# Design basis and economic aspects of different types of retaining walls

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**Abstract**

Soil retention system has been revolutionized by the development of internally stabilized walls. Although walls of this type have gained wide acceptance in many parts of the world, Bangladesh is yet to take in such system significantly. The major reason may be the anticipation that such walls would be more expensive compared to the conventional externally stabilized walls, and also that the design procedures involved might be too cumbersome. This paper presents step-by-step design procedures for externally stabilized walls and internally stabilized walls as suggested by different codes/ researchers. Typical design examples of some of the externally stabilized and internally stabilized walls, i.e. design of reinforced concrete cantilever retaining walls, metal strip reinforced walls, geotextile reinforced walls and anchored earth walls of different heights have been provided for the purpose of cost comparison. The analyses reveal that the internally stabilized walls are significantly more economical compared to the externally stabilized wall considered in this study, and this economic benefit increases with increasing height of the walls.

## 1.    Introduction

Due to the development of materials and enhancement in technical understanding of geotechnical engineering, different types of soil retention systems have evolved over the last three to four decades. These systems may be classified into two groups, externally stabilizes walls and internally stabilized walls. The examples of first category are gravity walls, reinforced concrete cantilever (Figure 1a) and reinforced concrete counterfort

walls. These walls are essentially characterized by the concept that the lateral earth pressures due to self weight of the retained fill and accompanied surcharge loads are carried by the structural wall. This necessitates a large volume of concrete and steel to be used in such walls. The construction sequence of these walls involves casting of base and stem followed by backfilling with specified material. This requires considerable amount of time as concrete has to be adequately cured and sufficient time spacing has to be allowed for concrete of previous lift to gain strength before the next lift is cast.

The internally stabilized walls include metal strip walls (Figure 1b), geotextile reinforced walls (Figure 1c) and anchored earth walls (Figure 1d). These walls comprise of horizontally laid reinforcements which carry most or all of the lateral earth pressure via soil-reinforcement interaction or via passive resistance from the anchor block. If the reinforcements are spaced closely enough, the stiffness of the soil-reinforcement system may be so high that practically very insignificant lateral thrust will have to be carried by the wall facing elements. This reduces the volume of concrete and steel reinforcement in the wall significantly. An additional feature of the internally stabilized walls is their relatively fast speed of construction. This is firstly because of less volume of concrete and steel fabrication work, and secondly because the placing of wall panels, laying of reinforcements and compaction of reinforced fill are carried out simultaneously.
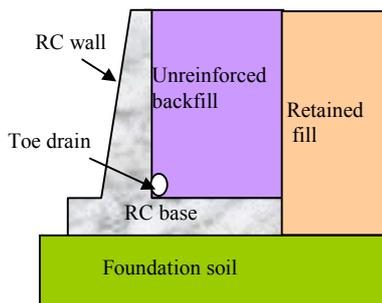


Fig. 1a.  RC cantilever wall
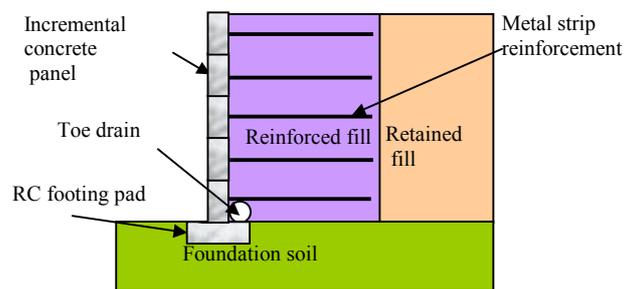


Fig. 1b.  Metal strip wall
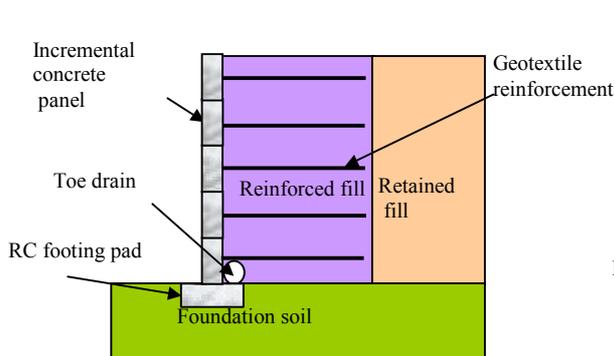


Fig. 1c. Geotextile reinforced wall
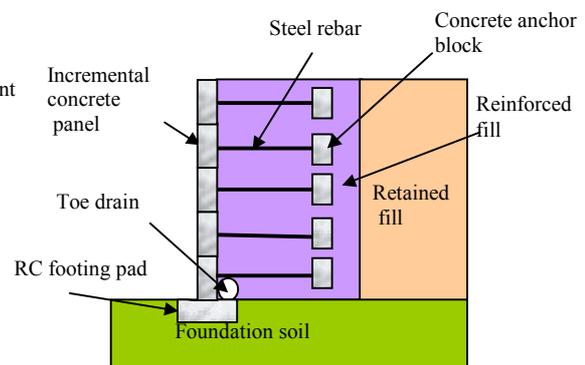


Fig. 1d. Anchored earth  wall

## 2.     Stability analyses and design method

Two stability analyses, namely external stability analysis and internal stability analysis, are considered in the design of retaining walls. The external stability analysis, which is applicable for both externally stabilized and internally stabilized walls, includes check against sliding at the base, overturning about the toe, bearing failure of the foundation soil and overall stability failure. The internal stability analysis, which is applicable for internally stabilized walls only, considers the check against rupture and pullout of the reinforcements within the reinforced material. The detail design procedures for each of the walls shown in Figure 1 are described below:

### 2.1     Design of RC cantilever wall (RCCW)

RCCW is perhaps the most widely used retaining wall. Therefore, the design procedure is very common and can be found in any text book (e.g., Bowles, 1988). Here the procedure outlined by Das (1990) is presented. Figure 2 shows the usual geometry of a RCCW and forces that normally act on it. The dimensions shown in the figure are only initial values for stability checks. If these dimensions do not satisfy the factor of safety against all the stability checks, the sections are revised. It should be noted that in the estimate of forces, no hydrostatic pressure is considered. This is ensured by considering both the backfill and retained fill as cohesionless soils and by providing sufficient weep holes or toe drains in the wall. The notations related to Figure 2 are described below:

$\gamma_1, \gamma_2, \gamma_3, \gamma_c,$ = Unit weight of backfill, retained fill, foundation soil and concrete
$\Phi_1, \Phi_2, \Phi_3$ = Angle of internal friction of backfill, retained fill and foundation soil
$c_3$ = Cohesion of foundation soil
D = Depth of embedment of foundation (depends on soil type and loading)
H = Height of the wall from EGL to the foundation level
$W_{s1}$ = Weight of surcharge on backfill
$W_{s2}$ = Weight of surcharge on retained fill
Ka = Rankine's coefficient of active earth pressure = $\dfrac{1 - \sin\phi_2}{1 + \sin\phi_2}$

$P_{a1}$ = Active force due to the retained fill = $0.5 Ka\, \gamma_2 H^2$
$P_{a2}$ = Active force due to the surcharge on retained fill, $W_{s2}$ = Ka $W_{s2}$H
$y_1$ = Vertical distance from base of the wall to the force $P_{a1}$ = H/3
$y_2$ = Vertical distance from base of the wall to the force $P_{a2}$ = H/2
$W_1$ = Total weight of concrete (stem and base)
$W_2$ = Wt. of backfill and surcharge $W_{s1}$ on backfill
$X_1$ = Horizontal distance from toe to the c.g of $W_1$
$X_2$ = Horizontal distance from toe to the c.g of $W_2$
B = Width of base of the retaining walle wall.

### 2.1.1     Check for overturning about toe

Overturning of the wall may occur about the toe, i.e. point A due to the lateral earth pressures shown in Figure 2. The Factor of Safety against such overturning can be expressed as:

$$FS_{(OT)} = \frac{\sum M_R}{\sum M_O} \; ; \; \geq 1.5$$

where, $FS_{(OT)}$ = Factor of Safety against overturning, $\sum M_R$ = Summation of resisting moment about point A, $\sum M_O$ = Summation of overturning moment about point A.

This yields, $$FS_{(OT)} = \frac{W_1 * X_1 + W_2 * X_2}{Pa1 * y_1 + Pa2 * y_2}$$
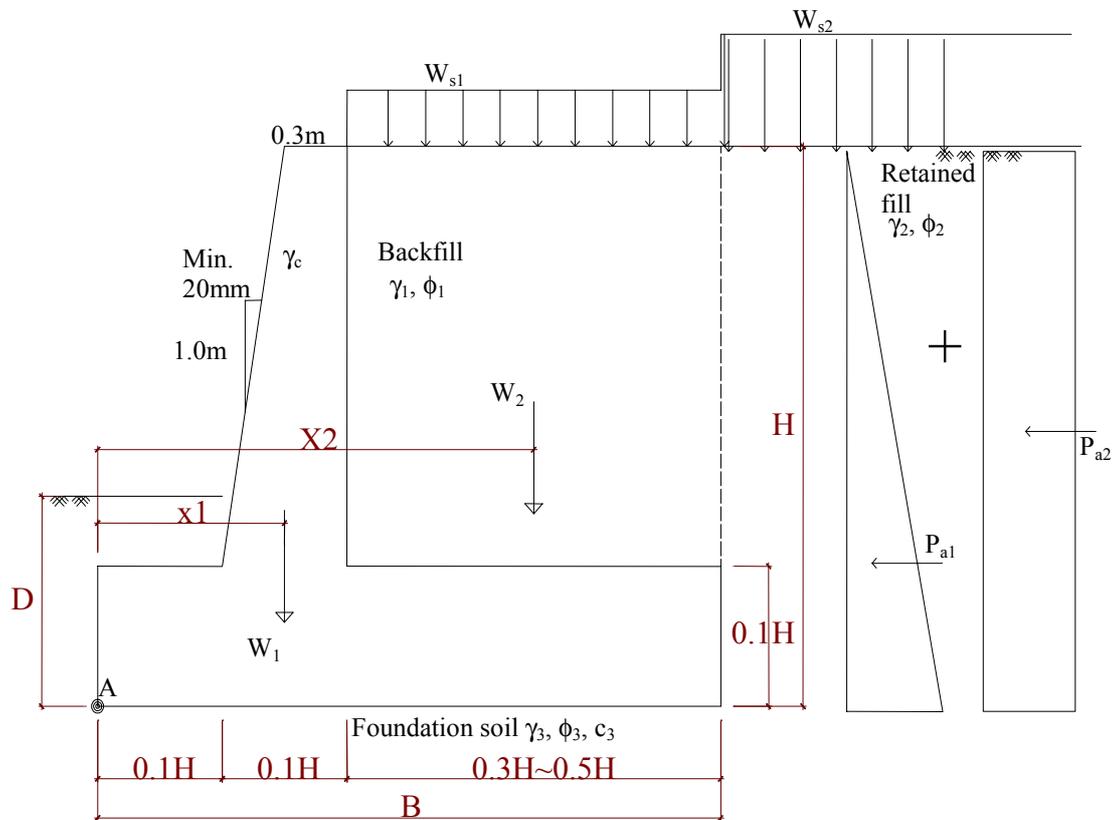
Fig. 2. Initial dimensions and forces for the design of a RCCW

### 2.1.2  Check for sliding at the base

The Factor of Safety against sliding at the base may be expressed as

$$FS_{(sliding)} = \Sigma F_R / \Sigma F_D \; ; \; \geq 1.5$$

where, $FS_{(sliding)}$ = Factor of Safety against sliding at the base; $\Sigma F_R$ = Summation of resisting forces against sliding; $\Sigma F_O$ = Summation of forces causing sliding at the base

This gives,

$$\Rightarrow \quad FS_{(sliding)} = \frac{(W_1 + W_2)\tan \Phi_3^{/} + B * c_3^{/}}{(Pa1 + Pa2)} \; ; \; \Phi_3^{/} = 2/3 \; \Phi_3, \; c_3^{/} = \; \tfrac{1}{2} \, c_3 \; to \; 2/3 \; c_3$$

### 2.1.3   Check for bearing capacity failure

The vertical pressure as transmitted to the soil by the base slab of the wall should be checked against bearing capacity of the soil. It should be appreciated that due to the lateral earth pressure, the bearing pressure will be maximum at the toe and minimum at the heel. The Factor of Safety against bearing capacity is then defined as:

$$FS_{(bearing)} = q_u / q_{max} \; ; \; \geq 3.0$$

where, $FS_{(bearing)}$ = Factor of Safety against bearing capacity failure; $q_u$ = Ultimate bearing capacity of the foundation soil; $q_{max}$ = Maximum pressure at the base of the wall

This provides,

$$= FS = \frac{c_3 * N_c + 0.5\, B\, \gamma_3\, N_\gamma + \gamma_3\, D\, N_q}{((W_1 + W_2)/B)*(1 + 6\, e/B)}$$

where, $N_c$, $N_q$ and $N_\gamma$ = Bearing capacity factors;

$e$ = Eccentricity of the resultant force at the base

$$= B/2 - \frac{\Sigma M_R - \Sigma M_O}{(W_1 + W_2)} \quad ; <= B/6, \text{ so that no tension occurs}$$

It may be appreciated that although some passive forces may generate from the soil in front of the toe, it is often safer to neglect this in the design as the soil in front of the toe may get eroded with time. However, in the situations where it may be estimated with certainty that the soil in front of the toe will never erode, the contribution from the passive force may be considered in calculating the factor of safety both against overturning and sliding.

## 2.2 Design of Metal Strip Wall (MSW) and Geotextile Wall (GTW)

The MSW and GTW use different types of reinforcing materials. The metal strip that is used in the MSW is inextensible in nature and its stress-strain behaviour is not sensitive to time and temperature. GTW, on the other hand, uses geotextiles as reinforcements the stress-strain behaviour of which is highly time and temperature dependent. The design procedures for MSW and GTW, detailed by Das (1990) and Koerner (1997), address such complex behaviour of geotextile reinforcements with appropriate safety factors. In addition to the external stability checks as described for RCCW in the earlier section, the design of MSWs and GTWs require check against internal stabilities such as check against rupture of reinforcements under operating loads and check against bond length of reinforcements into the passive zone so that they do not pullout under external loading (McGown et al., 1998). The minimum depth of embedment D for such walls must be 0.1H, BS8006 (1995) and AASHTO (1997).

### 2.2.1 Check for reinforcement rupture

The Factor of Safety against reinforcement rupture may be expressed by:

$$FS_{(R)} = T_D / T_i \; ; (2.5 \text{ to } 3.0 \text{ for MSW}, 1.3 \text{ to } 1.5 \text{ for GTW})$$

where, $FS_{(R)}$ = Factor of Safety against reinforcement rupture; $T_D$ = Allowable design strength of reinforcement (metal strip or geotextile)

$$= w\, t\, f_y \; ; \text{ for metal strip reinforcement}$$

$$= \frac{T_{ult}}{P_{Fid} * P_{Fcr} * P_{Fcd} * P_{Fbd}} \; ; \text{ for geotextile reinforcement}$$

w = Width of metal strip reinforcement
t= Thickness of metal strip reinforcement
fy = Yield strength of metal strip reinforcement
Tult = Wide width ultimate tensile strength of geotextile
$PF_{id}$ = Partial factor for installation damage (1.1 to 1.5)
$PF_{cr}$ = Partial factor for ensuring zero creep ( 2.0 to 2.5)
$PF_{cd}$ = Partial factor for chemical degradation (1.0 to 1.3)
$PF_{bd}$ = Partial factor for biological degradation (1.0 to 1.2)
Ti  = Maximum tensile force in a reinforcement at $h_i$ depth from EGL

$$= Ka_r ( \gamma_1 h_i + W_{s1} ) * S_v * S_h$$

$Ka_r$ = Rankine's coefficient of active earth pressure of reinforced soil

$$= \frac{1 - \sin\phi_1}{1 + \sin\phi_1}$$

$h_i$ = Depth of $i^{th}$ layer of reinforcement from EGL.
$S_v$ = Vertical spacing of reinforcements
    = $h_f$ (usually)
$S_h$ = Horizontal spacing of reinforcement
    = $w_f$ ( usually)
$h_f$ = height of facing
$w_f$ = width of facing
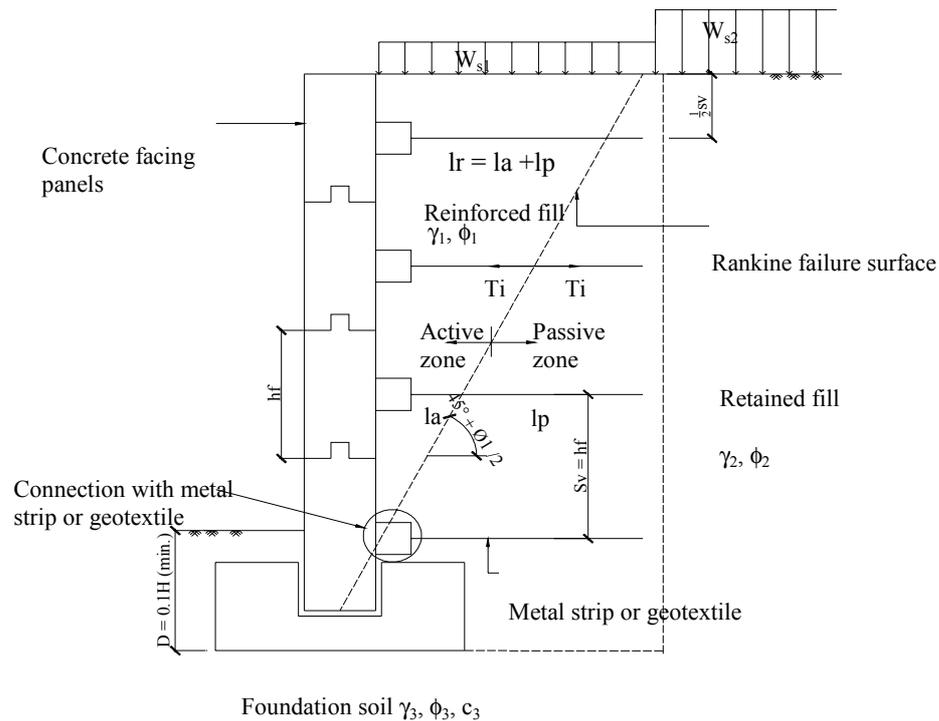


Fig. 3. Geometry and forces for the design of MSWs and GTWs

For the MSWs, a sacrificial thickness should be added to the thickness calculated above to allow for corrosion. The general guideline for the determination of sacrificial thickness, given by BS8006 (1995), is presented in Table 1.

Table 1
Sacrificial thickness to be allowed on surface exposed to corrosion

| Design service life (years) | Reinforcement material | Sacrificial thickness (mm) | |
|---|---|---|---|
| | | Land based structure | Fresh water structure |
| 5 | B | 0.25 | 0.25 |
| | G | 0 | 0 |
| | S | 0 | 0 |
| 10 | B | 0.35 | 0.4 |
| | G | 0 | 0 |
| | S | 0 | 0 |
| 50 | B | 1.15 | 1.55 |
| | G | 0. | 0.55 |
| | S | 0.05 | 0.07 |
| 60 | B | 1.35 | 1.68 |
| | G | 0.38 | 0.63 |
| | S | 0.05 | 0.09 |
| 70 | B | 0.45 | 0.7 |
| | G | 0.05 | 0.1 |
| 120 | B | 0.75 | 1.0 |
| | G | 0.1 | 0.2 |

B = black steel, (ungalvanized), G = galvanized steel, S = stainless steel

### 2.2.2    *Check against pullout failure*

The Factor of Safety against pullout of the reinforcements from the passive zone due to external loading may be estimated by the following:

$FS_{(p)} = T_r / Ti$ ; (2.5 to 3.0 for metal strip and 1.5 for geotextile)

where,  $FS_{(p)}$ = Factor of safety against pullout

        $Ti$ = Maximum tensile force in a reinforcement at $h_i$ depth from EGL (described earlier )

        $Tr$ = Pullout resisting force mobilized by the length $l_{pi}$ of reinforcement in the passive zone.

            $= 2*l_{pi} * w * \sigma_{vi} * \tan\phi_\mu$

    $l_{pi}$ = Length of reinforcement in the passive zone ; > 1.0m [AASHTO (1997), Koerner (1997)]

    $\sigma_{vi}$ = Total vertical pressure at $i^{th}$ layer of reinforcement

    $\phi_\mu$ = Soil – reinforcement friction angle

        $= 2/3 \phi_1$(usually).

It may be noted that once the length of embedment, $l_p$ is determined at any level of reinforcement, the total length of reinforcement, $l_r$ can be estimated as $l_r = l_a + l_p$ ; $l_a$ being the length of reinforcement in the active zone as shown in Fig. 3. Koerner (1997) and AASHTO (1997) specify that the minimum length of reinforcement should be 0.6H to 0.7H.
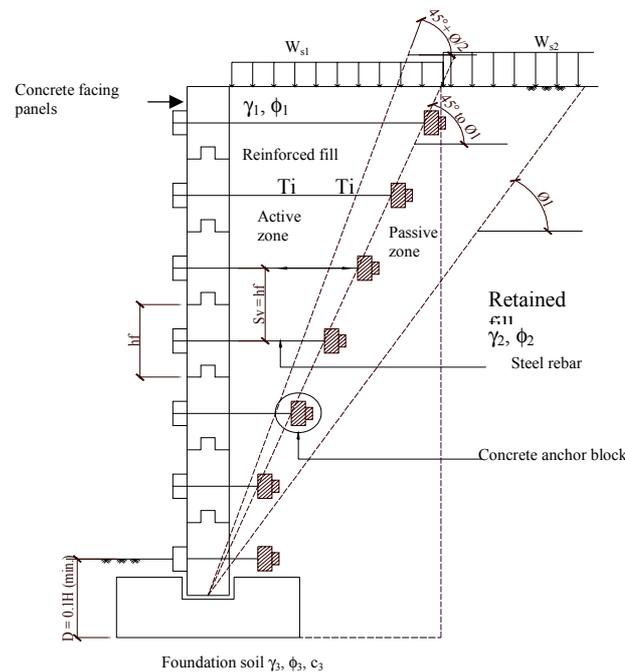
Fig. 4. Geometry and forces for the design of an AEW

## 2.3  Design of Anchored Earth Wall (AEW)

The external stability analyses and internal stability analyses methods for AEWs are similar to those of MSWs and GTWs. However, in AEWs the resistance against pullout is mobilized solely by the anchor blocks located at the end of reinforcements and seated in the deeper passive zone. The analyses method and guidelines suggested by Jones (1996), BS8006 (1995) and NAVFAC (1982) may be adopted for the design of AEWs. The minimum depth of embedment D for such walls must be 0.1H, BS8006 (1995).

### 2.3.1   Check for reinforcement rupture

The Factor of Safety against reinforcement rupture is given by:

$$FS_{(R)} = T_D / Ti \geq ; 2.5 \text{ to } 3.0$$

where,     $FS_{(R)}$ = Factor of Safety against reinforcement rupture
             $T_D$  = Allowable design strength of rebar
                 = $A_s * f_y$.
              $A_s$ = X - sectional area of circular rebar (sacrificial thickness given in

### 2.3.2 Check against pullout failure

The Factor of Safety against pullout of reinforcements via pullout of anchor blocks may be given by:

$$FS_{(p)} = T_r / Ti ; >2.5 \text{ to } 3.0$$

 where,     $FS_{(p)}$   = Factor of safety against pullout

$T_i$ = Maximum tensile force in a reinforcement at $h_i$ depth from EGL (described earlier)

$T_r$ = Pullout resisting force mobilized by passive pressure on anchor block.

$\quad = 4 * K_p * \sigma_v * w_b * h_b$

$K_p$ = Rankine's passive earth pressure coefficient.

$$= \frac{1 + \sin\phi_1}{1 - \sin\phi_1}$$

$\sigma_v = (\gamma_1 h_i + W_{s1})$

$w_b$ = Width of the anchor block.

$h_b$ = Height of the anchor block.

It may be noted that the resistance offered by the rebar is usually insignificant and hence ignored in estimating the Factor of Safety against pullout. The length of reinforcement at any level is determined by the location of the anchor block in the passive zone. For full passive resistance to mobilize, it is suggested that the anchor blocks be placed outside the surface making an angle $\phi_1$ with the horizontal, NAVFAC (1982). This may require a lot of space for the walls to be constructed. As implemented in a number of walls, the Author finds the placement of anchor blocks just outside the 45° surface with the horizontal to be adequate (Fig. 4). This means, for a 3.0m height of wall, at least 3.0m space should be available behind the wall as the length of the topmost layer of reinforcement will be 3.0m. However, even if it may not be necessary from calculation, a minimum of 1.0m length of reinforcement should be used at any level.

## 3.  Design examples and outcome designs

By way of example, RCCWS, MSWs, GTWs and AEWs of 2.1m, 3.0m, 4.2m, 5.1m and 6.0m height above the existing ground level (EGL) have been analysed and designed on the basis of design procedures presented above. All the soils, i.e. the unreinforced fill, reinforced fill, retained fill and foundation soil, are considered to have an angle of friction of $30^0$ and a cohesion value of 0.0 kN/m². It is assumed that the toe drains or weep holes are adequately provided in order to ensure that no pore pressure develops behind the walls even in the most critical hydraulic condition. The ultimate strength of the concrete and the yield strength of steel rebars/metal strips used in the design are 20 MPa and 415 Mpa, respectively.

The depth of foundation for RCCWs have been assumed to be at 1.0m from EGL and that for all the internally stabilized walls, i.e. MSWs, GTWs and AEWs have been assumed to be at 0.6m from EGL to comply with the minimum embedment depth guideline as suggested by BS8006 (1995) and AASHTO (1997). The incremental concrete facing panels of MSWs, GTWs and AEWs, having dimensions of 0.3m x 0.3m x 0.2m, have been designed to be supported by a 0.8m wide and 0.25m thick footing pad. The dimensions of the stem and base of RCCWs have been determined by trial, so that all the external stabilities are satisfied. In the design of MSWs, a minimum thickness of 3.0mm and width of 75.0mm metal strips have been used. For the design of GTWs, locally available grades of geotextiles have been taken into considerations. The minimum length of geotextile reinforcements and metal strip reinforcements used in the design of GTWs and MSWs, respectively, is taken as the 70% of the height of the wall as recommended by BS8006 (1995), Koerner (1997) and AASHTO (1997). The minimum length of embedment of the geotextiles and metal strip reinforcements within the passive zone is taken to be 1.0m as suggested by AASHTO (1997) and Koerner (1997). The epoxy coated plain rebars of minimum 12mm diameter, anchored to the

concrete blocks having minimum dimensions of 0.15m x 0.15m x 0.15m, have been considered to be practical for the design of AEWs. A uniformly distributed surcharge of 20 kN/m$^2$ is assumed on the reinforced fill, backfill and retained fill.

The critical loading pattern of surcharges on the backfill/reinforced fill and retained fill for determining the factors of safety in the external stability and internal stability analyses is given in Table 2.

Table 2
Critical loading pattern for stability analyses

| Stability Condition | Wall Type | Surcharge on backfill/ reinforced fill | Surcharge on retained fill |
|---|---|---|---|
| Sliding | RCCW, MSW,GTW,  AEW | - | √ |
| Overturning | RCCW, MSW,GTW,  AEW | - | √ |
| Bearing capacity | RCCW, MSW,GTW,  AEW | √ | √ |
| Reinforcement rupture | MSW,GTW,  AEW | √ | - |
| Reinforcement pullout | MSW,GTW, AEW | √ | - |

The outcome designs of the walls are detailed in this section. It should be noted that the dimensions provided in the tables in keeping with the notations used in the associated figures have been determined to satisfy both the geotechnical safety and structural safety of the walls. Figure 5 and Table 3 provide the details of the RCCWs. Figure 6 and Table 4 show the details of MSWs. Figure 7 and Table 5 summarise the dimensions of GTWs. Finally, the designed dimensions of AEWs are given in Figure 8 and Table 6.
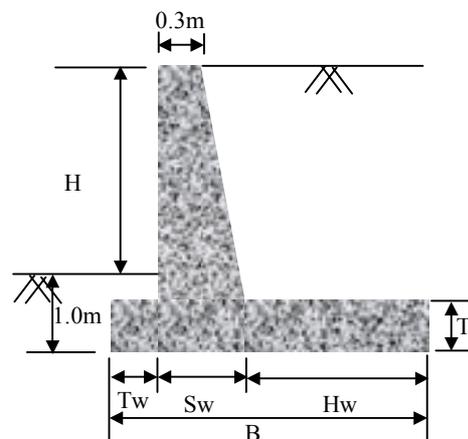


Fig. 5.  Dimension notations of RCCWs

Table 3
Dimensions of the designed RCCWs

| H (m) | T (m) | Sw (m) | Hw (m) | B (m) | ρ (%) |
|-------|-------|--------|--------|-------|-------|
| 2.1 | 0.4 | 0.3 | 2.9 | 3.8 | 1.2 |
| 3.0 | 0.6 | 0.3 | 3.9 | 4.8 | 1.6 |
| 4.2 | 0.75 | 0.3 | 4.9 | 5.8 | 1.6 |
| 5.1 | 0.75 | 0.38 | 5.9 | 6.88 | 2.0 |
| 6.0 | 0.75 | 0.45 | 6.9 | 7.95 | 2.0 |

Note: T = thickness of base, Tw= toe width, Sw = stem width, Hw = heel width, B = width of the base, H = height of wall above base, ρ = percentage of steel reinforcement

Table 4
Dimensions of the designed MSWs

| H (m) | L1 (m) | L2 (m) | N1 | N2 | Lr (m) |
|-------|--------|--------|----|----|--------|
| 2.1 | 3.0 | 2.5 | 5 | 4 | 3.0 |
| 3.0 | 3.5 | 2.5 | 6 | 6 | 3.5 |
| 4.2 | 4.5 | 3.0 | 8 | 8 | 4.5 |
| 5.1 | 5.0 | 3.5 | 10 | 9 | 5.0 |
| 6.0 | 5.5 | 3.5 | 11 | 11 | 5.5 |

Note: L1 = length of N1 nos. of strips, L2 = length of N2 nos. of strips, Lr= length of the reinforced section

Table 5
Dimensions of the designed GTWs

| H (m) | L1 (m) | L2 (m) | N1 | N2 | Lr (m) |
|-------|--------|--------|----|----|--------|
| 2.1 | 2.5 | 2.5 | 5 | 4 | 2.5 |
| 3.0 | 3.5 | 2.5 | 6 | 6 | 3.5 |
| 4.2 | 4.5 | 3.0 | 8 | 8 | 4.5 |
| 5.1 | 5.0 | 3.5 | 10 | 9 | 5.0 |
| 6.0 | 5.5 | 4.5 | 11 | 11 | 5.5 |
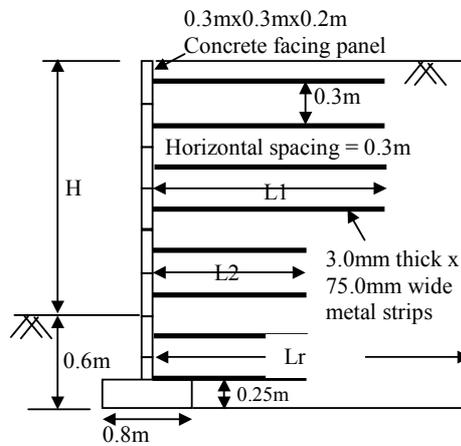
Fig. 6.  Dimension notations of MSWs
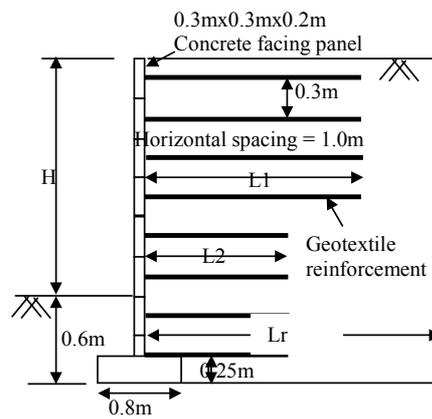


Fig. 7. Dimension notations of GTWs
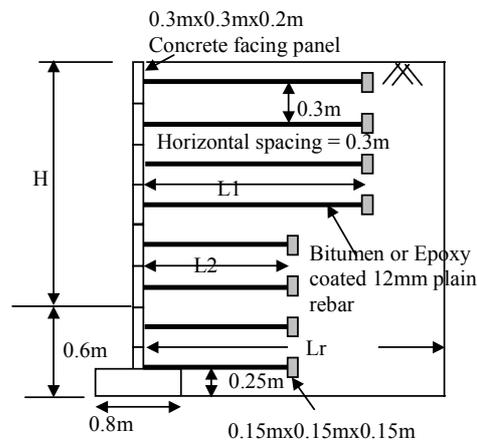


Fig. 8.  Dimension notations of AEWs

Table 6
Dimensions of the designed AEWs

| H (m) | L1 (m) | L2 (m) | N1 | N2 | Lr (m) |
|-------|--------|--------|-----|-----|--------|
| 2.1 | 3.0 | 1.5 | 5 | 4 | 3.0 |
| 3.0 | 4.0 | 2.0 | 6 | 6 | 4.0 |
| 4.2 | 5.0 | 2.5 | 8 | 8 | 5.0 |
| 5.1 | 6.0 | 3.0 | 10 | 9 | 6.0 |
| 6.0 | 7.0 | 3.5 | 11 | 11 | 7.0 |

## 4.    Cost comparison

The unit prices (in local currency) for various materials/items including labor and other cost considered at the time of this study are as follows:

-    Backfill material: 350 Tk/m$^3$
-    Concrete: 3600 Tk/m$^3$
-    Steel/metal strip reinforcement: 27,000 Tk/Ton
-    Geotextile: 110 Tk/m$^2$
-    Excavation: 50 Tk/m$^3$

On the basis of these unit prices and outcome designs tabulated in the preceding section, the total cost per running meter of the walls with respect to the heights considered in the present study have been determined. A summary of these costs is presented in Table 7 and Figure 9.

Table 7
Cost of walls of different type with respect to height

| Height (m) | Total cost (Taka) | | | |
|------------|------|------|------|------|
| | RCCW | MSW | GTW | AEW |
| 2.1 | 17482 | 9952 | 7759 | 8059 |
| 3.0 | 32557 | 14128 | 12461 | 12537 |
| 4.2 | 48438 | 22356 | 19577 | 19301 |
| 5.1 | 67284 | 29134 | 25359 | 26282 |
| 6.0 | 90848 | 35646 | 32271 | 34101 |

It may be worth noting from Table 7 and Fig. 9 that the internally stabilized walls, i.e. MSWs, GTWs and AEWs, are significantly more economical compared to the externally stabilized walls, i.e. RCCWs. The cost of the RCCWs compared to the MSWs, GTWs and AEWs increases more rapidly from 2.1m to 6.0m height for the loading and geometric conditions considered here. The difference of cost between the RCCWs and the internally stabilized walls, therefore, increases with the height of the wall.

The per cent savings of the internally stabilized wall systems, i.e. MSWs, GTWs and AEWs with respect to the externally stabilized walls, i.e. RCCWs considered in this study are summarized in Table 8 and Figure 10. These show that the economic benefit that might be accrued by implementing internally stabilized walls may range from 43 to 64 per cent. Thus, it appears that for the walls of large height, internally stabilized systems are more preferable solution over the conventional ones, provided the site situation permits such implementation.
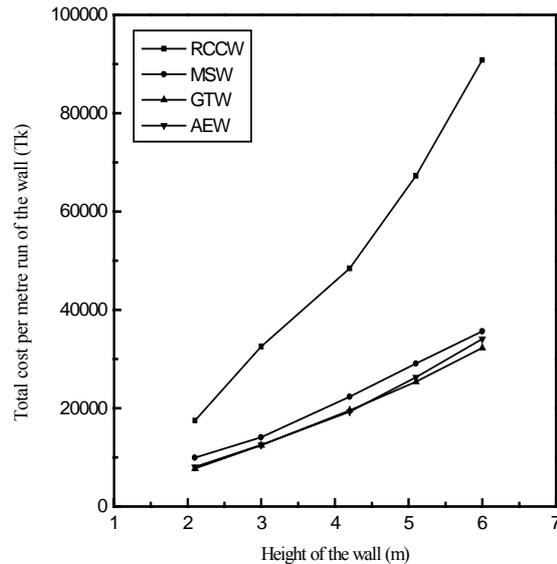


Fig. 9. Total cost per running meter vs height of wall
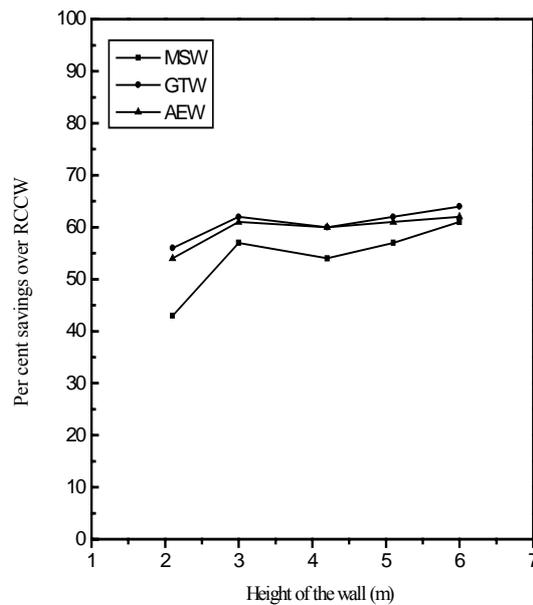


Fig. 10. Per cent savings of the internally stabilised walls
over externally stabilised wall (RCCW)

Table 8
Per cent savings of the internally stabilised walls
compared to the externally stabilised wall

| Height (m) | Per cent savings (%) | | |
|---|---|---|---|
| | MSW | GTW | AEW |
| 2.1 | 43 | 56 | 54 |
| 3.0 | 57 | 62 | 61 |
| 4.2 | 54 | 60 | 60 |
| 5.1 | 57 | 62 | 61 |
| 6.0 | 61 | 64 | 62 |

Besides the total cost of the walls per meter run and the percent savings of the MSWs, GTWs and AEWs over the RCCWs, it might be worthwhile appreciating the per meter run cost of some common salient components of all these walls. The cost of excavation, backfill, concrete, steel rebars and geotextiles per meter run of the walls have been presented in Figures 11 to 15. These figures indicate that the cost of excavation, backfill and geotextile reinforcements do not contribute significantly to the difference in total cost per meter run of the RCCWs compared to the MSWs, GTWs and AEWs. Rather, it is evident from the Figures 13 and 14 that the huge cost of concrete and the cost of steel rebars make the significant difference between the cost total cost of the RCCWs and its counterparts.
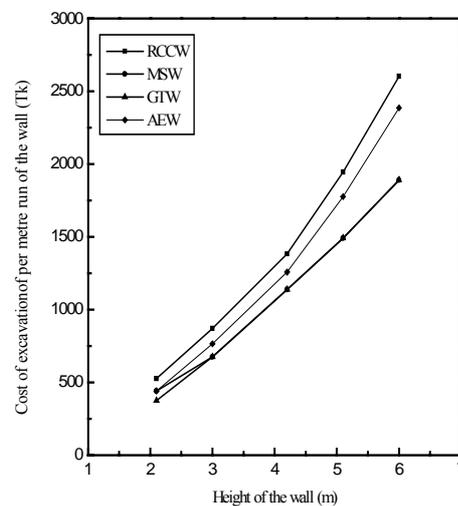


Fig. 11. Cost of excavation per meter run of the RCCWs, MSWs, GTWs and AEWs

## 5.    Conclusions

Procedures for the design of design of different types of externally and internally stabilized walls have been presented in detail. By way of example, RCCWs, MSWs, GTWs and AEWs of 2.1m, 3.0m, 4.2m, 5.1m and 6.0m height above the existing ground level (EGL) have been analysed and designed. The walls are then detailed out in order to estimate the cost per running meter of the walls. It is found that the internally stabilized

walls are significantly more economical compared to the externally stabilized wall for the given geometric and loading conditions considered in this study.  The major contribution in the cost difference is attributed to the huge amount of concrete and steel rebars usually required in the RCCWs compared to its counterparts. The economic benefit accrued from the internally stabilized systems increases with the height of the walls. The per cent savings of the internally stabilized walls may range from 43 to 64. Therefore, implementation of the internally stabilized walls in Bangladesh should not be restricted in anticipation that such wall systems involve extravagant cost, especially for a situation that permits implementation of such soil retention system.
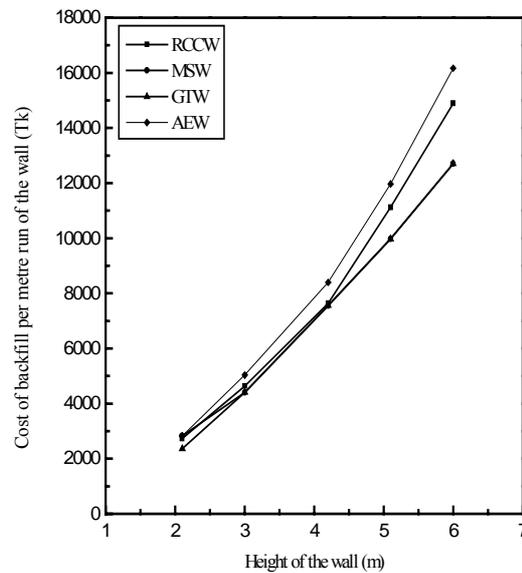


Fig. 12. Cost of backfill per meter run of the RCCWs, MSWs, GTWs and AEWs
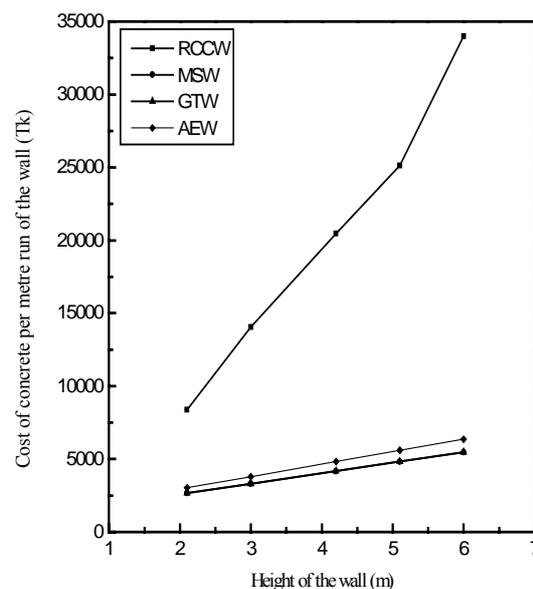


Fig. 13.  Cost of concrete  per meter run of the RCCWs, MSWs, GTWs and AEWs
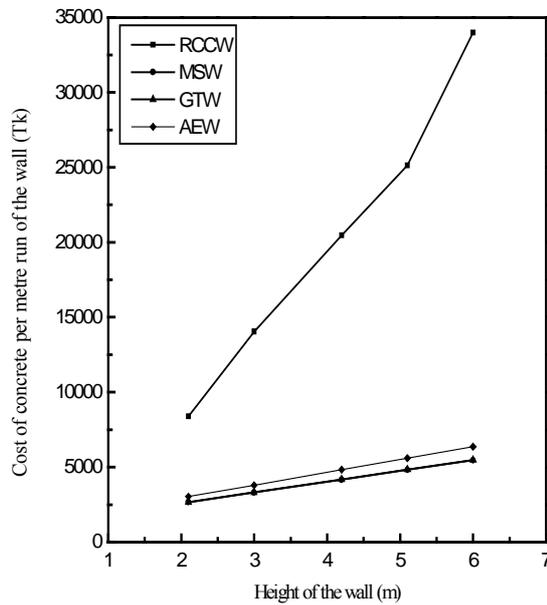
Fig. 14.  Cost of steel rebar  per meter run of the RCCWs, MSWs, GTWs and AEWs
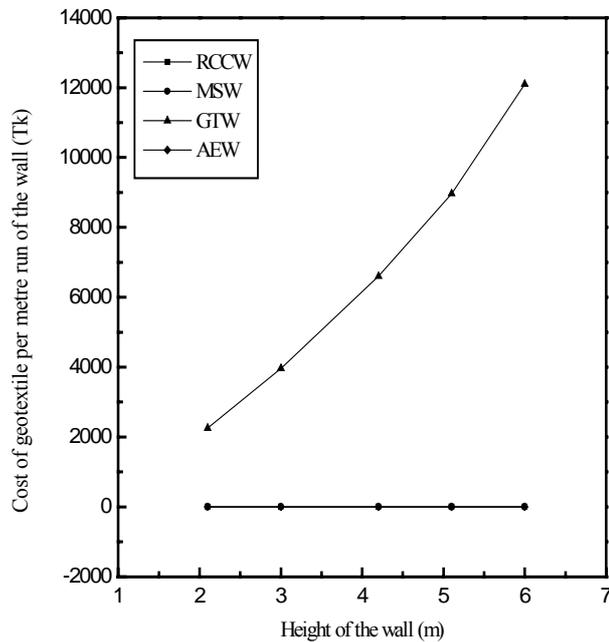


Fig. 15. Cost of geotextile per metre run of the RCCWs, MSWs, GTWs and AEWs

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