DRAFT

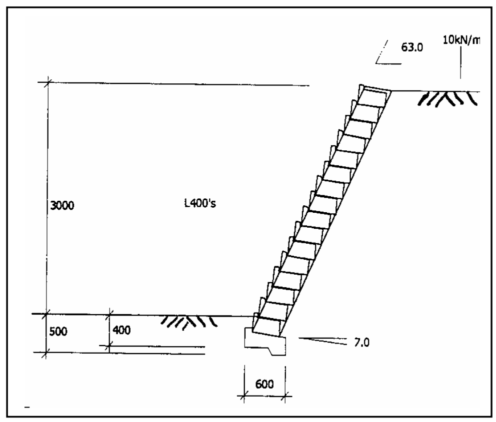
CODE OF PRACTICE FOR DRY STACK CONCRETE BLOCK RETAINING WALLS

1. INTRODUCTION

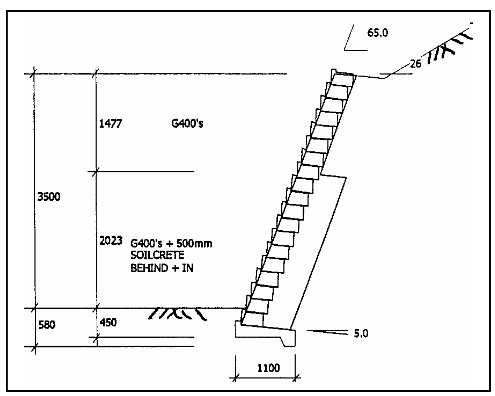
In recent years dry stack concrete block retaining systems have come into common usage, and in many instances are being used in preference to conventional reinforced concrete walls. Note that

herein the dry stack concrete block retaining walls will be referred to as CRB (concrete retaining block) walls. Such walls are also referred to as segmental retaining walls. The CRB walls can be divided into three groups of gravity retaining walls.

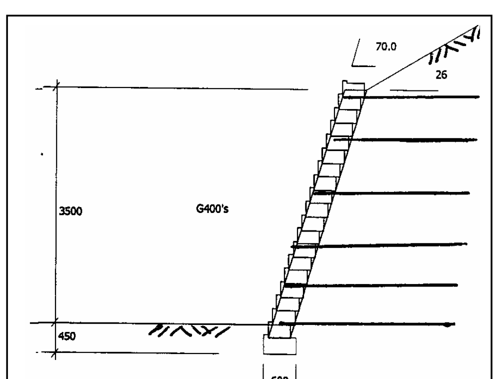
Conventional CRB retaining walls resist destabilising forces, due to retained soils, solely through the self-weight of the CRB units combined with the weight of any soil within the units and the batter of the CRB units.



Soilcrete enhanced CRB retaining walls are walls in which the effective depth and weight of the CRB wall is increased by cement/lime stabilising a prescribed depth of soil behind the CRB units to form soilcrete.



Soil-Reinforced CRB retaining walls are composite retaining systems consisting of CRB units in combination with a mass of soil stabilised by horizontal layers of geosynthetic reinforcement materials.



Essentially CRB walls comprise precast concrete blocks, which are stacked row upon row with a prescribed backward offset to form a retaining wall with a specified backward slope. Typically the slopes of the conventional CRB walls range between 55 and 70 degrees to the horizontal. During construction, each row of blocks is filled with soil in conjunction with the placement and compaction of backfill behind the wall.

It is current practice to design these walls as composite gravity retaining walls, which are reliant on both the weight of the blocks and the weight of the soil infill. In designing a wall, the destabilising force on the wall due to active earth pressures and the weight of the wall counteracting this force, are calculated based on the assumption that the stacked precast concrete blocks and soil infill, act together as a single body. Having thus computed these two forces, it is usual to ensure that geometry of the wall is such that the factor of safety both against overturning and against block on block sliding does not exceed 1.5. Often no check is carried out to ensure that the line of action of the resultant external force passes through the middle third of the base of the wall (i.e. the bottom row of blocks), which is common practice in the design of gravity walls. This middle third condition is incorporated in gravity wall designs in order to ensure that no tension exists within the structure. While this requirement is perhaps not essential in dry stack systems, what is considered to be of critical concern in this regard, is that in many instances, the line of action of the resultant force passes behind the back of the bottom row of blocks. Under such conditions there can be no certainty as to the forces that exist between the rows of blocks. Such conditions also bring into question, the assumption that the wall acts as a single body, regardless of its height and its backward slope.

It is considered that in cases where the computed line of action of the resultant force passes behind the bottom row of blocks, the effective height and corresponding effective weight of the wall, should be reduced by an amount sufficient to ensure that the line of action of the resultant passes within the base of the bottom row of blocks. In this way it is assumed that the uppermost portion of the wall is in essence supported on the slope behind the wall and that it has no influence on the sliding resistance at the base of the wall and the resistance to overturning about the toe of the wall. In the following section an approach to the design of dry stack retaining walls is presented. While there are many different types and sizes of precast blocks available for use in the construction of dry stack retaining walls, the basic design approach is applicable to all CRB wall types.

2.1 The Basic Design Procedure

There is more than one approach to the design of CRB retaining walls, and one can not be entirely prescriptive as regards the design procedure. Nonetheless in presenting the design procedure below, the intention is to provide certain design guidelines that will apply regardless of any variation in the specifics of a design procedure.

Prior to discussing each stage of the design in detail, a brief description of each stage of the design procedure for dry stack retaining walls, is given below:

(1) **Decide upon soil parameters** for both the material to be retained by the wall and the material in front of the wall, the wall friction, the foundation wall friction, and the position of any ground water. It will also be necessary to know the weight per square metre of wall face of the CRB blocks filled with soil, the block on block friction, and in cases where the blocks possess nibs, the shear strength of the nibs per metre run of wall.

(2) **Select a trial wall slope** and in cases where the ground slopes away from the top of the wall, determine the height of the wall.

(3) **Calculate the destabilising forces** acting on the rear of the wall applied by the retained soil and any external loads such as a line load or a uniformly distributed load (UDL) behind the wall.

(4) **Calculate the resultant of the destabilising forces and the self weight of the wall** for the proposed block type and size(s).

(5) **Check that the line of action of the resultant force passes behind the front third of the bottom row of blocks, and that it passes within the blocks.** If the line of the resultant passes within or ahead of the front third, then the rear of the wall would be in tension. In the design of gravity walls, it is generally considered that such a condition should not be allowed to arise. If the line of action of the resultant force passes behind the bottom row of blocks then reduce the "effective" height and corresponding "effective" weight of the wall until it passes within the blocks. The "effective" height and weight of the wall being that portion of the wall which is considered to be contributing to the resistance of the wall to both overturning about the toe of the wall and to sliding at the base of the wall.

.

(6) **Check the factor of safety of the wall against overturning** using the effective weight of the wall. Generally acceptable if above 1.5.

(7) **Check the factor of safety of the wall against block on block sliding** between the lowest two rows of blocks. Generally acceptable if above 1.5. The blocks in the bottom row are

invariably either set in the wet foundation concrete or are restrained from sliding off the found by means of a concrete nib, and thus the most critical level for block on block sliding is invariably between the lowest two rows of blocks.

(8) Through an iterative process, **determine the minimum founding depth** that will provide the required factor of safety (usually 1.5) against sliding of the wall at the level of the base of the foundation. For each trial founding depth, this involves the computation of the passive earth force that if mobilised, acts together with the frictional resistance developed along the underside of the foundation to provide the resistance to sliding.

(9) **Calculate the bearing pressures beneath the front and back of the foundation.** If these are unacceptable, then increase the width of the foundation.

(10) In the case of a design using a walling system which has more than one compatible block size, if the wall comfortably meets the design criteria when it is comprised solely of the larger blocks, the **block mix should be optimised** through the inclusion of as many of the smaller blocks as the design criteria limits will allow.

(11) If the dry stack retaining wall at its selected wall slope, does not satisfy all the above design criteria then there are two basic options open to the designer, namely

(i) flatten the wall slope and repeat steps 1 to 10, or

(ii) increase the effective width and effective weight of the wall by either stabilising a specified width of suitable backfill behind the wall or by incorporating geofabric reinforcement between rows of blocks at pre-determined spacings, which tie the block wall to a body of backfill behind the wall. Then repeat steps 1 to 10.

(12) In circumstances where there could be long-term slope instability involving a large mass of soil/rock surrounding the wall, a slope stability analysis should be carried out in order to **assess the possibility of a deep seated slip failure passing beneath the wall**. Such slope instabilities typically occur in soft clayey soils or bedrock with planar weaknesses. Overall slope stability analyses are beyond the scope of this Code of Practice, suffice to say that when designing retaining walls one should always be on the lookout for adverse conditions that may give rise to slope instability. In cases where one is in essence cladding a rock face to protect it against weathering, it is generally advisable to consult a geotechnical specialist as to the stability of the rock slope.

2.2 The Selection of Soil Parameters

The conventional approach in geotechnical engineering is to describe the shear strength of a soil using Mohr-Coulomb failure criteria. The shear strength τ is expressed as,

$$\tau = c^{/} + \sigma_n^{/} \tan \phi^{/}$$

where $\sigma_n^{/}$ is the effective normal stress acting on the internal soil failure plane (i.e. the normal stress less the pore water pressure u acting at the same location, and $c^{/}$ and $\phi^{/}$ are the cohesion and internal angle of friction of the soil under effective stress conditions.

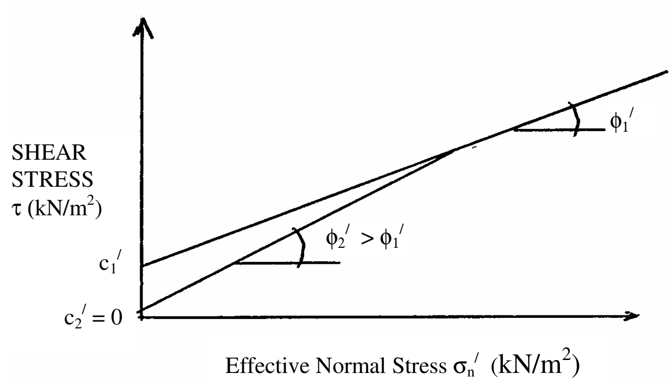
Values for $c^{/}$ and $\phi^{/}$ can be determined from direct shear tests (shear box tests) in the case of mainly granular soils or triaxial compression tests mainly clayey soils. However in designing dry stack retaining walls, it is not usual to carry out extensive field and laboratory testing in order to establish appropriate soil shear strength and bulk unit weight parameters, and it is common for designs to be based on typical soil properties. For reference purposes, some typical soil properties are given in Table 1, below. It should be noted that it is assumed that the soils are purely frictional, i.e. they are cohesionless and their shear strength is solely dependent upon their internal angle of friction, ϕ. Generally earth pressures behind retaining walls are computed based on the assumption that the retained material is purely frictional. This is hardly surprising, since, although there are standard formulae and Tables of coefficients available to compute earth pressures for purely frictional soils, apart from for the case of a vertical wall with horizontal retained material, there are no such formulae or Tables of coefficients available that enable designers to readily calculate the effect on earth pressures of any cohesive component of the retained materials shear strength. Instead designers are referred to relatively cumbersome graphical techniques to compute forces on retaining walls applied by soils which possess both effective cohesion (c') and effective friction (ϕ').

TABLE 1 : TYPICAL VALUES OF SOIL PROPERTIES

TYPE OF MATERIAL	ϕ - degrees	γ - kN/m^3
Loose, sandy silt or clayey sand	25	18
Very loose, uniformly graded sand /slightly silty sand	28	17
Loose uniform sand, round grains or dense sandy silt	30	18
Dense or partially cemented uniform sand, or Loose, well graded sand	33	19
Dense well graded sand - angular grains	35	20 - 21
Loose sandy gravels	35	19
Dense sandy gravels	35 - 40	20 - 22

One can use the graphical wedge analysis (see Section 2.4) to analyse (c', ϕ') soil and there is at least one computer programme that has incorporated an analytical adaptation of the graphical wedge analysis technique in a dry stack retaining wall computer programme. However it should be appreciated that attributing an effective cohesion even as low as c' = 5 kN/m^2 to the retained soil, dramatically reduces the earth pressure behind a walls, particularly behind those which slope back at 70° or less. Consequently the assignment of effective cohesion to the soil behind or in front of a wall should be done with extreme circumspection.

In general it is inadvisable to use soil cohesion in the determination of the destabilising forces on the retaining wall, because in many cases the wetting and drying of the material near the face of the wall leads to loss of cohesion. Thus in cases where there is likely to be cohesion during the lifetime of the structure we recommend that the internal angle of friction of the material be increased to ensure a steeper critical failure envelope than would have been the case had the material not had cohesion. Refer to the Figure below.



MOHR-COULOMB DIAGRAM

Wall Friction at rear of retaining wall

In most cases each row of blocks up a dry stack retaining wall has a prescribed backwards offset, and thus when the active wedge behind the wall is mobilised, its shear surface at the rear of the wall will practically be an entirely soil on soil contact. The wall friction (δ) will therefore be equal to or very nearly equal to the φ' of the retained material. A δ of between 0.8 φ' and 0.9φ' is recommended.

Foundation Wall Friction

Foundation wall frictions for rigid retaining walls such as reinforced concrete cantilever walls, are generally limited to less than or equal to 1/3 φ', the primary reason being that the development of full passive earth resistance requires a relatively large wall displacement. However dry stack retaining walls are relatively flexible and less sensitive to differential displacements. Therefore a higher wall friction may be used, and provided that a displacement sensitive structure is not situated in close proximity to the rear of the wall, a foundation wall friction as high as 2/3 ϕ' may be used.

Base Friction

Provided a concrete foundation is provided for the wall, and the foundation is cast in situ the base friction (μ) may be taken as being equal to the ϕ' of the underlying soil. If no foundation is cast, then μ should be reduced to between 1/2 and 2/3 ϕ', because in such cases the base friction is developed between soil and precast concrete. Thus it is recommended that all dry stack retaining walls be supported on cast in situ foundations.

2.3 Selection of a trial wall slope

The final design's wall slope is dependent upon many factors, inter alia, the height to be retained, the nature of the soil to be retained, the slope of the ground behind the wall, whether or not external loads will be applied behind the wall, whether or not there are space constraints within which the wall must be constructed, and naturally type and size(s) of the blocks under consideration. Thus one can not be sure of selecting and initial trial wall slope that will be close to the final design's wall slope. Nonetheless, as a general rule, provided there are no space constraints, one would start off with a wall slope of between 65 and 70 degrees, and then if necessary flatten the wall slope in order to achieve a design, which meets the desired design criteria. As can be seen from Figures 5 & 6, which show plots of the maximum wall height versus wall slope for three sizes of a type of CRB block, the flattening of the wall slope sharply increases the maximum wall height until a wall slope of 60 degrees in the case of level backfill, and until approximately 55 degrees in the case of the ground slope of 26 degrees, thereafter no significant improvement in maximum retaining height is gained by further flattening the wall slope. This is based on the hypothesis that the effective wall weight should be reduced when the line resultant of action of the resultant passes behind the rear of the bottom row of blocks. This hypothesis is discussed in detail in Section 2.6. The limit beyond which no significant improvement in maximum retaining height can be gained by flattening the wall, appears to be independent of block size, but it is dependent on the ground slope behind the wall, and it will also be dependent upon the retained soil's shear strength and its weight.

In some cases there are space constraints and the wall must be built steeper than a certain prescribed angle, for example 85 degrees. Clearly in such cases there is no longer the option of flattening the wall slope in order to achieve the desired retained height. In these cases, it is unlikely that the blocks will be able to retain the desired height without either stabilising backfill behind the wall or introducing geofabric reinforcement into the backfill.

In circumstances in which the ground slopes steeply down to the top of the wall, and the toe position of the wall is fixed, it is often useful to be able to calculate the height of a sloping wall relative to a vertical wall. An expression that enables one to do this is as follows:

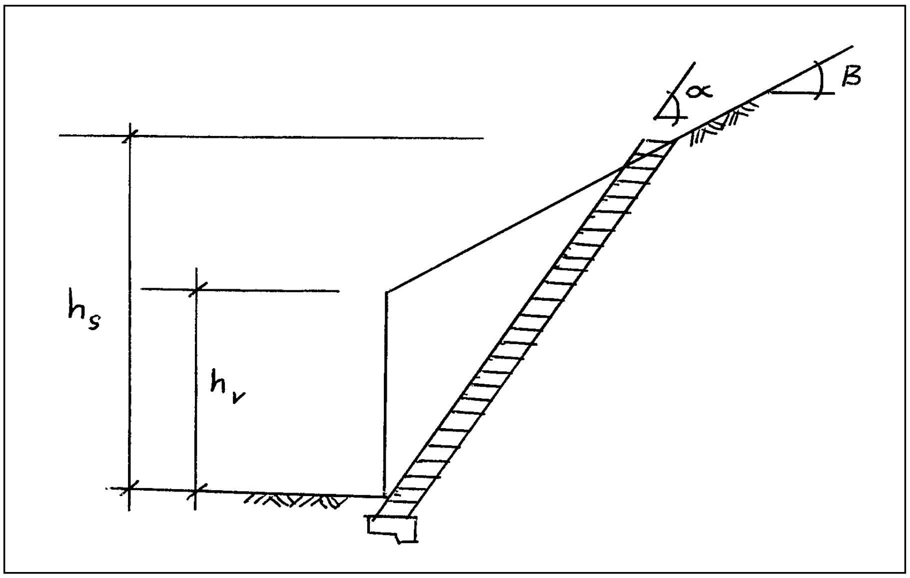
$$h_S = \frac{h_V \sin\alpha \cos\beta}{\sin(\alpha - \beta)}$$

where h_S = height of wall with face sloping at α

h_V = height of vertical wall with the same toe position

α = angle of the front slope of the wall to the horizontal

β = angle of inclination of the retained soil



It should be appreciated that in such cases, beyond a certain wall slope, any apparent improvement gained in the block mix of a wall by flattening the wall, is negated by the increase in the wall height as a result of flattening the wall. Note the type of improvement in the block mix referred to here, is one in which the percentage of relatively cheap smaller blocks in the wall is increased while the percentage of larger blocks is reduced. If flattening the wall to increase the percentage of smaller blocks results in a higher wall, then there would only be an overall cost saving on the wall if the cost of having to use more smaller blocks, is less than the saving made on reduced number of larger blocks.

2.4 Calculation of the Destabilising Forces

As with other types of retaining walls, the forces applied to dry stack retaining walls by the retained soil are invariably calculated based on the assumption that sufficient slight forward movement of the wall occurs to allow the development of active earth pressures. Active earth pressures are computed based on the assumption that a wedge of soil slides forward against the wall on the most critically inclined shear plane. Coulomb in 1776 initially published a solution containing formulae to calculate the active force applied by a cohesive frictional soil for the case of a vertical wall with horizontal backfill (total stresses only), and with no allowance for wall friction. In 1808 Mayniel extended the solution to include wall friction, but only for frictional non-cohesive soils. In 1906 the general solution for frictional cohesionless soils, was further extended by Muller-Breslau to allow for sloping backfill, a sloping back face to the wall and friction on the back face of the wall. Their solution as presented in Earth Pressure and Earth-Retaining Structures (C R I Clayton and J Milititsky, 1986), is given in Figure 1. The Coulomb and Mayniel solutions are also given in this book.

Comment: Page: 8

$$Q_a = \tfrac{1}{2}\gamma H^2 . \frac{f_1}{\sin\alpha . \cos\delta}$$

where

$$f_1 = \frac{\sin^2(\alpha+\phi).\cos\delta}{\sin\alpha.\sin(\alpha-\delta)\left[1+\sqrt{\left[\dfrac{\sin(\phi+\delta).\sin(\phi-\beta)}{\sin(\alpha-\delta).\sin(\alpha+\beta)}\right]}\right]^2}.$$

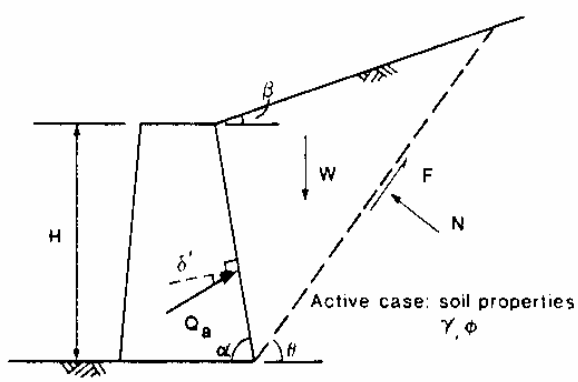


Figure 1 Muller-Breslau solution for the active force applied by a frictional cohesionless soil

In most cases designs are carried out on the basis of the Muller-Breslau solution in which it is assumed that the retained soil is purely frictional. In order to allow for soil cohesion in cases other than the straight forward case of a vertical wall with horizontal backfill, designers have had to resort to graphical techniques such as the wedge analysis illustrated in Figure 2 below. Figures 1 and 2 are taken directly from C R I Clayton and J Milititsky (1986). As mentioned above, this graphical wedge analysis technique enables the designer to enter parameters for the effective cohesion c', and effective friction ϕ' of the soil.

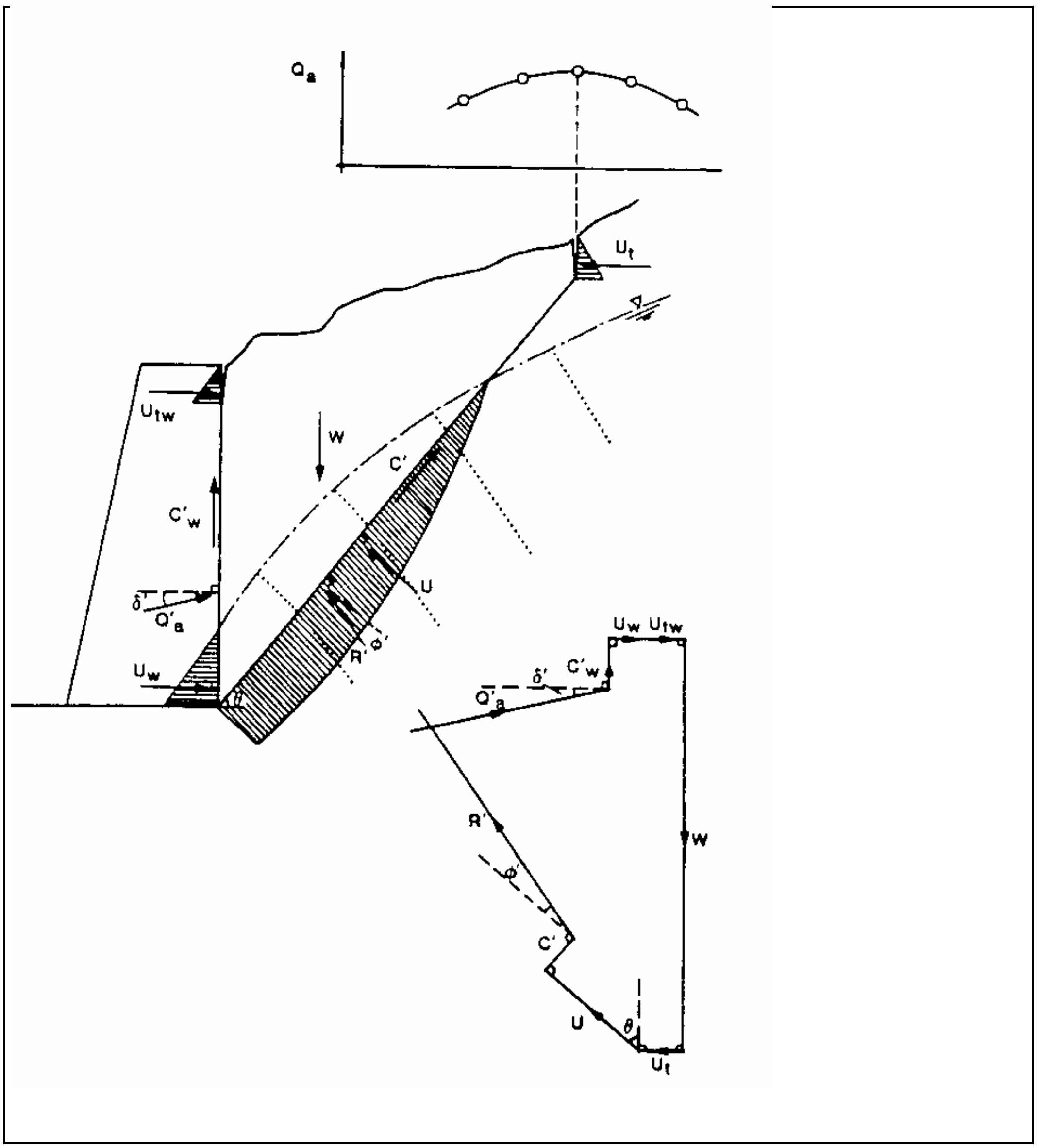


Figure 2 Wedge analysis for the active force case

Note that with the graphical wedge analysis, the inclination of the trial slip/shear surface at the base of the wedge, has to be varied until the critical surface is established, i.e. the surface that gives the maximum active force.

External Loads

In numerous books such as in C R I Clayton and J Milititsky (1986), there are standard elastic solutions for the horizontal stress increase due to point loads, line loads and loaded areas, for varying distances from and orientations to the back of retaining walls. These will not be reproduced here. One empirical method that can be used for assessing the effect of the line load is that of Terzaghi and Peck (1948), and it is shown in Figure 3.

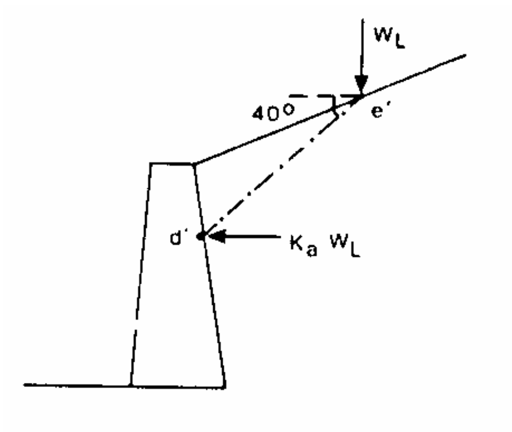


Figure 3 Method of assessing the effect of a line load (Terzaghi and Peck, 1948)

When dealing with a purely frictional backfill, one of the methods for assessing the effect of a UDL behind the wall is as that of treating the load as equivalent to an extra height of soil. The expression used for the equivalent height is as given by G.N. Smith (1982):

$$h_e = \frac{w_s}{\gamma}\,\frac{\sin\alpha}{\sin(\alpha+\beta)}$$

where γ = unit weight of soil
w_s = intensity of uniform load/unit area
α = angle of the back of the wall to the horizontal
β = angle of inclination of the retained soil

With the graphical wedge analysis the weight of the surcharge on each wedge is merely added to the weight of each wedge.

2.5 Calculation of the Resultant force and check on its Line of Action

The forces to be considered in analysing the stability of a dry stack retaining wall, are shown on Figure 4. The resultant force between the destabilising forces and the self weight of the wall can then readily be calculated using a number of different approaches, one of which is as follows:

$$Q_{av.} = Q_a . \sin(\delta + \alpha - 90°)$$
$$Q_{as} = Q_a . \cos(\delta + \alpha - 90°)$$
$$Q_{uv} = Q_u . \sin(\delta + \alpha - 90°)$$
$$Q_{uh} = Q_u . \cos(\delta + \alpha - 90°)$$
$$Q_{lv} = Q_l . \sin(\delta + \alpha - 90°)$$
$$Q_{lh} = Q_l . \cos(\delta + \alpha - 90°)$$
$$\psi = \tan^{-1}((Q_{av} + Q_{uv} + Q_{lv} + W_e)/(Q_{ah} + Q_{uh} + Q_{lh}));$$
$$R = (Q_{ah} + Q_{uh} + Q_{lh})/ \cos(\psi);$$

where Q_a = active force due to earth pressures
Q_u = force on wall due to the Uniformly Distributed Load
Q_l = force on wall due to the Line Load
W_e = Effective Weight of the wall
ψ = Inclination of the resultant force to the horizontal
R = Resultant force
δ = wall friction
α = inclination of the back of the wall to the horizontal
the additional subscripts $_v$ and $_h$ signify respectively the vertical and horizontal components of the forces acting on the wall

Note that all the above forces are calculated per metre run of the wall.

Initially the effective weight of the wall is taken as being equal to the total weight of the wall, i.e. the total weight of the blocks and soil infill. Having computed the resultant force, the distance of its line of action from the toe of the bottom row of blocks is computed by taking moments of the wall weight and destabilising forces about this point, and then dividing the moment by the resultant force. If the line of action of the resultant passes behind the back of the bottom row of blocks, then it is recommended that the effective height and corresponding effective weight of the wall be decrimentally reduced until the line of action passes within blocks. In adopting this approach one is assuming that the blocks at the top if the wall are effectively lying on and are supported by the retained slope, and that they are not contributing to the sliding resistance at the base of the wall. This approach certainly makes sense when you consider the extreme case of blocks stacked to form a "wall" to a height in excess of 10 metres at a slope of say 35 degrees, because under such circumstances the blocks over the entire height of the wall would be primarily supported by the backfill and it would be ludicrous to utilise the full weight of the wall in calculating the block on block sliding resistance at the base of the wall. The design procedure of reducing the effective height and weight of the wall in order to keep line of action within the bottom blocks, has the effect of limiting the maximum retaining wall height that can be achieved by flattening the wall slope. This is clearly evident in Figures 5 and 6, which are plots of the maximum CRB block retaining wall height versus wall slope for three sizes of particular type of CRB block. Curves are shown both for the approach of keeping the line of action of the resultant force within the bottom blocks, and for the approach of assuming that the total weight of the wall is always effective against sliding and overturning, even when the resultant's line of action passes behind the bottom row of blocks. As can be seen from Figures 5 and 6, with the former approach, down to an angle of 60 degrees with level backfill and down to 55 degrees with the ground behind sloping at 26 degrees, flattening the wall slope increases the maximum wall height, thereafter there is practically no increase in maximum wall height with decreasing wall slope. By contrast, with the latter approach there is a continued and near exponential increase in the maximum wall height when flattening the wall slope beyond 55 degrees.

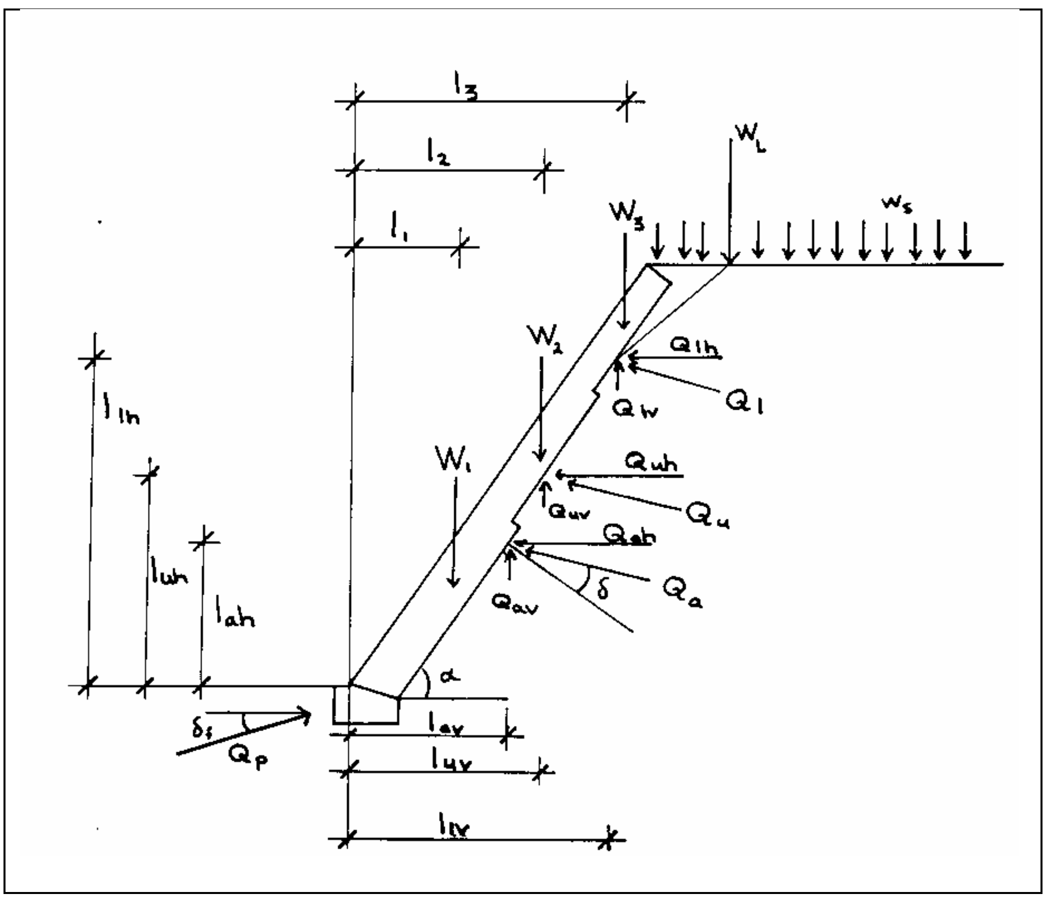
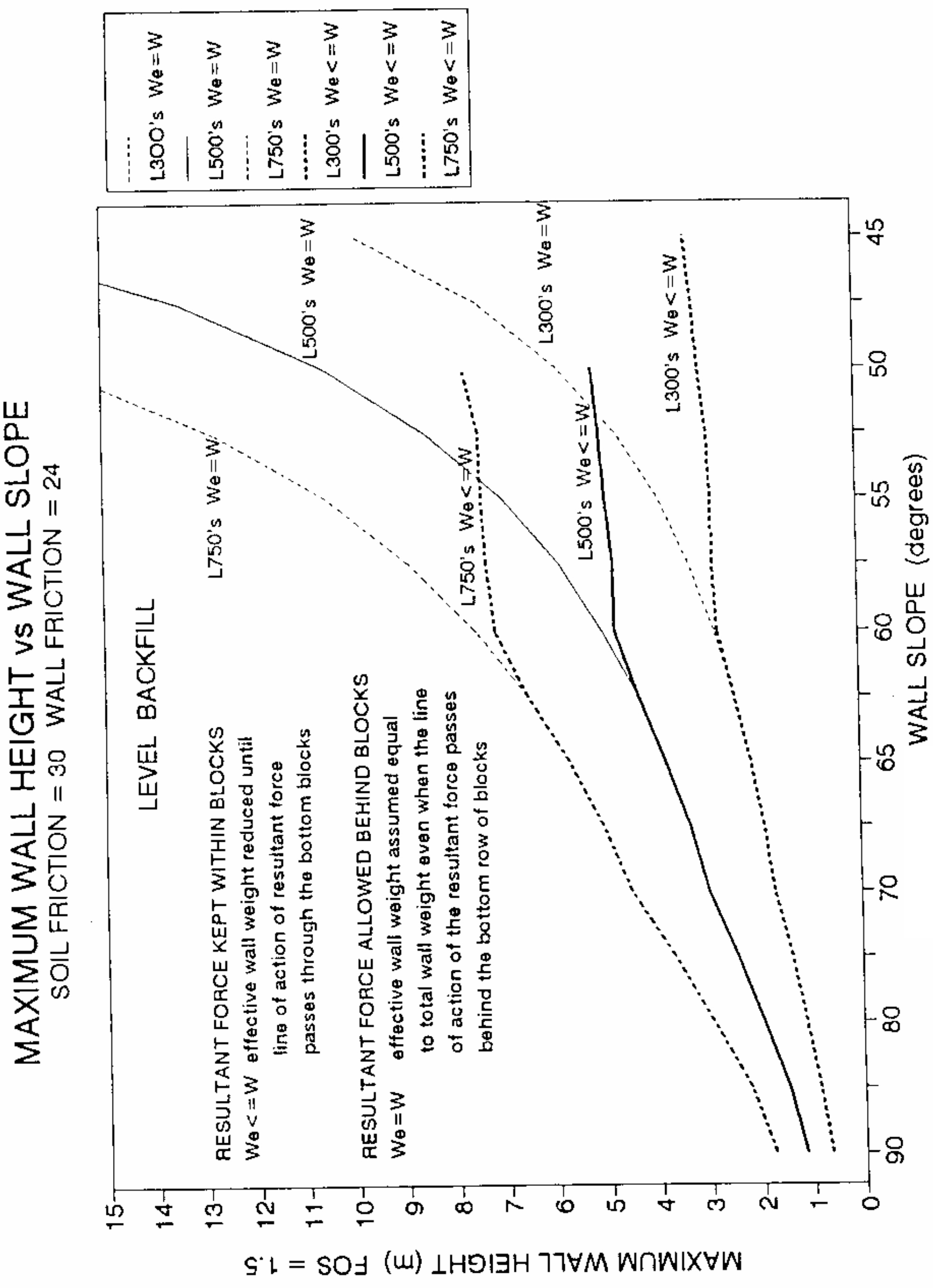
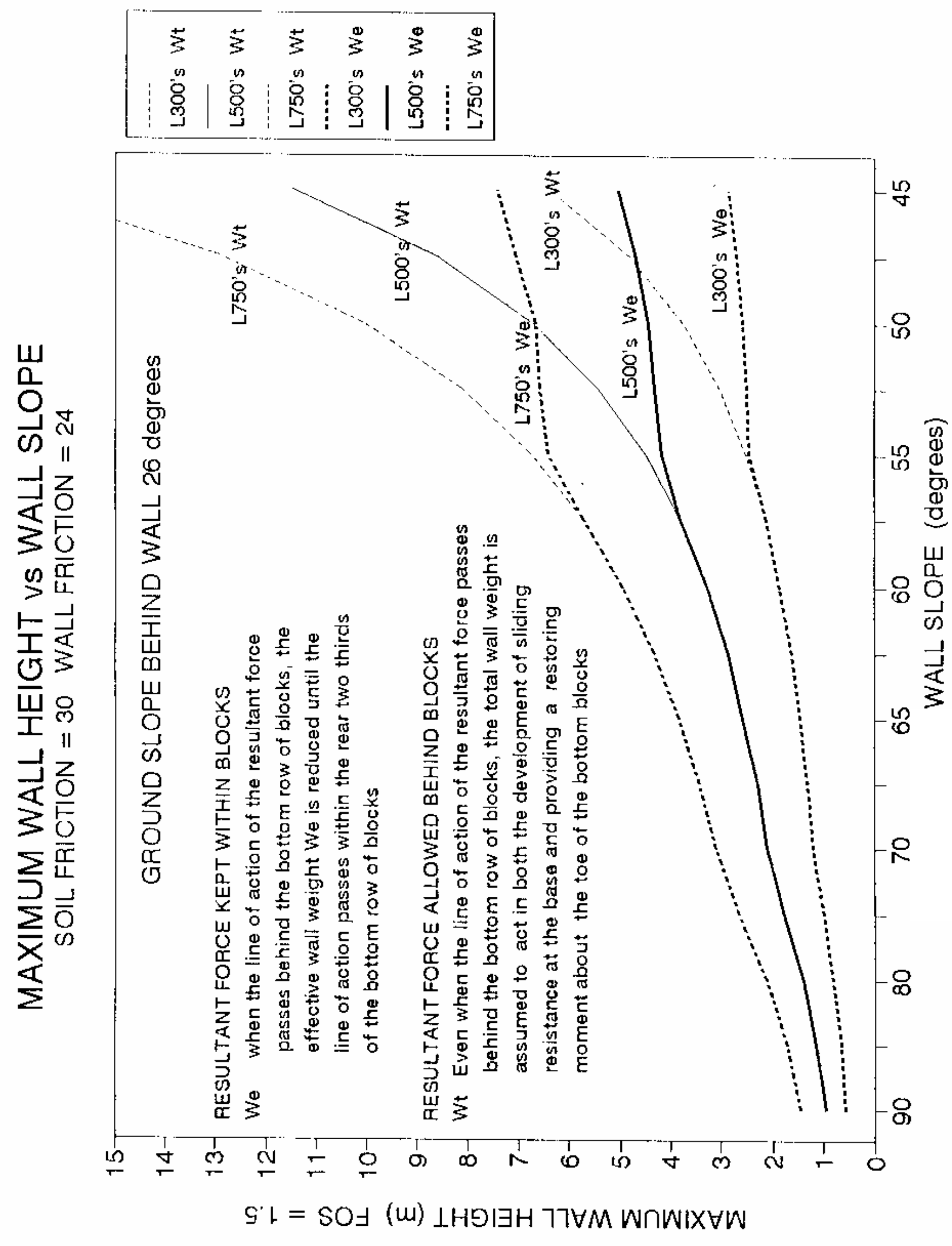


Figure 4 The forces to be considered in analysing a dry stack retaining wall



MAXIMUM WALL HEIGHT vs WALL SLOPE
SOIL FRICTION = 30 WALL FRICTION = 24
LEVEL BACKFILL
RESULTANT FORCE KEPT WITHIN BLOCKS
We<=W effective wall weight reduced until line of action of resultant force passes through the bottom blocks
RESULTANT FORCE ALLOWED BEHIND BLOCKS
We=W effective wall weight assumed equal to total wall weight even when the line of action of the resultant force passes behind the bottom row of blocks
L750's We=W
L500's We=W
L300's We=W
L750's We<=W
L500's We<=W
L300's We<=W
L3OO's We=W
L500's We=W
L750's We=W
L300's We<=W
L500's We<=W
L750's We<=W
MAXIMUM WALL HEIGHT (m) FOS = 1.5
WALL SLOPE (degrees)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
90 80 70 65 60 55 50 45

FIGURE 5



MAXIMUM WALL HEIGHT vs WALL SLOPE
SOIL FRICTION = 30 WALL FRICTION = 24
GROUND SLOPE BEHIND WALL 26 degrees
RESULTANT FORCE KEPT WITHIN BLOCKS
We when the line of action of the resultant force passes behind the bottom row of blocks, the effective wall weight We is reduced until the line of action passes within the rear two thirds of the bottom row of blocks
RESULTANT FORCE ALLOWED BEHIND BLOCKS
Wt Even when the line of action of the resultant force passes behind the bottom row of blocks, the total wall weight is assumed to act in both the development of sliding resistance at the base and providing a restoring moment about the toe of the bottom blocks
L300's Wt
L500's Wt
L750's Wt
L300's We
L500's We
L750's We
MAXIMUM WALL HEIGHT (m) FOS = 1.5
WALL SLOPE (degrees)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
45
50
55
60
65
70
80
90

FIGURE 6

2.6 Calculation of Overturning Factor of Safety

The overturning factor of safety is determined by calculating the restoring moment and the overturning moment about the toe of the second to lowest row of blocks, and dividing the restoring moment by the overturning moment. The blocks in the bottom row are invariably set in wet foundation concrete, and as a result effectively become part of the foundation. It is for this reason that overturning is considered at the level of the second to lowest row of blocks.

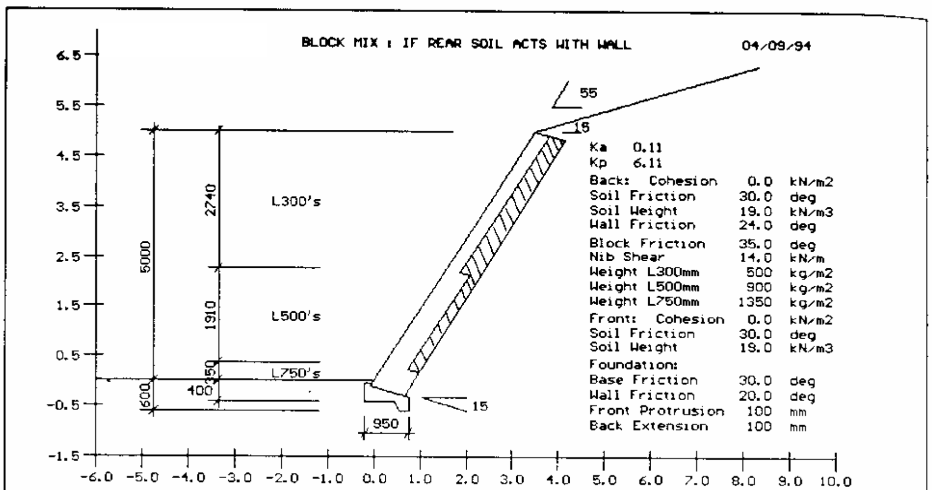
$$\text{Overturning Factor of Safety} = \frac{\text{Restoring Moment}}{\text{Overturning Moment}}$$

Referring to Figure 4 and to Section 2.4:

$$\text{Restoring Moment} = W_1.l_1 + W_2.l_2 + W_3'.l_3' + Q_{av}.l_{av} + Q_{uv}.l_{av} + Q_{lv}.l_{lv}$$
$$\text{Overturning Moment} = Q_{ah}.l_{ah} + Q_{uh}.l_{uh} + Q_{lh}.l_{lh}$$

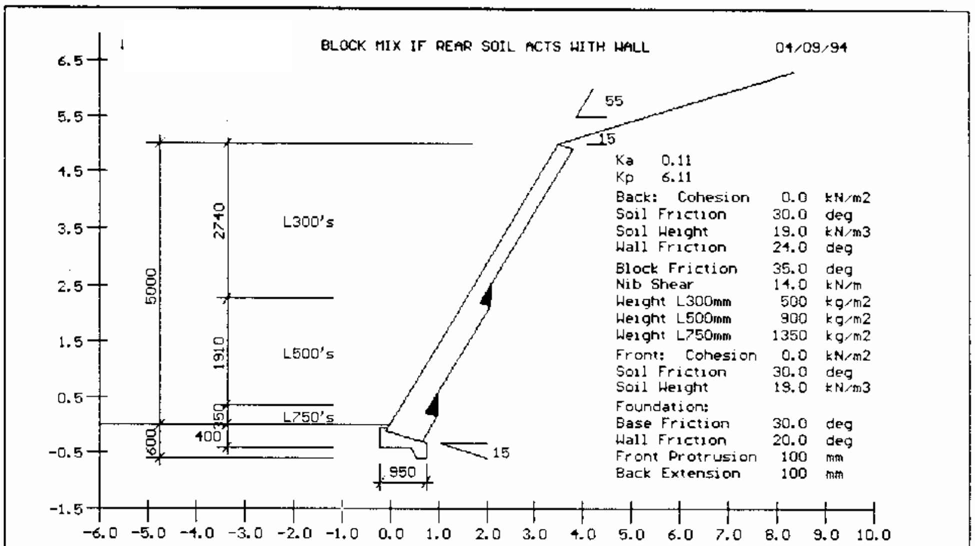
It is recommended that effective weight of the wall be used in the calculation of the restoring moment. Although in the past some designers of CRB walls that comprise a mixture of block sizes, assume that all the soil that lies above the larger lower blocks within a projection of the back line of these blocks, see Figure 7, acts together with the wall in resisting overturning and sliding. It is considered that while this is a reasonable assumption for vertical walls, with sloping CRB walls which typically slope at 70 degrees or flatter, there is no justifiable reason to assume that in such conditions a long narrow inclined block of soil would act as if is were part of the wall. Certainly a relatively small wedge of soil immediately above the top row of the larger blocks would contribute towards the weight of the wall, see Figure 8. However in most cases, other than when the slope of the wall is near vertical, the size of such a wedge is insignificant. Thus for CRB walls that have a batter (wall slope) flatter than 80 degrees, it is recommended that the contribution of any such wedge is ignored.

It is usual to take the point of application of the active force on the rear of the wall as being at a third the height of the wall (h/3) and the point of application of the force due to a uniformly distributed load (Q_u) as being half way up the wall (h/2).



		DESIGN	L300's	L500's	L750's
Maximum Wall Height	(m)	5.00	2.74	4.65	6.97
Founding Depth	(m)	0.60	0.32	0.54	0.83
Overturning FOS		1.17	1.50	1.50	1.50
Block Sliding FOS (-Nib)		1.48	1.80	1.89	1.89
Block Sliding FOS (+Nib)		2.31	4.79	2.94	2.36
Found Sliding FOS		1.47	1.50	1.50	1.50
Found Front Pressure	(kN/m2)	-	0.00	0.00	0.00
Found Back Pressure	(kN/m2)	-	67.07	174.78	348.52
Resultant's Base Int.	(m)	0.20	0.29	0.48	0.72

Figure 7 Diagrammatic illustration showing block of soil that in the past some designers have assumed would act together with the wall to resist sliding and overturning.



		DESIGN	L300's	L500's	L750's
Maximum Wall Height	(m)	5.00	2.74	4.65	6.97
Founding Depth	(m)	0.60	0.32	0.54	0.83
Overturning FOS		1.17	1.50	1.50	1.50
Block Sliding FOS (-Nib)		1.48	1.80	1.89	1.89
Block Sliding FOS (+Nib)		2.31	4.79	2.94	2.36
Found Sliding FOS		1.47	1.50	1.50	1.50
Found Front Pressure	(kN/m2)	-	0.00	0.00	0.00
Found Back Pressure	(kN/m2)	-	67.07	174.78	348.52
Resultant's Base Int.	(m)	0.20	0.29	0.48	0.72

Figure 8 Diagrammatic illustration showing the scale of the wedges of soil that undoubtedly directly contribute to the weight of the wall.

2.7 Calculation of Factor of Safety against Block on Block Sliding

The factor of safety against block on block sliding is calculated at the most critical level, i.e. between the bottom two rows of blocks.

$$\text{Block on Block Sliding Factor of Safety} = \frac{\text{Resisting Force}}{\text{Mobilising Force}}$$

where,

Resisting Force = $R \cdot \sin(\alpha + \omega) \cdot \tan(\rho) + N_S$

Mobilising Force = $R \cdot \cos(\alpha + \omega)$

α = Inclination of the resultant force to the horizontal

ω = Backwards tilt/inclination of the blocks

R = Resultant force

ρ = block on block friction angle

N_S = nib shear strength of the blocks per metre run of wall

Note that the nib shear strength of the blocks, for block types that posses nibs, should only be taken into account if the wall is constructed so that each row of blocks is placed hard up against the nibs of the row below. In practice this is achieved by ensuring that the sum of the angle of the backwards tilt of the blocks and the design slope angle of the wall equals the maximum slope for that type of block wall. For example if the maximum wall slope of the blocks is 70 degrees, and the design slope of the wall is 55 degrees, then the backwards tilt of the blocks should be 15 degrees.

It is thus always good construction practice to ensure that the nib interlock is achieved. With some types of retaining blocks which do not have nibs, the use of precast (or cast insitu) concrete keys wedged between rows of blocks are recommended as a substitute for nibs. In cases where such keys are to be installed, they should be included in the block on block sliding resistance in a similar manner to nibs.

2.8 Determination of a Suitable Founding Depth

Initially a trial founding depth is selected, either based on experience with walls in similar conditions or an arbitrary depth of say 0.5 metres is selected. Then the factor of safety against foundation sliding is calculated as shown on the following page. If the factor of safety is found to be too low then the founding depth would have to be increased until a satisfactory factor of safety is obtained. This iterative approach is well suited to computer analysis.

:

$$\text{Foundation Sliding Factor of Safety} = \frac{\text{Resisting Force}}{\text{Mobilising Force}}$$

where,

Resisting Force = $(Q_{av} + Q_{uv} + Q_{lv} + Qp.\sin(\delta_f) + W_e + W_f)\tan(\mu) + Qp.\cos(\delta_f)$

Mobilising Force = $Q_{ah} + Q_{uh} + Q_{lh}$

and

Q_a = Active Force applied by the rear soil pressure down to the base of the found

Q_p = Passive Force applied by the front soil pressure down to the base of the found

δ_f = Wall Fiction between the front of the foundation and the soil

μ = Friction angle between the base of the foundation and the soil

W_e = Effective weight of wall determined as described in Section 2.6

W_f = Weight of the foundation

Note: the remainder of the variables are as defined in Section 2.5

The active force (Q_a) is calculated in the same manner as described in Section 2.4, the only difference being that the height of the active pressure distribution extends down to the base of the foundation and not just over the retained height of soil.

The foundation pushing against the soil in front of the wall, induces what is termed a passive pressure state. There is a Muller-Breslau solution for the calculation of the passive force, which is given below in the form it appears in C R I Clayton and J Milititsky (1986). As with the active case, the solution assumes that the failure occurs on a critical discrete planar shear plane, and that the soil is rigid, frictional and cohesionless.

$$Q_p = \tfrac{1}{2}\gamma H^2 . \frac{f_2}{\sin\alpha\cos\delta}$$

where

$$f_2 = \frac{\sin^2(\alpha - \phi)\cos\delta}{\sin\alpha.\sin(\alpha+\delta)\left[1 - \sqrt{\left[\frac{\sin(\phi+\delta).\sin(\phi+\beta)}{\sin(\alpha+\delta).\sin(\alpha+\beta)}\right]}\right]^2}$$

In most designs, the passive force resisting foundation sliding is calculated on the basis of the above equation with the founding depth substituted for H. Thus it is generally assumed that the soil is purely frictional. As with the computation of the active force, in order to allow for soil cohesion in cases other than with the straight forward cases with horizontal ground in front of the wall, designers have had to resort to cumbersome time consuming graphical techniques. One such technique is explained in C R I Clayton and J Milititsky (1986). This technique assumes a combined curved and planar slip surface.

2.9 Calculation of Foundation Bearing Pressures

A method that can be utilised for calculating the foundation bearing pressures beneath the back and front of the foundation is the standard method applicable to eccentrically loaded foundations, and essentially is as follows:

$R_V = R \,.\, \sin(\psi + \omega)$

$E_f = B / 2 - X$

if the line of action of resultant force R intersects the foundation behind its front two thirds, then Found Front Pressure $= 0$

and Found Back Pressure $= 2R_V / 3 / (B - X)$

if the line of action of R intersects the foundation within its front third, then

Found Front Pressure $= 2R_V / 3 / X$

and Found Back Pressure $= 2R_V / 3 / (B - X)$

if the line of action of R intersects the foundation within its middle third, then

$M = 6\, R_V \,.\, E_f / B^2$

Found Front Pressure $= R_V / B - M$

and Found Back Pressure $= R_V / B + M$

where R_V = vertical component of the Resultant Force

E_f = Eccentricity of the Resultant Force from the middle of the foundation

X = Distance from the front of the foundation to the point of intersection of the line of action of the resultant force with the foundation

B = Width of the Foundation

ψ = Inclination of the resultant force to the horizontal

ω = Inclination of the top surface of the foundation to the horizontal

Note that the weight of the foundation has been ignored.

If the computed bearing pressure beneath either the front or back of the foundation is unacceptably high then it should be reduced by increasing the foundation width

REFERENCES

Clayton, C. R. I. and Milititsky, J. (1986). Earth Pressure and Earth Retaining Structures. Surrey University Press.

Smith, G.N. (1982). Elements of Soil Mechanics for Civil and Mining Engineers. Granada Publishing Limited.

3. TESTING OF CRB BLOCKS

In order to carry out a design for a particular type of CRB block wall, a designer needs the following design parameters for the block, viz.

(i) The length, width & height of the block.

(ii) The estimated weight per square metre of the CRB blocks combined with the any soil that will be filled in and between the blocks (applicable to CRB block walls with open spacing).

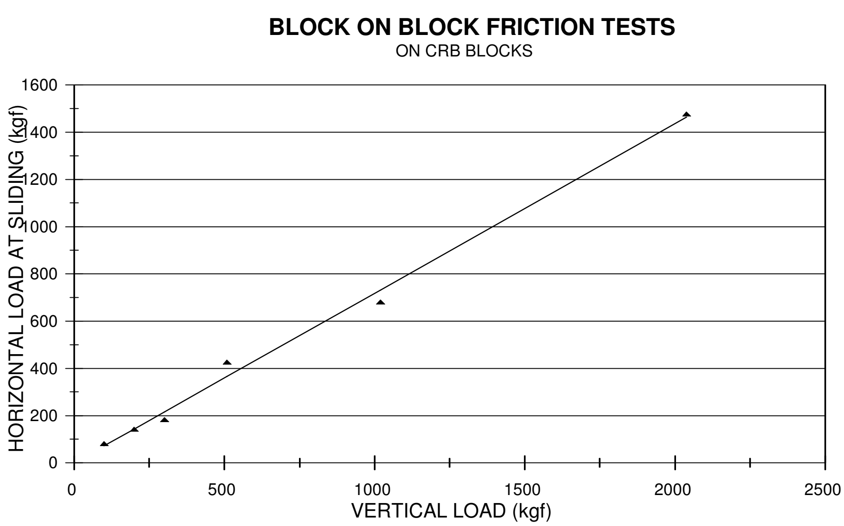
(iii) The coefficient for block on block friction, expressed as tan ϕ_b. As explained below, tan ϕ_b, is obtained from block on block friction tests. In the absence of such tests, it is recommended that a value of $\phi_b = 32$ degrees is assumed.

(iv) The nib shear strength per metre run of wall in cases where the CRB units have interlocking nibs.

(v) The crushing strengths of the CRB block applicable to the retaining conditions. Appropriate tests to determine meaningful crushing tests are described below.

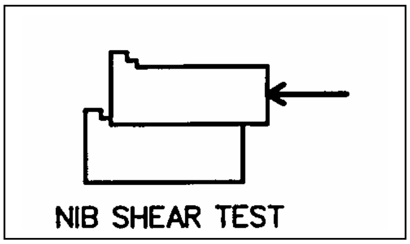
3.1 Block on Block Friction Tests

The coefficient for block on block friction can be determined by placing one CRB block on top of two other CRB blocks, positioned slightly back from the front nibs of the lower blocks (if the blocks have nibs). Then a series of vertical loads is applied to the top block, and for each load a horizontal applied to the top block is increased until sliding of the block occurs. In the cases of the relatively low vertical loads, up to 300 kg, the load can be applied using stacked steel weights. In the cases of the higher vertical loads, the load can be applied by means of a hydraulic jack to a steel plate supported on steel rollers off another steel plate on top of the block.



3.2 Nib Shear Strength Tests

Nib shear strength tests can be carried out on CRB blocks by supporting the front of each of the blocks tested at the bottom corners, at the positions where the nibs of the blocks below would normally occur, and then applying an increasing horizontal load to the front nib at the outside corners of the nib. The horizontal load at failure being the nib shear strength of each block. We recommend that the design nib shear strength be taken as 80 % of the characteristic shear strength of the nibs, i.e. the shear strength below which only 5 % of the nibs of blocks are expected to fail.



3.3 Crushing Strength Tests

It is considered that a crushing strength determined by applying a vertical load to the entire area of the top of a retaining block, is a meaningless strength in terms of the design and performance of dry stack CRB walls. This is because generally, at the maximum retaining heights of the dry stack block walls, the line of the resultant force on the wall, passes through the back edge of the bottom row of blocks, and in theory there is infinite stress on the blocks along this line. It should be noted that in the case of steep high walls, if the active pressures are higher than anticipated, the situation could arise that the resultant force passes through the front of the blocks.

To determine the crushing strength of CRB blocks under these two extreme potential crushing conditions the following tests are recommended.

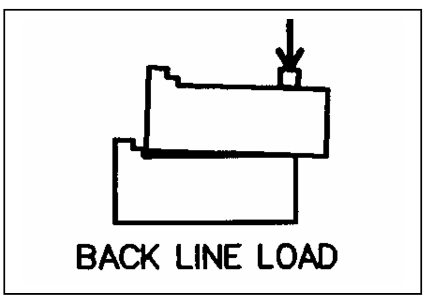
(i) **Back Line Load Crushing Strength**

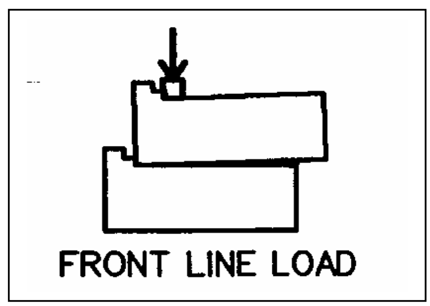
To simulate the case in which the resultant force's line of action passes through the back edge of the bottom blocks; it is recommended that the following crushing test be carried out. Support a block on top of two other blocks with the front of the block raised by about 10 mm using a spacer on each side. Position a 50 mm by 50 mm steel bar across the back of the top block, immediately in front of the line of support at the rear of the bottom blocks, then increase the vertical load on the bar until failure of the block occurs. The vertical load at failure represents the rear line crushing strength of the CRB block.

(ii) **Front Line Load Crushing Strength**

To simulate the case, in which the resultant force's line of action passes through the front edge of the second to lowest row of blocks in a CRB wall, it is recommended that the following crushing test be carried out. Support a block on top of two other blocks with the back of the block raised by about 10 mm using a spacer on each side. Position a 50 mm by 50 mm steel bar across the front of the top block, immediately in behind the line of support towards the front of the bottom blocks, then increase the vertical load on the bar until failure of the block occurs. The vertical load at failure represents the front line load crushing strength of the CRB block.

It should be appreciated that in both cases, the block being crushed is effectively supported on a knife-edge, close to the line of application of the load.

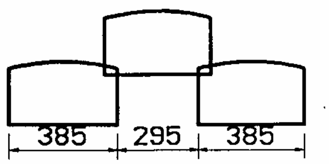




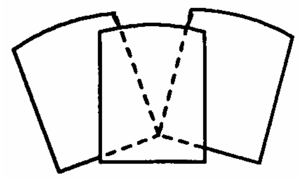
CRB blocks that are generally installed with an open spacing, it is recommended that the front and back crushing tests be carried out both with the supporting two blocks at the standard block spacing and with these two blocks at a closed block spacing. For some blocks it may also be prudent to perform crushing tests on the blocks at some intermediate spacing that is likely to result in the worst case crushing strength of the block. A worse case block spacing commonly occurs within curved sections of walls where the blocks are not supported evenly on the shoulders of the supporting blocks. Refer to the Figures below for a clearer understanding of the various block spacing configurations.

Note that tests on a certain CRB indicated that for tight corner conditions, the rear line load block crushing strength should be reduced by about 30%.

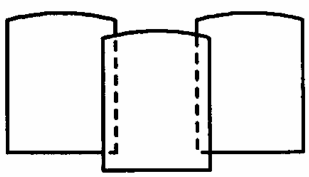
It is recommended that in the design of relatively high CRB retaining walls (>2.5 metres), the resultant force should be computed, and that its vertical component should be at least 20 % less than the relevant characteristic crushing strength of the blocks.



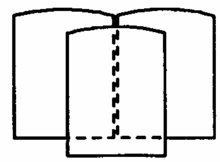
STANDARD BLOCK SPACING



CONVEX CURVE

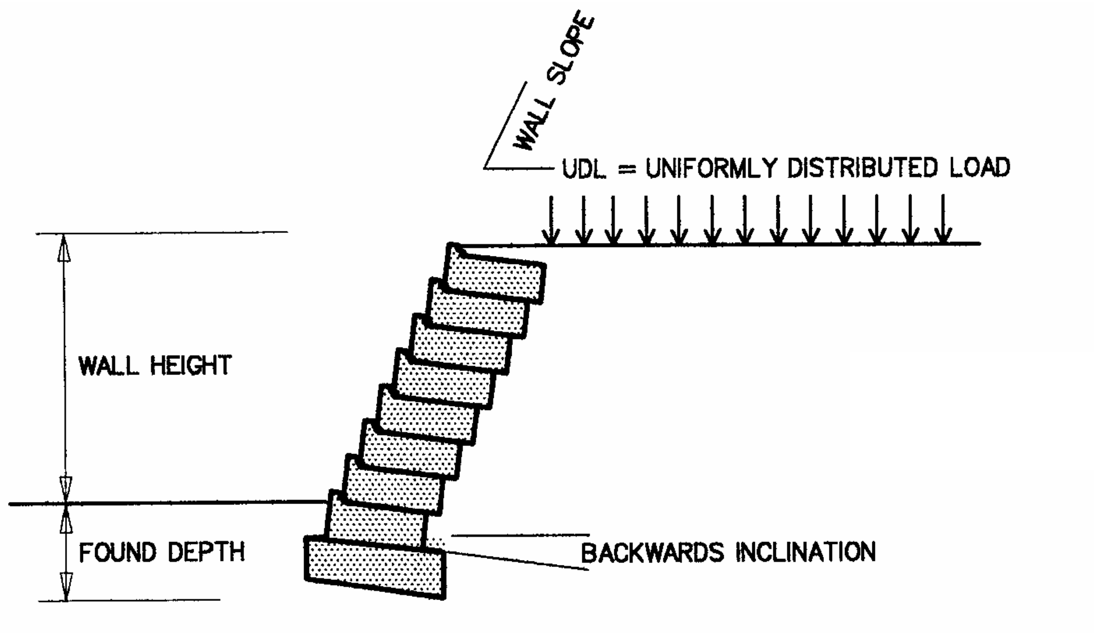


STANDARD BLOCK SPACING



CLOSED BLOCK SPACING

4. DETAILING & INSTALLATION OF CRB WALLS



4.1 FOUNDATIONS

The founding depth specified for a CRB Wall should take into account the lowest depth of any excavation that is to take place immediately in front of the wall. For example if a road pavement is to be constructed in front of a wall after the installation of the wall, then the founding depth should be specified below the underside of the road pavement. Another situation in which care should be exercised when specifying the founding depth is when service trenches are to be excavated immediately in front of the CRB Wall. In such instances the specification of the founding depth should take cognisance of the anticipated maximum depth of the service trench excavation. Founding depths should generally be specified as minimum founding depths, and the final founding depths for a wall should be confirmed by the Engineer on site.

The minimum, concrete strength, the minimum foundation thickness and the minimum foundation width should be specified for the wall.

For general applications a minimum factor of safety of 1.5 is recommended for foundation sliding resistance, but when structures are in close proximity to the wall a minimum FOS of 2 should be considered.

Notwithstanding the results of any analysis to determine the minimum founding depth for a CRB retaining wall, the following minimum foundation depths should be specified in cases where the analysis indicates a shallower founding depth would provide an acceptable FOS against sliding. Obviously in cases where competent bedrock that is not pickable, obtains at a shallower depth than the minimum founding depth given in the table, it is likely that it will not be necessary to achieve the specified minimum founding depth.

CRB RETAINING WALL HEIGHT (m)	**FOUNDING DEPTH** MINIMUM ALLOWABLE Applies in cases where the analysis indicates a shallower depth is acceptable. (mm)	**FOUNDATION THICKNESS** MINIMUM ALLOWABLE (Assuming 20 MPa Concrete) (mm)
<1.2 m	0.3	100
- 2.0 m	0.4	150
- 3.0 m	0.5	200
- 4.0 m	0.6	200
> 4.0 m	0.7	250

Note that unless there is a likelihood of localised soft spots beneath the foundations, it is not usual to reinforce the foundations of a CRB wall. In cases where the foundation is keyed into the subsoils to achieve greater sliding resistance, the need for reinforcement of the key will have to be considered, and the reinforcement is likely to be required when the depth of the key exceeds 250 mm.

4.2 TOLERANCES ALLOWED IN THE CONSTRUCTION OF A CRB WALL

4.2.1 TOLERANCES WITHIN THE LAYING OF ANY SINGLE ROW OF BLOCKS

In most instances CRB Walls are installed with the rows of blocks laid horizontally. There have been exceptions to this rule on slopes with relatively flat and even inclines. However in general the laying of CRB blocks on the incline is not recommended for the usual case of CRB blocks being laid row by row horizontally, the variation from level should not exceed 20mm within any metre and should not exceed 50mm over the length of the wall. In the rare cases where the blocks are laid parallel to the incline of a uniform ground slope, the above tolerances apply to the deviation from the slope of the ground.

4.2.2 TOLERANCES TO BE ALLOWED IN THE SLOPE OF A CRB WALL

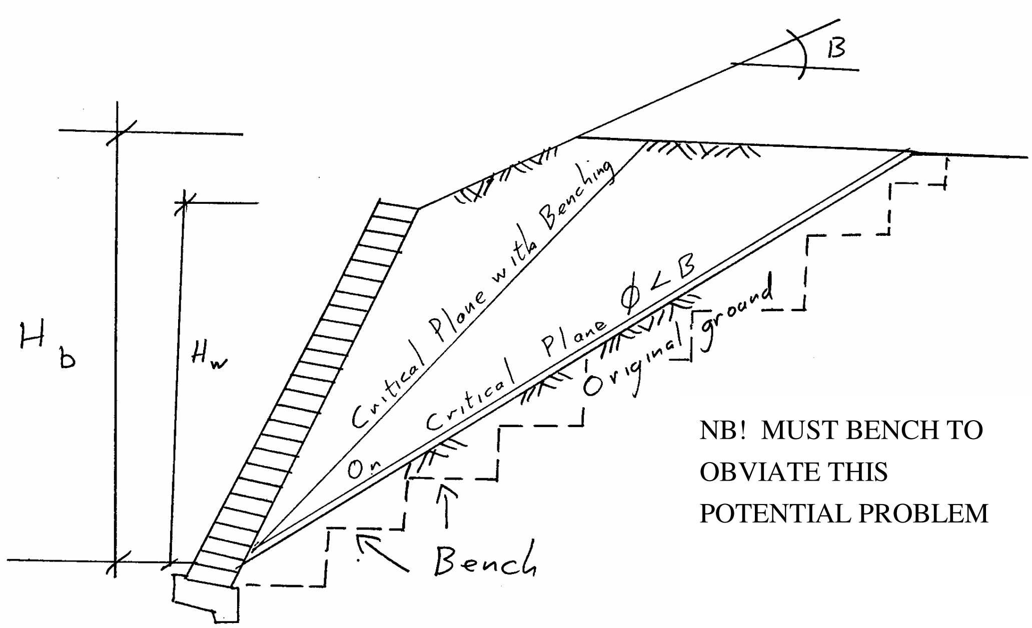
The designer of a CRB Wall will have specified an angle for the slope of the wall. Since the angle of CRB Walls often changes around corners, the acceptable tolerances in the deviation from the specified slope of a CRB wall is applicable to straight lengths of walls. The acceptable tolerance from the specified wall slope angle is plus 1 degree or minus 2 degrees, e.g. if specified wall slope is 65 degrees then the actual wall slope should be between 63 degrees and 66 degrees.

4.3 COMPACTION OF BACKFILL

We recommend that backfill placed within the blocks be compacted to a minimum of 90% of its maximum modified AASHTO density, and that the backfill placed behind the blocks be compacted to at least 93% of its maximum modified AASHTO density.

4.4 BENCHING OF BACKFILL

When building a CRB Wall either on or at the toe of an existing slope be it a fill slope or a natural slope, the backfill must be benched into competent material within the existing slope. The minimum width of the bench within the competent material should be at least 500mm. The objective of the benching being to avoid a potential slip plain at the interface of the new backfill behind the CRB Wall and the existing slope.



4.5 CEMENT/LIME STABALIZED BACKFILL

In many instances where the height to be retained by a CRB Wall is such that the effective thickness of the CRB Wall needs to be increased beyond the length of the block, cement stabilised backfill is utilised to achieve this end. This cement-stabilised backfill needs to be strong enough to form what is commonly termed a soilcrete. Thus it is vital that the percentage stabilisation used and the type of backfill stabilised are such that they form a uniform cemented material. In many instances it will be necessary to import a good quality granular material. As a guide the cement stabilised backfill/soilcrete, should conform to the following specifications.

(i) The unconfined compressor strength (UCS)of the soilcrete should be at least 2 MPa.
(ii) The percentage cement/lime should be specified by percentage weight of the total cement stablised material's weight.
(iii) The tolerance allowed for the percentage stabilisation should be such that even at the minimum acceptable percentage, the desired UCS of the material is achieved.
(iv)The minimum degree of compaction of the backfill should be given along with the percentage stabilisation. Unless otherwise specified the minimum degree of compaction should be at least 93% modified AASHTO density.

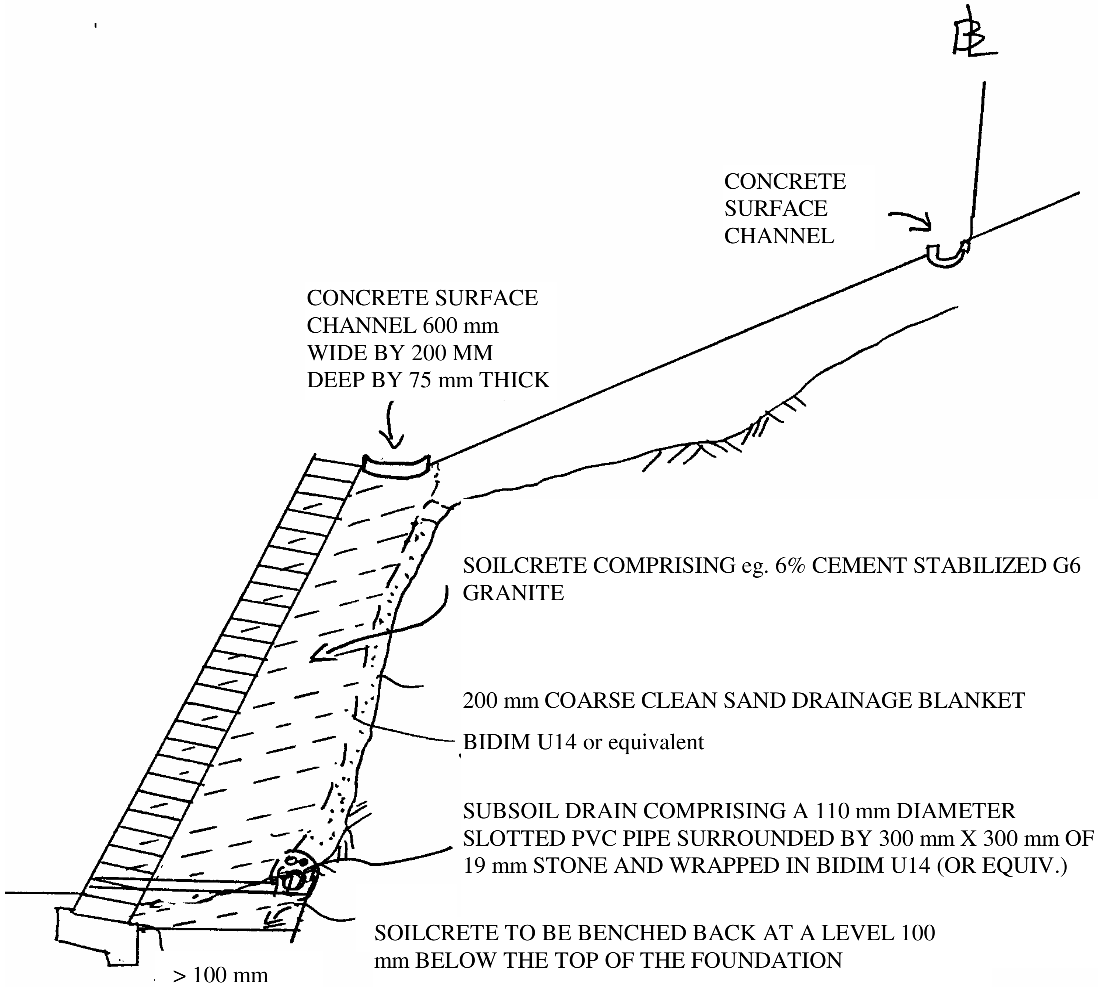
In order to achieve a uniform soilcrete, it is generally advisable to mix the soilcrete out of position.

Testing of percentage stabilisation and of percentage compaction of the stabilisation should be carried out at least 1m lifts over the height of the stabilisation, at the discretion of the engineer.

4.6 SUBSOIL DRAINAGE BEHIND CRB WALLS

4.6.1 SUBSOIL DRAINAGE BEHIND CONVENTIONAL CRB WALLS

Conventional CRB Walls by their very nature are free draining and therefore generally do not require weep holes. However in areas of high subsoil seepage. The seepage can result in leaching out of the backfill within and behind and the CRB blocks, and the seepage may lead to excessive saturation of the backfill. In circumstances where there is relatively strong ground water seepage, or a potential for such seepage, subsoil drains should be installed behind the CRB Wall. Depending upon the potential severity of the subsoil seepage, the subsoil drainage should comprise either an agricultural/subsoil drain behind the base of the wall, or an agricultural/subsoil drain connected to a drainage blanket that which up the back of the wall along the interface of the backfill and the original ground. See the figure on the following page.



SUBSOIL DRAINAGE MEASURES IN AN AREA IN WHICH
RELATIVELY HIGH GROUND WATER SEEPAGE IS ANTICIPATED

An agricultural/subsoil drain typically comprises a 100 mm diameter slotted PCV pipe surrounded by 300mm x 300mm of 19mm stone and wrapped in a highly porous geofabric.

The drainage blanket will typically be 200mm of coarse clean sand sandwiched between porous geofabric. In cases of exceptionally high subsoil seepage it may be necessary to utilise stone in the place of clean sand within the drainage blanket.

4.6.2 SUBSOIL DRAINAGE IN CASES OF CEMENT/LIME STABALIZED BACKFILL

Subsoil drainage is particularly important to include behind CRB retaining walls, which comprise CRB Blocks in combination with stabilised backfill. If the CRB Wall is to be constructed in an area where subsoil seepage is not expected, then it will suffice to incorporate coarse clean sand " weep holes" at regular intervals through the cement stabilised backfill. Typically such weep holes would be at least 150mm x 150mm and be 1.5 metre centres. The first row of such weep holes would be at 200mm above finished ground level in front of the wall, and the weep holes would be repeated at 1 metre lifts up the height of the stabilisation, see Figure …

In cases where subsoil seepage is anticipated behind the stabilised material, then a drainage blanket should be installed behind the stabilised material over it's full height, and this should be connected to a subsoil drain behind the base of the wall.

4.7 CONTROL OF STORM WATER BEHIND CRB WALLS

Most failures of sections of CRB Walls occur because of insufficient control of storm water behind the top of such CRB Walls. These failures generally occur because there has been a concentration of storm water at some point behind the wall. Thus it is vital to ensure that no storm water concentrates at any point behind a CRB Wall. In the case of CRB Walls, which have appreciable catchment areas behind the wall, a concrete surface channel should be installed behind the top of the wall. The surface channel should be sized in accordance with the anticipated storm water run-off.

In all instances it is preferable to manage the storm water in such a manner as to obviate excessive scour behind the CRB Wall and overtopping of the wall. Even in cases where minimal storm water run-off is expected behind the top of the wall, it is good practice to run a geofabric up the back of the wall, and tuck it underneath the top row of blocks. Clearly in cases where stabilized backfill is used behind and within the blocks, a geofabric will not be required behind the blocks. However if the backfill is not stabilized over the full height of the wall, a geofabric should be installed behind the blocks, above the level of the stabilized backfill.

4.8 TYPICAL NOTES TO BE INCLUDED ON A CRB WALL DRAWING

NOTES

1. Engineer to approve founding conditions prior to casting of concrete.

2. Either set the bottom row of blocks in wet concrete or form a 100 mm high by 100 mm wide nib on the front of the foundation.

3. Concrete strength to be 20 MPa at 28 days.

4. Engineer's certificate of completion to be provided upon completion.

5. All backfill to be compacted to 93% MOD. AASHTO density.

6. Unless otherwise agreed with engineer, contractor to supply engineer with results of compaction tests, and when applicable, percentage stabilisation tests on the backfill.

7. Storm water behind the top of the wall to be managed in such a manner as to obviate scour behind or over-topping of the wall.

8. 170mm X 170mm clean sand drainage weepholes through the stabilisation at 2m centres. First row 200mm above FGL and repeat in 1m lifts.

9. Where applicable, backfill to be benched into competent original ground.

5. MEASUREMENT AND PAYMENT FOR CRB WALLS

Unless otherwise agreed between the developer and CRB Walling Contractor, the following criteria should apply for the measurement and payment of a CRB Wall. A vitally important aspect of preparing a bill of quantites for a CRB walling contract is to ensure that typical cross sections through the wall and are issued in conjunction with the bill of quantities. Even in cases in which the wall the contractor is required to submit a "design, supply and construct" quote/tender, information should be provided in the form of sections and elevations that show any details relevevant to the design of the wall, such as the slope of the wall, ground slopes behind and/or in front of the wall, parking areas or buildings in close proximity to the top of the wall, and drainage requirements.

5.1 Typical Bill of Quantities

Bill of Quantities					
Item	**Description**	**Unit**	**Quantity**	**Rate**	**Amount**
1	Preliminary and General	Sum			
2	Trimming of batter faces to correct angle	m^3			
3	Placement and compaction of fill to 93% ModAASHTO density behind the retaining wall. This excludes the initial 500 mm immediately behind the CRB blocks, which should be allowed for in the block installation rate (Item 8)	m^3			
4	Trenching for Foundations	m			
5	Clearing and grubbing	m^2			
6	Foundation concrete – 20 MPa at 28 days				
	a) Type 1 – 600 mm wide by 200 mm thick	m^3			
	b) Type 2 – 700 mm wide by 200 mm thick with a 150 mm by 150 mm heel	m^3			
	c) Type 3 – 700 mm wide by 250 mm thick with a 200 mm by 200 mm heel	m^3			
7	Steel Reinforcement within foundations	ton			
8	Supply and install Concrete Retaining Blocks including the placement and compaction of fill within the blocks and for a distance of upto 500 mm behind the blocks. Fill to be supplied by others				
	a) min. 300 mm long/deep CRB blocks with fill so that the min. total wall mass is 500 kg/m^2	m^2			
	b) min. 400 mm long CRB blocks with fill so that the min. total wall mass is 720 kg/m^2	m^2			
	c) min. 500 mm long CRB blocks with fill so that the min. total wall mass is 900 kg/m^2	m^2			
	d) min 400 mm long CRB blocks with stabilised 6% PBFC cement staibilised G6 fill in the blocks and for 600 mm behind the blocks so that the effective block length/depth is 1000 mm and minimum total wall mass is 1900 kg/m^2	m^2			

	OR COMBINING ITEMS 4, 6, 7, 8 & 9 IN CASES OF DESIGN AND SUPPLY				
8	Excavate and cast foundations inclusive of any reinforcement. Supply and install Concrete Retaining Blocks including the placement and compaction of fill to 93% ModAASHTO density within the blocks and for a distance of upto 500 mm behind the blocks. The rate should include for any cement staiblised backfill and or geofabric that the wall's designer may require. Fill to be supplied by others.				
	a) 1 to 2 metre high retaining walls with level ground behind the walls and no other surcharge.	m^2			
	b) 2 to 4 metre high retaining walls with level ground behind the walls and no other surcharge.	m^2			
	a) 1 to 2 metre high retaining walls with a 1 in 2 slope ground slope behind the walls and no other surcharge.	m^2			
	b) 2 to 4 metre high retaining walls with a 1 in 2 slope ground slope behind the walls.	m^2			
9	Supply and place geotextile as per design requirements as specified on Figures ….				
	a) Type 1 eg. Kaymat U12 or U24 or U44	m^2			
	b) Type 2 eg. Kaytape S120 or S210 or S270	m^2			
	c) Type 3 eg. Restrain 50 or 75 or 100	m^2			
	d) Type 4 eg. Geogrid 110 or Fortrac 35/20-20	m^2			
10	Supply and install subsoil drain comprising eg. 110 mm diameter slotted PVC pipe surrounded by 300 mm by 300 mm of 19 mm stone and wrapped in Kaymat U14.	m			
11	Supply and install wickdrains to seepage areas as indicated by engineer on site	m			
12	Supply and place 19 mm stone infill to blocks in areas of relatively high potential seepage behind the wall such as along the interface between an upper permeable colluvial soil layer and an underlying clayey residual soil layer.	m^3			
13	Supply and install a 550 mm wide by 170 mm deep precast surface channel immediately behind the top of the wall.	m			
14	Import G6 material and stabilise it with 6% PBFC cement and compact it in position to at least 95 % ModAASHTO density. Note this iiem is only included if it has not been allowed for in the CRB block installation rate, i.e. in Item 8 above				
15	Engineers design fees				
	a) Design, monitoring of construciton and certification of the wall.	%			
	b) As above but with submission drawings	sum			

5.2 **CRB WALL**

The CRB wall itself should be paid for on the basis of a rate per square metre vertical face area of wall. This rate generally includes the supply of blocks, the laying of the blocks and the backfilling in and behind the blocks. Where cement stabilised backfill is to be utilised in and behind the blocks, the rate for such stabilisation should also be measured and paid for on the basis of a square metre vertical area of the wall face, over which it applies. The vertical face area of the wall is calculated at the vertical height of the wall measured from the top of the foundation to the top of the wall. Thus the area of wall to be paid for will include those rows of blocks installed below finished ground level in front of the wall.

Any geofabric installed behind the wall should also be measured and paid for on the square meter face of wall. The same applies to a drainage blanket specified behind the CRB Wall.

If a concrete surface channels and/or a subsoil drain is installed as part of the CRB wall contract, these are to be measured and paid for on a cost per linear metre basis.

6. MISCELLANEOUS

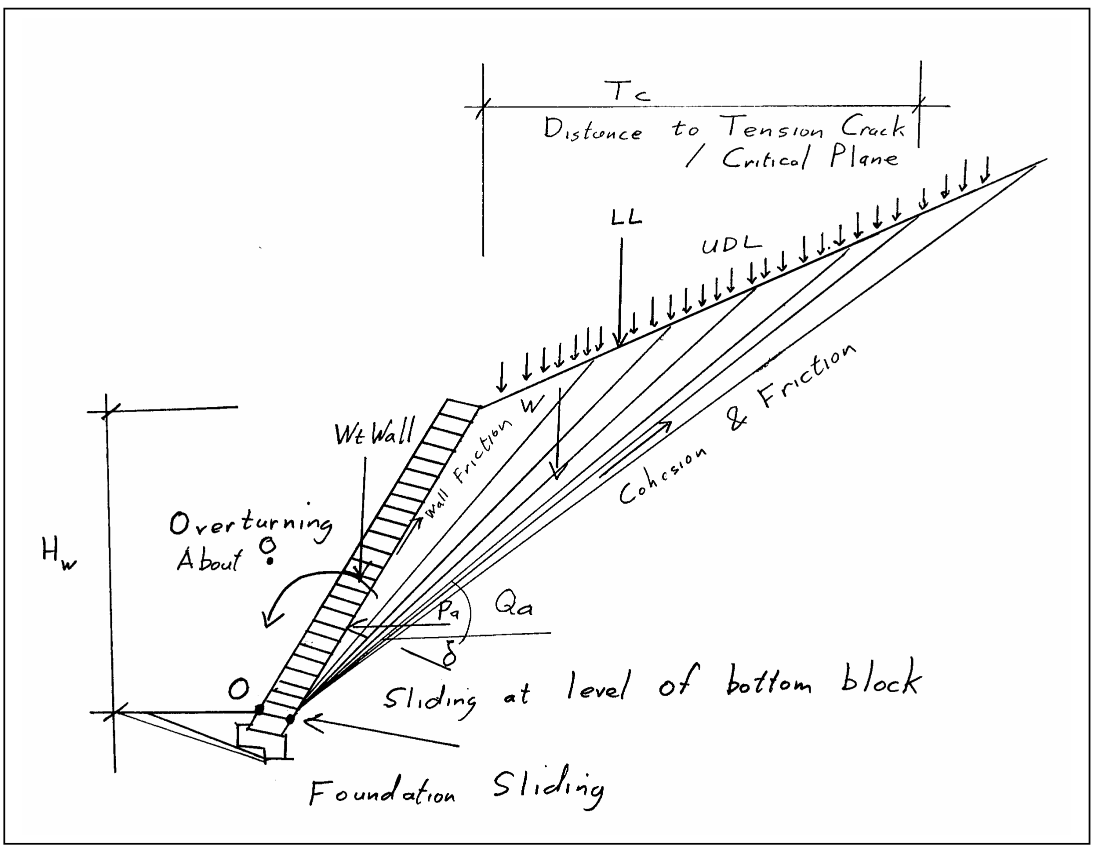
6.1 VARIOUS RETAINING CONDITIONS

It is considered that it is worth taking note of certain aspects of various commonly encountered retaining conditions that are referred to below.

6.1.1 Uniform Soil Retained By a CRB Wall

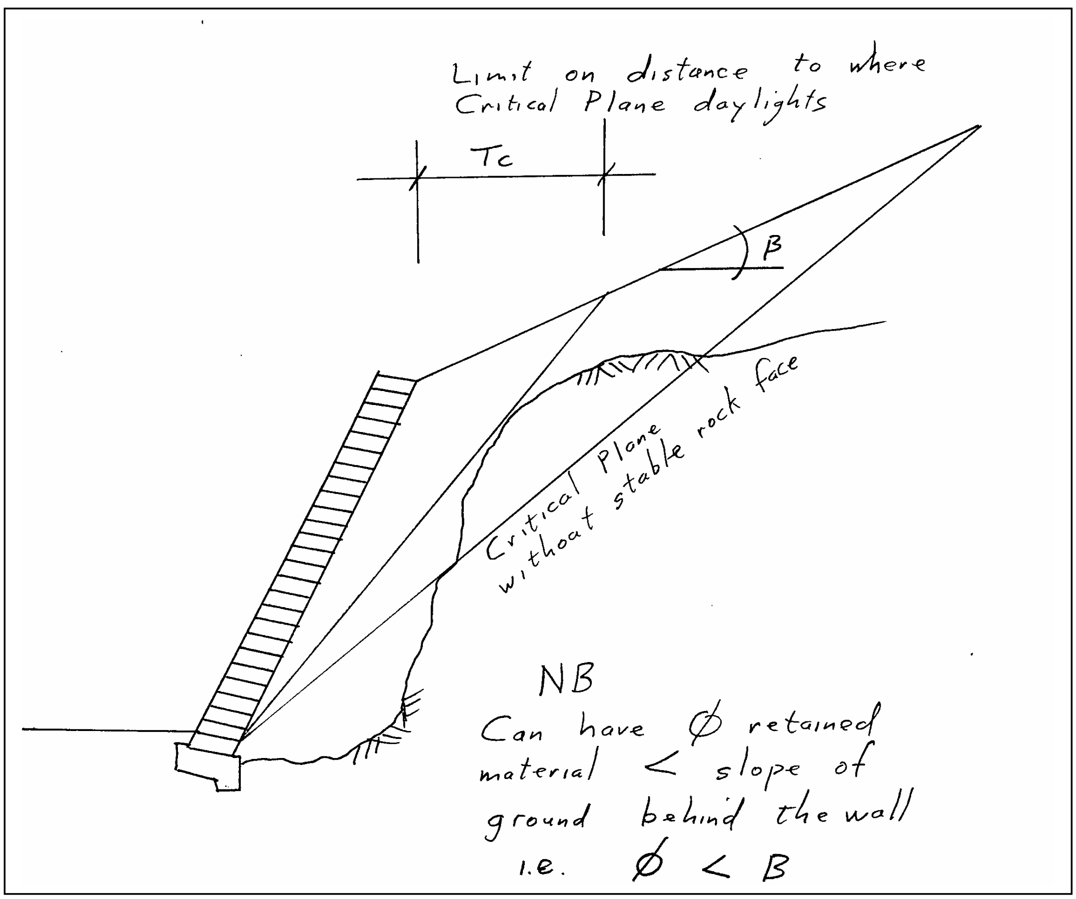
The diagram below shows some of the wedges that would need to be considered in carrying out a wedge analysis to determine the destabilising forces on the wall that would be mobilised by sliding on the critical shear plane within a soil which is assumed to be uniform. This simple case is included here in order to highlight differences when considering more complete conditions.

It should be appreciated that under such conditions there is no restriction on the position at the critical sliding plane will develop. This is in contrast to the following example, which deals with the case in which the wall is constructed in front of a stable rock slope.

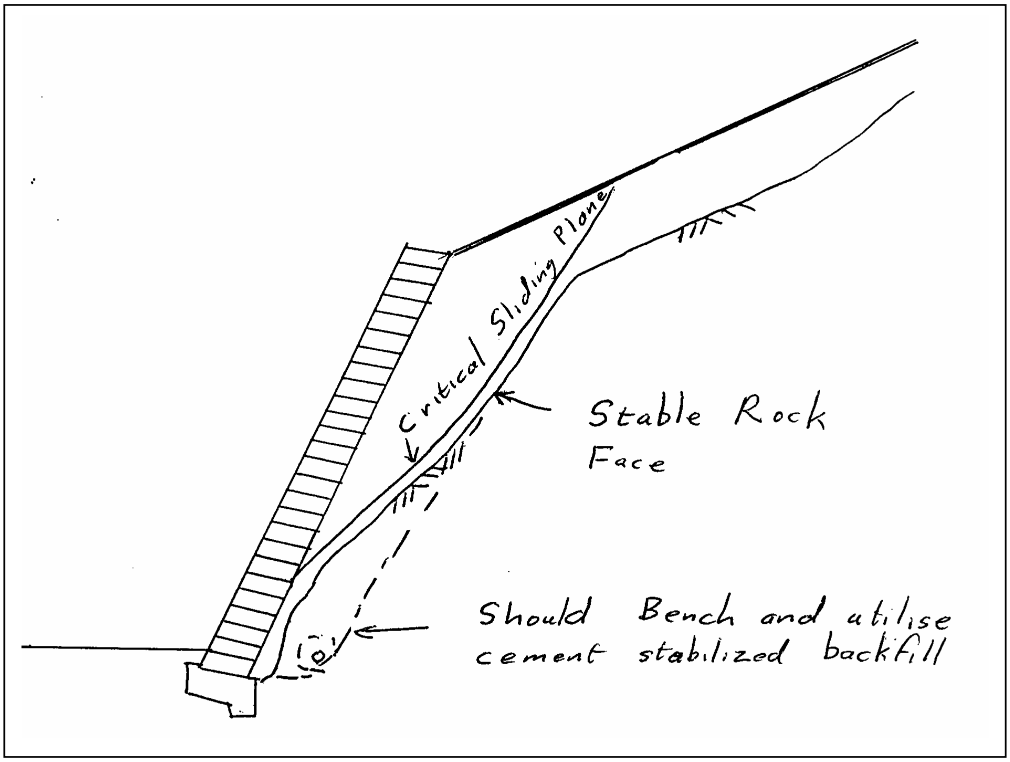


6.1.2 CRB Wall in Front of A Stable Rock Face

For the case of a CRB wall in front of a stable rock face, the distance between the stable rock face and the wall is critical to the magnitude of the destabilising forces imposed on the wall by the backfill between the wall and the rock face. If the distance between the back of the CRB units wall and the rock face is less than 0.2 metres, the forces on the wall will relatively insignificant and the wall will effectively act as a cladding to the stable slope. However in many cases cut slopes are over excavated and instead of the wall acting solely as a cladding to the slope, it has to withstand appreciable destabilising forces imposed by the backfill between the wall and the slope. This condition is illustrated in the diagram below. Note that often the best solution to the problem is to stabilise the backfill between the wall and the slope, thereby negating the potential development of any destabilising forces on the wall, provided that one has included effective drainage measures behind the stabilised backfill. Note that under such conditions it may be prudent within the analysis to use a friction angle that is representative of the coefficient of friction between the backfill and the slope's rock rather than the angle of internal friction of backfill. One should be aware that there is at least one computer programme available that allows one to limit the distance to the critical plane as illustrated in the diagram.

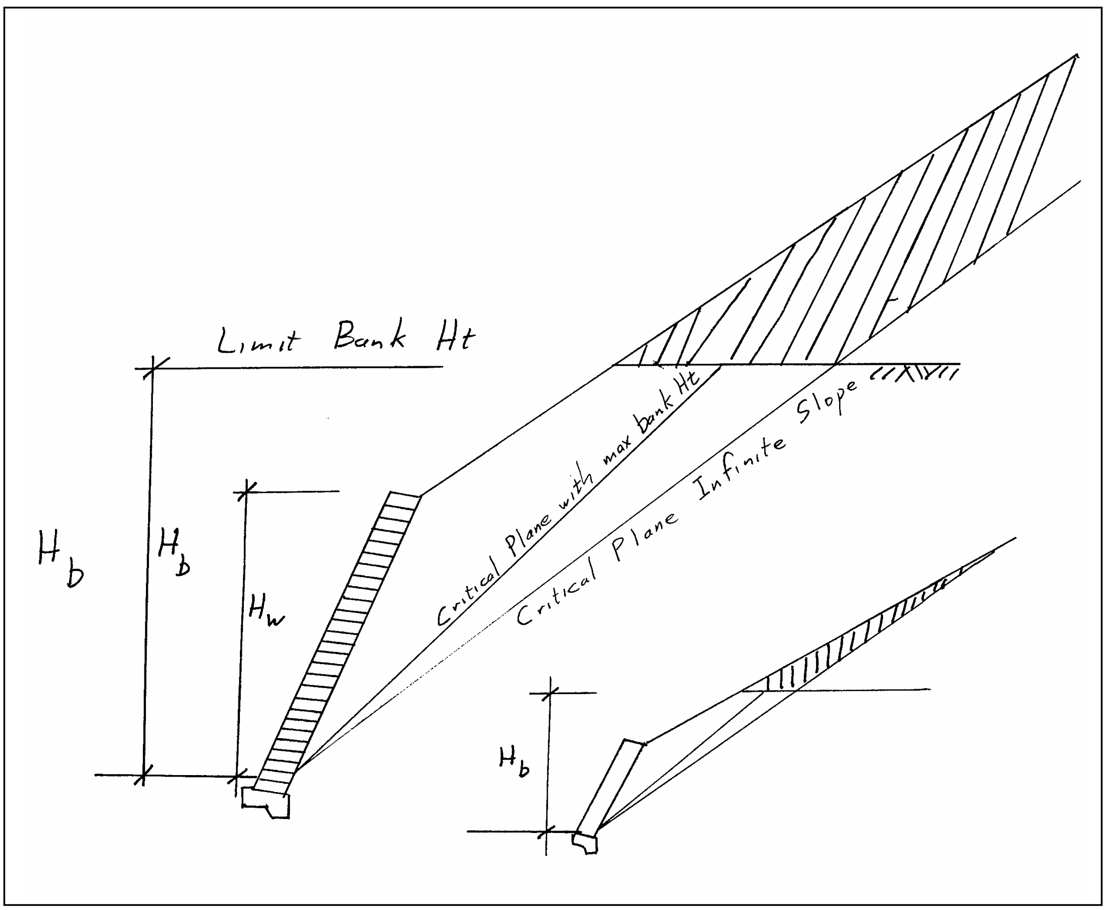


A highly dangerous condition that can arise when cladding a stable rock face with a CRB wall, even when one is cement stabilising the backfill between the blocks and the rock face, is shown in the diagram below. Note that in such a case one should cut back the rock face in such a manner as to ensure that there is a sufficient thickness of soilcrete behind the blocks to prevent sliding on the critical backfill/rock face interface.



6.1.3 **Limit on Bank Height**

A commonly encountered retaining condition is one in which the retaining wall has a slope behind the wall that surcharges the wall, but the slope behind only extends for a relatively short distance above the wall. This condition is depicted in the diagram below. It should be appreciated that standard formula for determining the active earth pressure on a retaining wall for the case of a slope behind the wall, are based on the assumption that the slope behind the wall is effectively infinite. Thus in cases where the slope behind the wall is relatively short, the active earth pressure determined by the formula will be unrealistically high. In order to compute a more realistic destabilising force on the wall, one would have to resort to a wedge analysis. There are some computer programmes that can carry out such wedge analyses.



.

6.1.3 Tiered CRB retaining walls

In the case of tiered CRB retaining walls, it should be appreciated that unless as is shown in the Figure below, the foundations of the upper wall(s) are below the line of the natural angle of repose of the retained soil as measured from the heel of the lowermost wall, such walls will surcharge the lower wall(s).