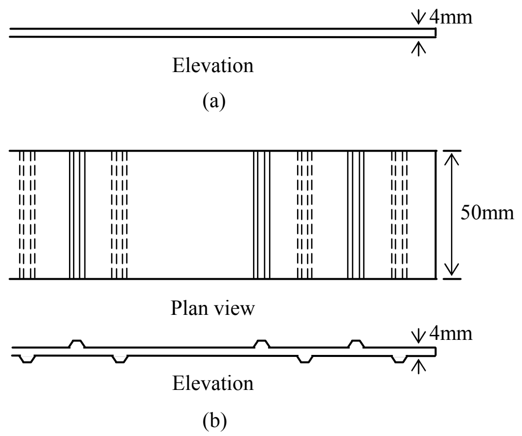

Fig. 3. (a) Smooth reinforcement (b) Highly adherent (H.A.) reinforcement.

The distribution of the shear stress is assumed constant along the reinforcement for simplification. In the case of smooth reinforcement (as shown in Fig.3(a)), the coefficient f* is constant all over the reinforced fill mass. However, in the case of highly adherent reinforcement (as shown in Fig.3(b)), the effect of dilatancy leading to increased soil-reinforcement friction at shallow depths is given by the following equations as a function of depth z measured from the level of the mechanical height (FMOT, 1980):

For $z \le z_o = 6$m

$$f^* = f_o^* \,(1 - z / z_o) + \tan \phi' \,(z / z_o) \qquad (2a)$$

For $z > z_o$

$$f^* = \tan \phi' \qquad (2b)$$

where f_o^* = Soil-reinforcement friction coefficient at top of wall mechanical height (Figure 2.21, FMOT)
z = Depth from top of wall
ϕ' = Effective angle of friction for backfill materials

While for anchored earth systems, a typical reinforced block 200mm x 200mm x 100mm thick with a reinforcement bar of T16 is adopted to check for pull-out failure as shown in Figure 4. All the reinforcement strips should be connected to the concrete panel at 0.5m centre to centre spacing horizontally and 0.75m centre to centre vertically in order to satisfy the initial internal stability.

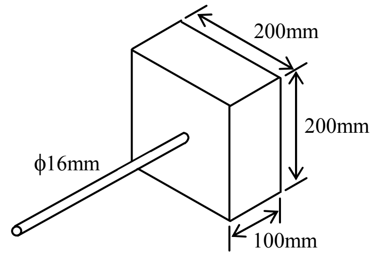

Fig. 4 Anchor reinforcing element.

5 RESULTS AND DISCUSSION

5.1 *Tensile Force*

Fig. 5 shows the variation of maximum acting tensile force, T_j to be resisted by each layer of reinforcement due to the effect of hydrostatic pressure. This force is mainly contributed by vertical loading due to self weights plus any surcharge and bending moment caused by external loading acting on the wall based on the vertical stress estimated by assuming a uniform distribution according to Meyerhof's method and using a reduced width of the strip.

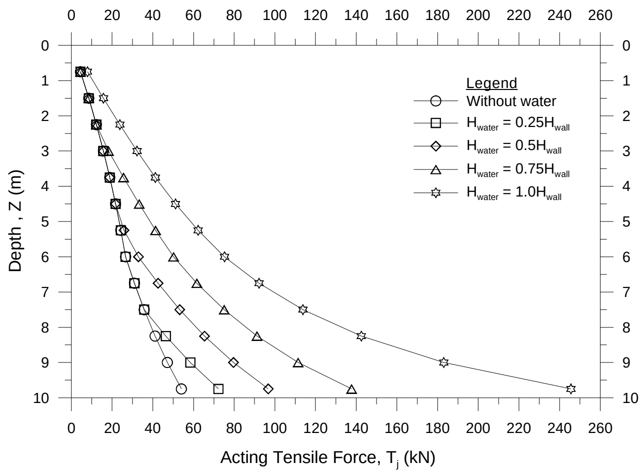

Fig. 5 Variation of maximum acting tensile force.

In the event of hydrostatic pressure existing, the effective overburden stress will reduce due to the buoyant weight of the soils. However, the increase in total horizontal stress due to hydrostatic pressure is more significant as compared to the decrease in overburden stress. Moreover, the external acting force due to water behind the reinforced fill will increase the eccentricity of resultant vertical load at each level of the wall and subsequently increases the repartition vertical stress exerted on reinforcement.

In this context, the efficiency of wall block size due to hydrostatic pressure was studied for typical wall block sizes ranging from $L = 0.4H_{wall}$ to $1.0H_{wall}$. The sensitivity of increase in acting tensile force for each layer of reinforcement with various water levels is presented in Fig 6. It should be noted that the impact of the above phenomenon is far more notable in the bottom levels of reinforcement with increasing water level. In addition, the increment of tensile force is fairly consistent for wall block size, $L \ge 0.7H_{wall}$, and water level, $H_{water} \le 0.5H_{wall}$. Nonetheless, it is critical for wall block size, $L < 0.7H_{wall}$, in which wall toppling or overturning would be more likely with the presence of water.

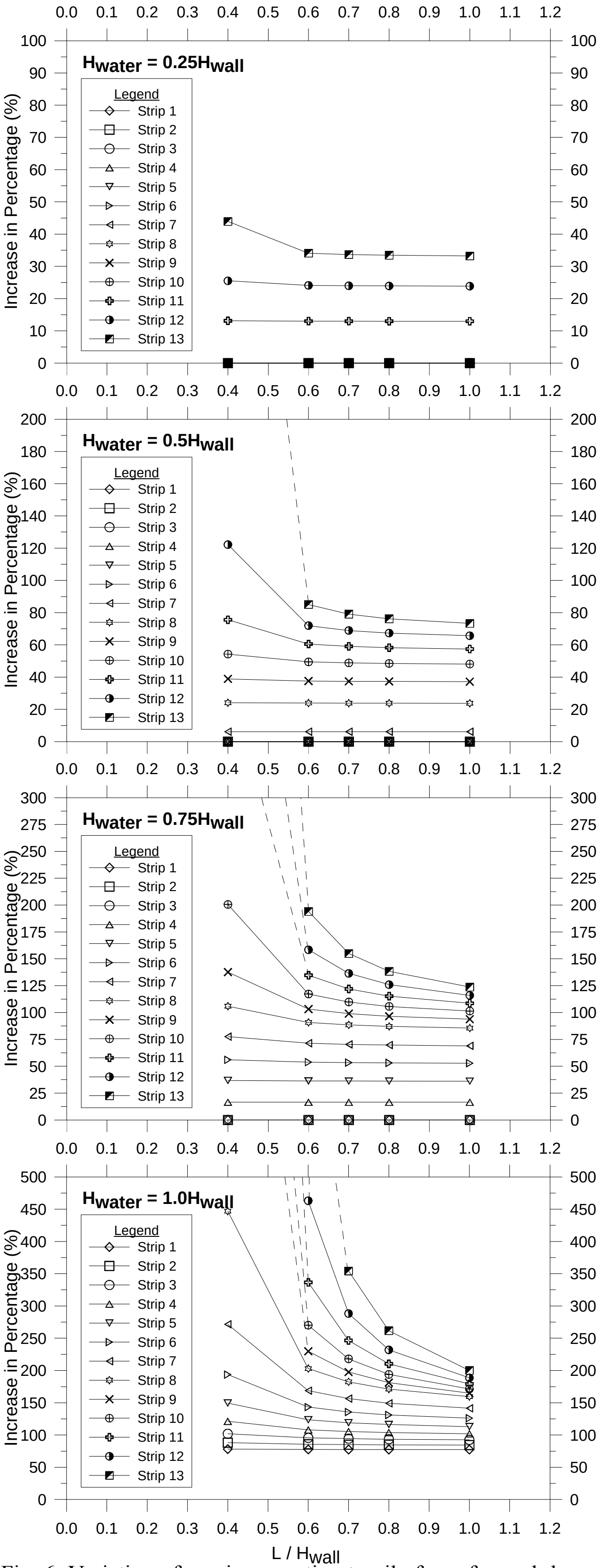

Fig. 6. Variation of maximum acting tensile force for each layer of reinforcement.

Fig. 7 summarizes the average addition in percentage of maximum acting tensile force of last bottom reinforcement due to hydrostatic pressure. This chart should be applicable to all wall heights up to 20m except low wall of less than 6m height, which overestimated the additional acting tensile force.

This chart can assist in preliminary assessment of disturbing tensile force due to hydrostatic pressure. Careful assessment is required for safe design of reinforced soil walls. Generally, adoption of wall block size L < $0.7H_{wall}$ should be avoided in view of potential overturning problems.

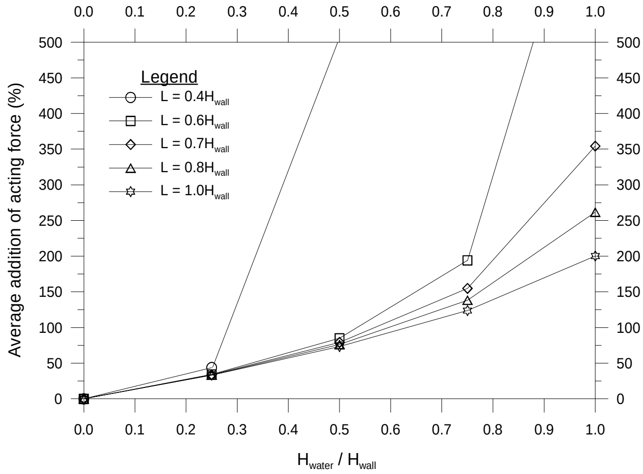

Fig. 7 Average percentage of addition in maximum acting tensile force.

5.2 *Adherence Capacity of the Reinforcement*

For limit state design, adherence should be checked beyond the line of maximum tension and compared with the relevant reinforcement tension at each of these points. Obviously, the buoyant soil under water will reduce the normal stress induced to the reinforcement and subsequently reduce the soil-reinforcement friction and adherence capacity. The reduction of adherence capacity for both smooth reinforcement and highly adherent reinforcement are shown in Fig. 8 and 9 respectively.

It is observed that the reduction in magnitude is relatively constant for each layer of reinforcement for the lower portion of a wall. Further analyses were performed for various wall heights ranging from 5m to 20m and the average percentage of reduction in maximum adherence capacity (last bottom reinforcement) due to water is presented in Fig. 10.

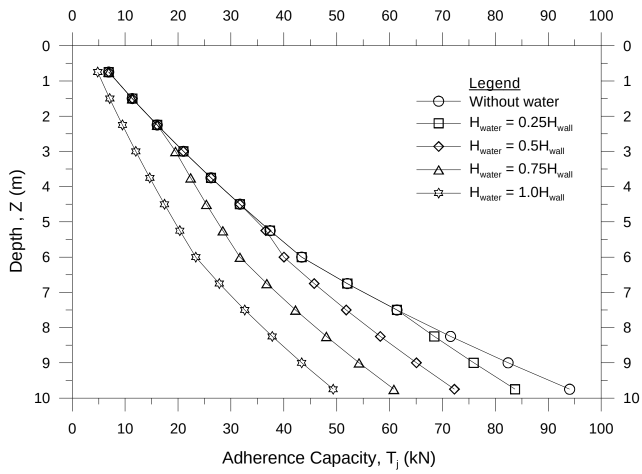

Fig. 8 Variation of adherence capacity for smooth reinforcement.

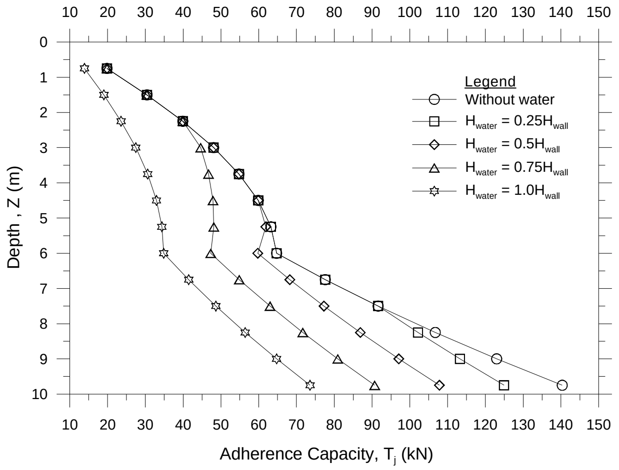

Fig. 9 Variation of adherence capacity for highly adherent (H.A.) reinforcement.

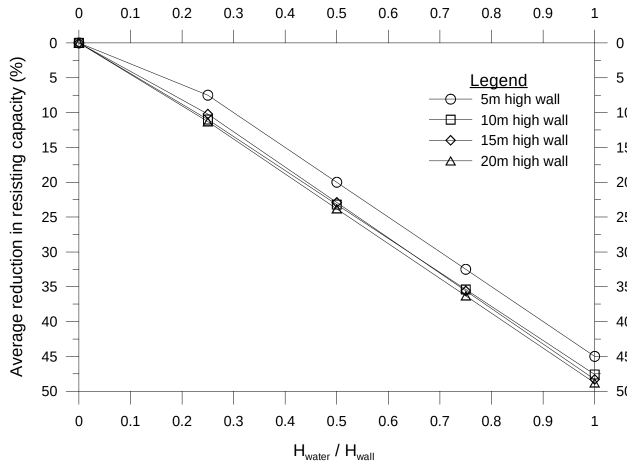

Fig. 10 Average percentage of reduction in resisting adherence or pull-out capacity.

Again, it is noted that the average percentage of reduction in adherence capacity is constant for wall heights of 6m to 20m. It is slightly variable for wall heights less than 6m due to the variation in coefficient of active earth pressure from K_o at the top of the wall reducing linearly with depth to a value of K_a at depth of 6m below the top of the wall as adopted in the coherent gravity method. As revealed from this chart, it could be expected that a reduction of adherence capacity due to hydrostatic pressure can be as much as 50%.

5.3 *Pull-out Capacity of the Anchor*

The pull-out capacity of anchor reinforcing elements to satisfy local stability is determined from the shaft or loop resistance developed by friction beyond the potential failure plane and bearing resistance at each layer of anchors (Cl. 6.6.4.2.3 BS 8006). The effect of hydrostatic pressure is shown in Figure 11.

Again, it is observed that the reduction in magnitude is relatively constant for each layer of reinforcement. This is mainly due to the constant increment in vertical applied pressure at each layer of anchors. It was also found that the average percentage of reduction in maximum pull-out capacity due to water for various wall heights is the same as Figure 10.

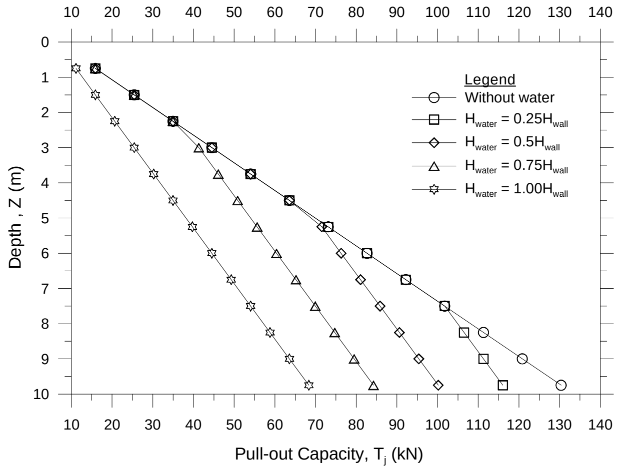

Fig. 11 Variation of pull-out capacity.

5.4 *Summary of Results of Analyses*

Fig. 12 shows the initial resisting capacities for a typical 10m reinforced soil wall design while Figures 13 to 16 show the inadequacy in rupture resistance, adherence and pull-out capacities taking into consideration the varying hydrostatic pressure.

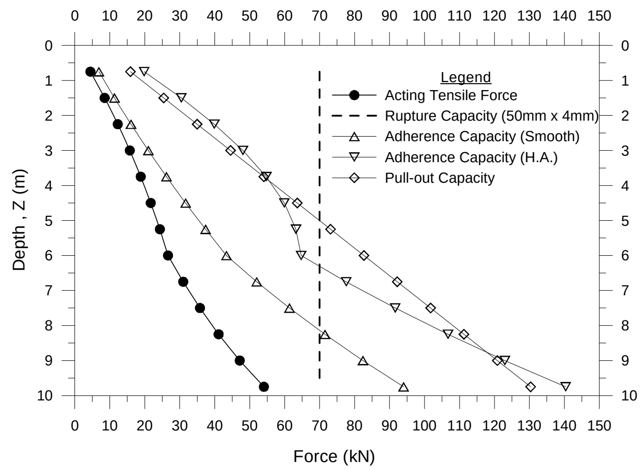

Fig. 12 Initial resisting capacities without water.

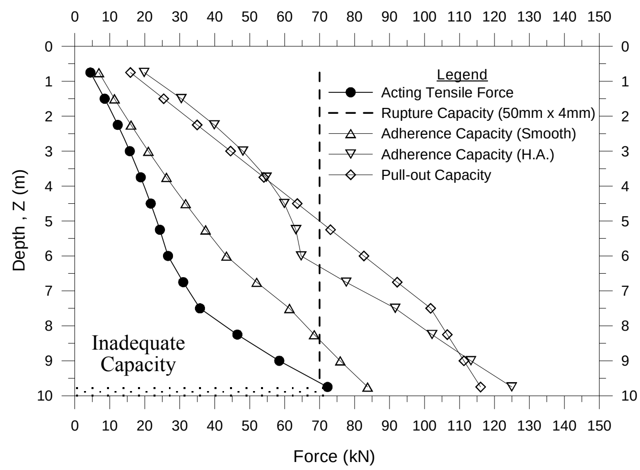

Fig. 13. Effects of hydrostatic pressure ($H_{water} = 0.25H_{wall}$).

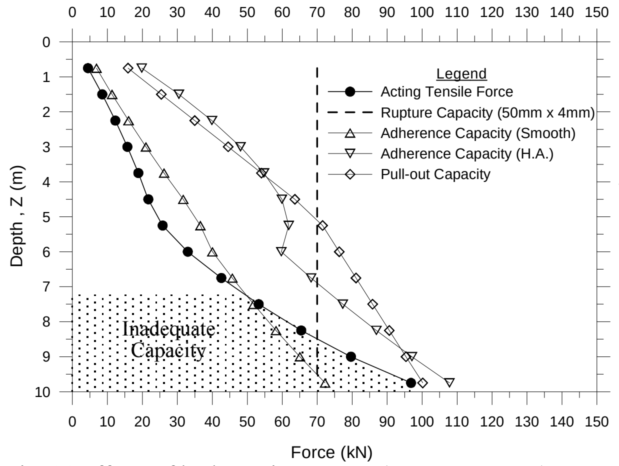

Fig. 14 Effects of hydrostatic pressure ($H_{water} = 0.5H_{wall}$).

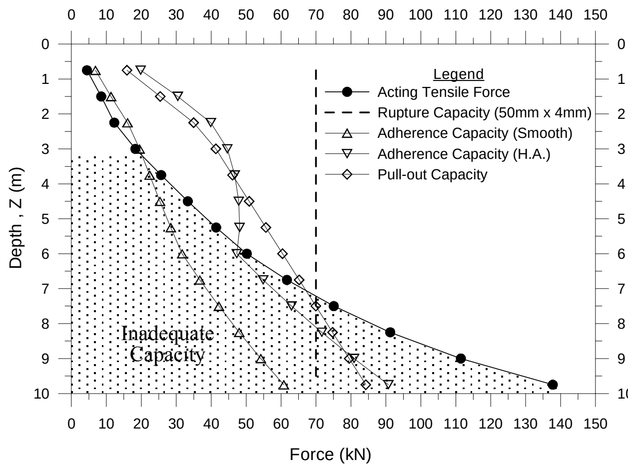

Fig. 15. Effects of hydrostatic pressure ($H_{water} = 0.75H_{wall}$).

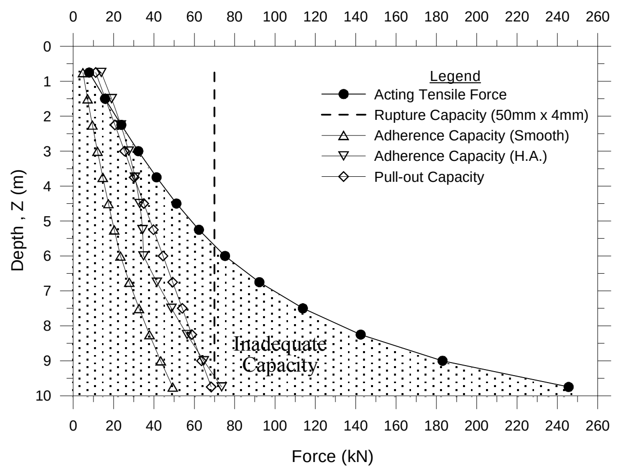

Fig. 16. Effects of hydrostatic pressure ($H_{water} = 1.0H_{wall}$).

From these charts, generally, it was observed that slight inadequacy of rupture capacity at the last bottom reinforcement with the water level (H_{water}) approaching $0.25H_{wall}$. On top of this, inadequacy of adherence capacity (for smooth reinforcement) starts at about Z = 7.5m when the water level is $0.5H_{wall}$. At the water level of $0.75H_{wall}$, both adherence capacity (for both smooth and highly adherent reinforcement) and pull-out capacity are inadequate after Z > 3m and worse if the water level is equal to the wall height.

6 CONCLUSION

Based on the design example above, it can be seen that hydrostatic pressure on a wall can result in multiple negative effects. Generally, when hydrostatic pressure builds up, it will increase the tensile force in the reinforcements and decrease the soil-reinforcement friction capacity. The additional acting tensile force is relatively constant for wall block size, $L \geq 0.7H_{wall}$ due to the resisting moment of block self weight in preventing overturning, but the acting tensile force increases drastically for wall block size, $L < 0.7H_{wall}$. The reduction in resisting capacity of either adherence or pull-out capacity is in proportion to H_{water}/H_{wall} ratio. Therefore, the effects of hydrostatic pressure on reinforced soil wall internal stability should be carefully assessed during design to avoid failure.

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