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Rigid Wall Retaining Bottom Ash Backfill with Geogrid Reinforcement

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Introduction

In the concept of reinforced earth, the soil is reinforced by the elements, which can take tension. These reinforcing elements may be in different forms, e.g., metal sheets, strips, nets, mats, synthetic fabrics or fibre reinforced plastics, etc. Their incorporation in the soil mass is aimed at either reducing or suppressing the tensile strain, which might develop under gravity and boundary forces. The qualities of reinforced earth are its flexibility, which enable it to be used on poor foundation soils, quickness and simplicity in construction and finally low cost.

Most popular use of this technology has been made in retaining wall construction. The other advantage of this technology is that there is no restriction on height of wall. There can be two ways in which the concept of earth reinforcement can be made use of in the construction of retaining walls. These are (1) reinforced earth wall and (2) wall with reinforced backfill. The reinforced earth walls are suitable for the places with poor sub-soil conditions. These walls require sufficient space for construction as the width of wall is determined by the length of reinforcement used. Thus, there may be situations where construction of these walls is not feasible. In such situations, wall with reinforced backfill may prove to be an ideal solution and that has been attempted in the present study.

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Wall with Reinforced Backfill

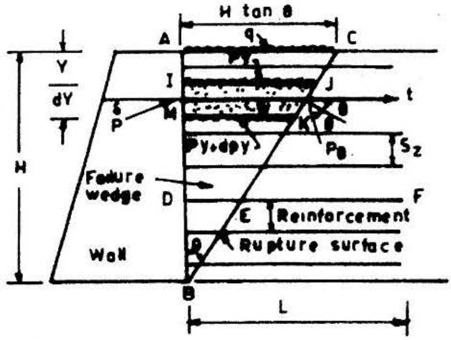
Since the invention of reinforced earth (Vidal, 1966), intensive world-wide research is in progress with particular emphasis on the performance of model and full-scale retaining walls. In wall with reinforced backfill, the lateral pressure on wall is reduced by reinforcing the backfill with unattached horizontal strips or sheets. Broms (1977) was the first to report about the wall with backfill reinforced with unattached continuous fabric reinforcement. According to his findings, (a) sufficient anchor zone which is capable of transferring a force more than allowable tension in the fabric, is needed just behind the wall elements for enabling the reinforcement thus provided to behave as attached reinforcement and (b) lateral earth pressure at any distance away from the wall face could also be computed.

Hausmann and Lee (1978) conducted small-scale model tests to investigate the behaviour of rigid walls with reinforced backfill to establish effectiveness of unattached reinforcement in the fill.

Talwar (1981) developed the analysis for computation of lateral earth pressure in rigid retaining walls having cohesionless backfill reinforced with unattached reinforcing strips. Expressions of resultant earth pressure and its point of application had been derived in terms of soil properties and characteristics and distribution of reinforcement. Results were presented in the form of non-dimensional charts, which indicated significant reduction in earth pressure with the increase in length of the reinforcing strips and decrease in their horizontal and vertical spacing. Theoretical results have been substantiated with carefully conducted model test data. Garg (1988) extended this work considering uniformly distributed surcharge load on the backfill. Garg et al. (1997) also developed a concept of economical placement of the reinforcement for vertical walls. Khan (1991) considered retaining wall with inclined back in the study and presented non-dimensional charts for design of wall with uniformly distributed surcharge load. Analytical work was supported by model tests also. Line load surcharge was also considered in the study and empirical relations were developed for determination of lateral earth pressure and moments on the retaining wall. The soil used by Garg (1988) and Khan (1991) in the model tests was poorly graded dry sand (SP).

Design and Construction of a Prototype Wall with Reinforced Backfill

Garg (1988) developed analysis for a retaining wall with cohesionless backfill reinforced with strips. In the analysis static equilibrium of a horizontal element of soil under the action of various intensities of forces acting on it, within a Coulomb's failure wedge, has been studied (Fig.1). The shorter portion of the reinforcing strip, which moves relative to the failure plane, provides the



H = Height of wall
Y = Distance along wall from top
dy = Thickness of an element of failure wedge
P = Active earth pressure intensity
P_y = Pressure acting on an element of soil in vertical direction
P_θ = Intensity of reaction on failure surface
$(P_y + dp_y)$ = Uniform reaction intensity acting upward on km
q = Intensity of surcharge loading
S_z = Vertical spacing of reinforcing elements
t = Uniform distributed tensile stress
W = Weight of element of soil
δ = Angle of wall friction
θ = Wedge angle with vertical
ϕ = Angle of internal friction

FIGURE 1 : Intensities of Forces acting on an Element of Soil within Coulomb's Failure Wedge

frictional resistance and is therefore termed as effective length of reinforcement. Effect of reinforcement in the analysis has been considered in terms of non-dimensional parameters, viz., "D_p" (spacing coefficient) and "L/H", where D_p is expressed as a ratio of the product of width (w) of reinforcement, coefficient of soil-reinforcement friction (f^*) and the height (H) of wall, to the product of horizontal (S_x) and vertical (S_z) spacing of reinforcement strips; i.e. DP = $wf^*H/(S_xS_z)$, "L" denotes the length of reinforcement.

Analytical results provided by Saran et al. (1992), in the form of design charts for ϕ = 30°, 35° and 40°, D_p = 0.2, 0.5, 1.0, 1.5 and 2.0 and L/H = 0.2 to 1, have been used in the analysis. One such typical design chart for ϕ = 40° is provided in Fig.2. The values of non-dimensional pressure coefficients K_γ and K_q reduce with an increase in L/H ratio upto about 0.6 and thereafter these are almost constant. These parameters also reduce with an increase in D_p upto about D_p = 1.0 beyond which the reduction is insignificant. The resultant lateral earth pressure (P) consists of (i) lateral earth pressure due to backfill earth (P_γ) and (ii) lateral earth pressure due to surcharge load (P_q), i.e. $P = P_\gamma + P_q$.

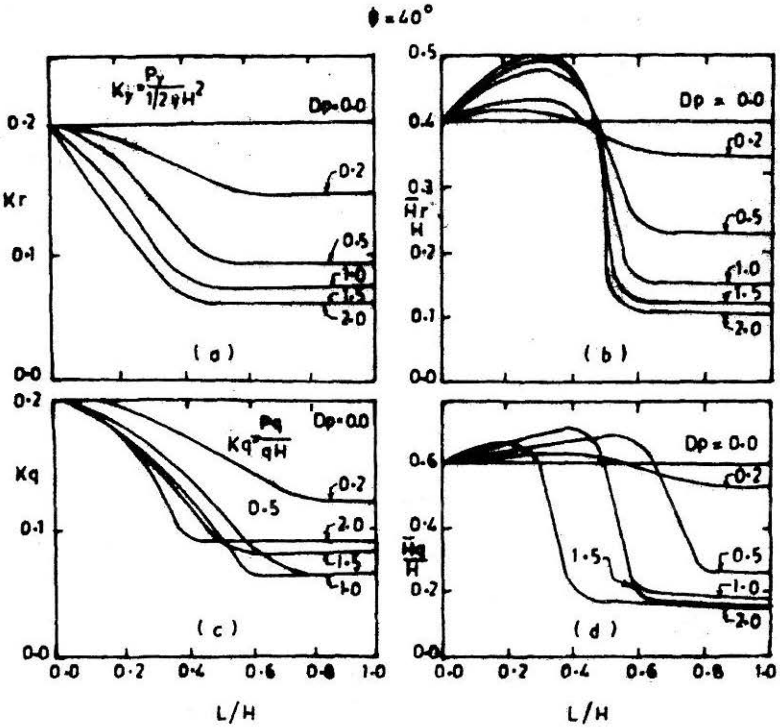


FIGURE 2 : Nondimensional Charts for Resultant Pressure and Height of Point of Application (i) a and b due to Backfill (ii) c and d due to Surcharge Loading (ϕ = 40°)

Using the above analytical studies, a retaining wall of 3.5 m height and 10 m length retaining cohesionless backfill (bottom ash) reinforced with geogrid had been designed and constructed.

Site Selection

A site was selected on a state highway in Sunderpur village, at about 42 kms from Roorkee on Roorkee-Dehradun road. At the site, a seasonal river flows along the main road (Fig.3). The river has an acute bend near the proposed retaining wall site (on its upstream side). Because of this bend, the river erodes the road embankment particularly when it runs full with water. Because of road cutting in every rainy season, this spot had rather become a death trap for vehicles like bicycles, cattle carts, two wheelers etc., which generally move on to one side of road. The state highway authorities allotted this problematic location to try the new technology. Therefore the site was selected.

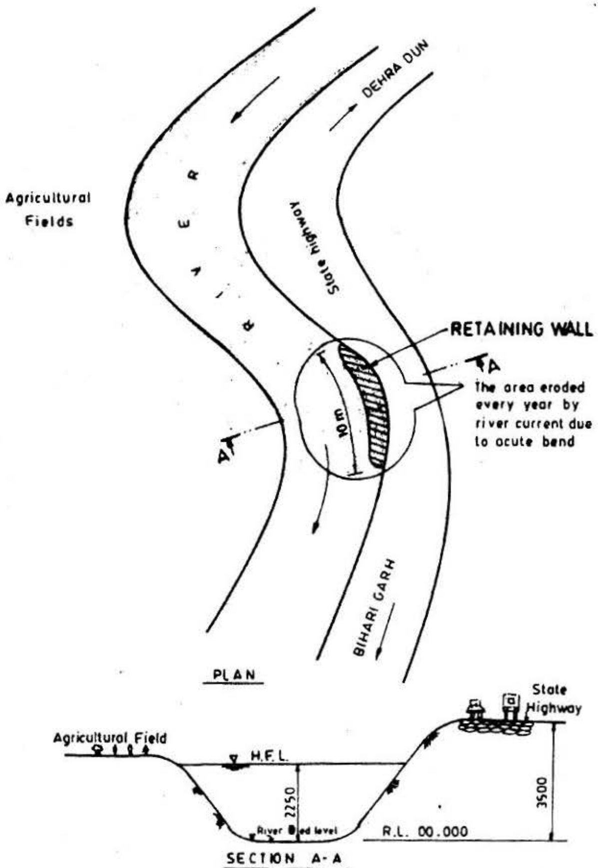


FIGURE 3 : Site Location

Construction and Backfill Materials

It was decided to construct the gravity retaining wall in random rubble masonry with 1:4 cement mortar mix. Council of Science and Technology, U.P. (UPCST) has provided financial assistance for the study with a view to explore the potential of using flyash / bottom ash, a waste material of thermal power stations and paper mills, as backfill material of the retaining walls in general. Therefore it was decided to use bottom ash as backfill of the wall.

Bottom Ash

Bottom ash is a waste material from thermal power stations and also paper mills. Bottom ash production is around 20% to 25% of the total ash produced from thermal power stations. The bottom ash is coarser to flyash.

Therefore handling of bottom ash is easier than flyash. It has no plasticity. The maximum and minimum densities of bottom ash are found to range from 11 kN/m^3 to 18.6 kN/m^3 and from 8 to 14.6 kN/m^3 respectively. The bottom ash for the study was collected from Star Paper Mills, Saharanpur, about 35 kms from the site. The engineering properties of bottom ash used at site were as follows (Table 1)

Reinforcing Material

The geogrid CE 121 (Netlon-India) has been used as the reinforcing material in the backfill. The advantage of using geogrid is that it is non-biodegradable and free from corrosion.

Design of Wall

Data

a)	Height (H)	= 3.50 m
b)	UDL	= 30 kN/m^2
c)	Density of backfill	= 16 kN/m^3
d)	Angle of internal friction (ϕ)	= 40°
e)	Angle of wall friction (δ)	= 25°
f)	Cohesion (c)	= 0
g)	Coefficient of sliding (μ) for Foundation	= 0.40
h)	Soil bearing capacity	= 100 kN/m^2
i)	Unit wt. of masonry	= 20 kN//m^3

Table 1 : Properties of Bottom Ash used in Study

Grain Size Analysis				Atterberg's Limits		Specific Gravity	g_{dry} (kN/m^3)	Shear Parameters*	
Gravel (%)	Sand (%)	Silt (%)	Clay (%)	L.L.	P.L.			c	ϕ
—	79.50	20.17	0.33	NP$^+$	NP$^+$	2.25	16	0	40.50

* Determined from large size direct shear box (300 mm × 300 mm × 200 mm) test with geogrid sandwiched between bottom ash.

\+ NP — Non-plastic

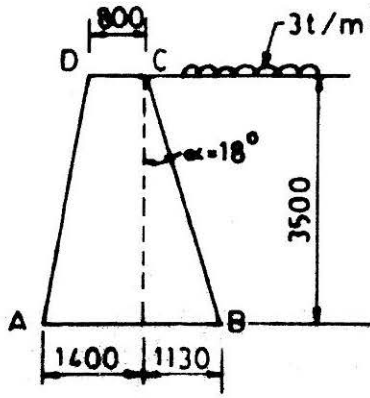


FIGURE 4 : Cross Section of Retaining Wall with Unreinforced Backfill

Masonry Gravity Wall with unreinforced backfill

The trial top width of retaining wall was taken as 0.80 m, and bottom width was taken as 2.53 m.

For analysis purpose, 1 m length of wall has been considered.

The section of wall was checked for its stability and yielded a factor of safety against sliding as 1.97, factor of safety against overturning as 4.75 and maximum base pressure equal to 63 kN/m^2. The section of the wall is shown vide Fig.4.

Masonry Gravity Wall with reinforced backfill

Let us assume wall section with following dimensions

Top width = 0.5 m

Base width = 2.20 m

Assume D_p = 0.5 and L/H = 0.4

For reinforced case (from Fig.2) for ϕ = 40°

K_γ = 0.112

K_q = 0.142

H_γ/H = 0.41, thus, H_γ = 1.22

H_q/H = 0.68, thus, H_q = 2.38

Use CE-121 geogrid as reinforcing material, the safe tensile strength of which is taken as 5 kN/m. The width of reinforcing strip is taken same as the length of wall.

The sectional layout and location of reinforcing strips along wall is shown in Fig. 5A.

The permissible vertical spacings (S_z) of geogrid are provided in Fig. 5B.

This section of wall yields a factor of safety against sliding as 1.66, factor of safety against overturning as 6.30 and maximum base pressure as 21 kN/m^2.

Detail design calculations are provided in Appendix.

Wall Construction

The foundation of wall was in riverbed. The height of the wall at site was 3.50 m above the ground level, with base width as 2.20 m and top width as 0.50 m (Fig.5A). The foundation depth was taken as 0.75 m. This wall was designed for a uniformly distributed load of about 30 kN/sqm which is the most expected load on a state highway. The length of wall was 10 m. The construction of the wall was taken up in August 1997 and completed in January 1998 with a break of about 3 months in the construction due to some official procedures. The backfill (bottom ash) was filled up in full length of wall and in width of 2.20 m. The geogrid strips were used as the reinforcing material. The bottom ash was laid layer by layer and it was

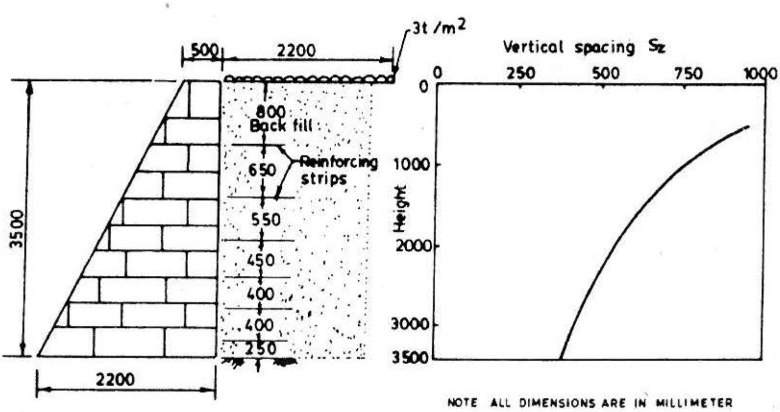


FIGURE 5 : (a) Sectional Layout and Location of Reinforcing Strips along Retaining Wall; (b) Permissible Vertical Spacing (S_z) of Geogrid

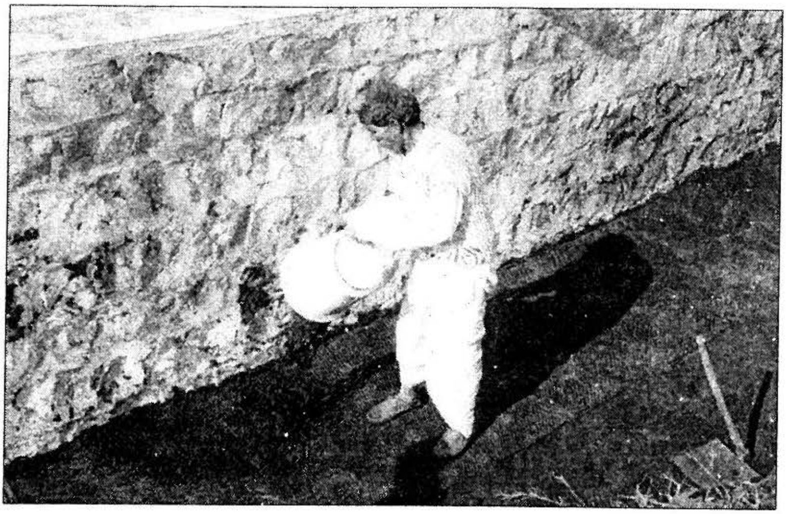


FIGURE 6 : Laying of Geogrid Reinforcement and Backfilling of Bottom Ash

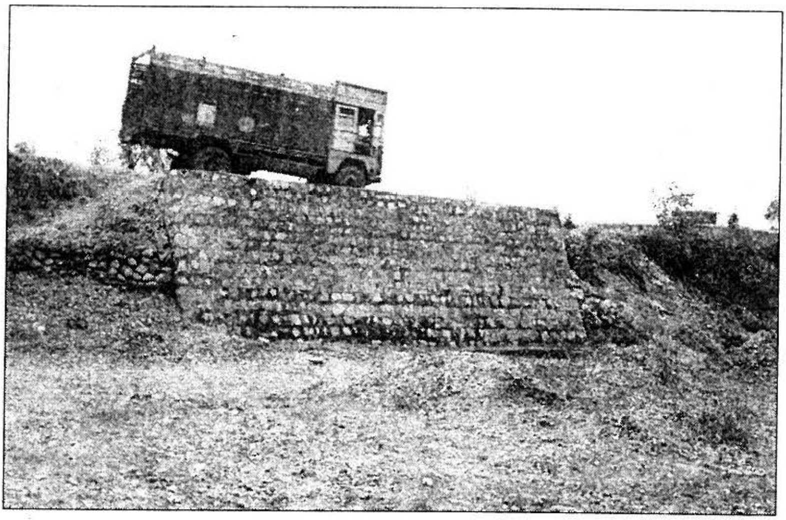


FIGURE 7 : A View of Completed Wall with Reinforced Backfill

compacted manually. At different depths (as per design) the geogrid strips were laid and straightened perfectly and thereafter another layer of bottom ash was laid over the geogrid as shown in Fig.6. Thus, the backfill was completed upto the top of the wall. Fig.7 shows the view of the completed wall.

Cost Comparison of Wall

The cost comparison of wall is given in Table 2. It is seen that wall with reinforced backfill is about 20 percent economical as compared to conventional wall. Experience has shown that the cost economics is dependent on the height of wall, more is the height of wall, higher will be the level of savings.

Conclusions

The construction of wall with geogrid-reinforced backfill has shown that there is a considerable saving in cost, space and construction time. Further, bottom ash which is presently a waste material, (and is also available almost free of cost) can be used as the backfill material. Thus, on the sites, which are close to thermal power stations or paper mills, bottom ash can successfully be used as the backfill material.

The wall is performing satisfactorily for almost last two years.

Table 2 : Comparative Analysis of Cost of Both kinds of Wall (length of wall – 10 m)

S. No.	Item	Cost per Unit	Wall with unreinforced backfill		Wall with Reinforced backfill	
			Qty.	Cost (Rs.)	Qty	Cost (Rs.)
1.	E.W. in Excavation	32	36 m^3	1152.00	32 m^3	1024.00
2.	PCC 1:2:4	1830	3.20 m^3	5856.00	2.9 m^3	5307.00
3.	RCR Masonry in foundation	1475	30 m^3	44250.00	27 m^3	39825.00
4.	RCR masonry in structure	1575	58.5 m^3	92137.50	47.25 m^3	40000.00
5.	Backfilling of soil	14	9 m^3	126.00	8 m3	112.00
6.	Reinforcement geogrid	150	—	—	95 m'	14250.00
7.	Transportation of material e.g. rubble etc. from nearest point	100	90 m^3	9000.00	75 m^3	7500.00
8.	Transport of sand from nearest quarry and filling at site	50	90 m^3	4500.00	90 m^3	4500.00
	Total Cost			157021.50		121343.00
	Misc. and unforeseen @ 3%			4710.60		4408.00
	Contingency @ 3 %			4710.60		4408.00
	Grand Total			166442.70		130150.00

Note : Rates are as per U.P. schedule of rates (1997)

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Appendix

Design Analysis

Masonry Gravity Wall with Unreinforced Backfill

$K_a = 0.196$ (for $\phi = 40°$) (Garg, 1988)

Resultant active pressure, $P = P_\gamma + P_q$

$$P_a = \frac{1}{2} K_a \times \gamma H^2 = \frac{1}{2} \times 0.196 \times 16 \times (3.5)^2 = 19.2 \text{ kN/m}$$

$$P_q = K_a \cdot q\, H = 0.196 \times 30 \times 3.5 = 20.6 \text{ kN/m}$$

$$P = P_a + P_q = 19.2 + 20.6 = 39.8 \text{ kN/m}$$

$$P_H = P_{aH} + P_{qH} = P\cos(\delta + \alpha)$$

Here, $\delta = 25°$; Wall angle, $\alpha = 18°$

Thus, $$P_H = 19.2\cos(25 + 18) + 20.6\cos(25 + 18)$$
$$= 29.1 \text{ kN/m}$$

$$P_v = P_{av} + P_{qv} = P\sin(\delta + \alpha)$$
$$= 27.1 \text{ kN/m}$$

The section of the wall is shown in Fig. 4

Thus, Weight of wall (W) $= \left\{(0.8 + 1.439.5) \times \frac{3.5}{2} \times 20\right\} + 39.5$

$$= 77.0 + 39.5 = 116.5 \text{ kN/m}$$

FOS against sliding $= \dfrac{\left[W + p\sin(\delta + \alpha)\right]\mu}{p\cos(\delta + \alpha)}$

$$\text{Thus, FOS against sliding} = \frac{[116.5 + 27.1] \times 0.4}{29.1}$$

$$= 1.97 \text{ (O.K.)}$$

$$\text{Overturning moment } (M_{OA}) \text{ about A} = 14.0 \times 1.16 + 15.0 \times 1.75$$

$$= 42.5 \text{ kN - m / m}$$

Moment of resistance (M_{RA}) about A

$$= \left\{77.0 \times \frac{2}{3} \times 1.4\right\} + \left\{39.5 \times \left(1.4 + \frac{1}{3} \times 1.3\right)\right\}$$

$$+ \{10.8 \times 2.53 + 11.6 \times 2.53\}$$

$$= 200.0 \text{ kN - m / m}$$

$$\text{FOS against overturning} = \frac{200.0}{42.5} = 4.75 \quad \text{(O.K.)}$$

Eccentricity (e) of vertical load

$$= \frac{2.53}{2} - \frac{200.0 - 42.5}{(W + P_v)}$$

$$= \frac{2.53}{2} - \frac{157.9}{143.6} = 0.16 \text{ m} < \frac{\text{Base Width}}{6} \quad \text{(O.K.)}$$

$$\text{Maximum base pressure } (p_{max}) = \frac{W}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{116.5 \times 1.37}{2.53} = 63.0 \text{ kN / m}^2$$

$$\text{Minimum base pressure } (p_{min}) = \frac{W}{b}\left(1 - \frac{6e}{b}\right) = 28.5 \text{ kN / m}^2$$

Masonry Gravity Wall with Reinforced Backtill

For reinforced case (from Fig.2) for $\phi = 40^{\circ}$

$$K_{\gamma} = 0.112$$

$$K_{q} = 0.142$$

$$H_{\gamma}/H = 0.41, \text{ thus, } H_{\gamma} = 1.22$$

$$H_{q}/H = 0.68, \text{ thus, } H_{q} = 2.38$$

Use CE-121 geogrid as reinforcing material, the safe tensile strength of which is taken as 5 kN/m. The width of reinforcing strip is taken same as the length of wall.

Active earth pressure, P_a

$$= \frac{1}{2} K_{\gamma} \cdot \gamma H^2 + K_q \cdot q H$$

$$= 0.5 \times 0.112 \times 16 \times (3.5)^2 + 0.142 \times 30 \times 3.5$$

$$= 25.8 \text{ kN/m}$$

$$P_a \cos\delta = 10.9 \cos(30) + 14.9 \cos(30)$$

$$= 22.3 \text{ kN/m}$$

$$P_a \sin\delta = 10.9 \sin(30) + 14.9 \sin(30)$$

$$= 12.8 \text{ kN/m}$$

$$\text{Wt. of wall} = (0.50 + 2.20) \times \frac{3.5}{2} \times 20 = 94.4 \text{ kN/m}$$

$$\text{F.O.S. against sliding} = \frac{(94.4 + 12.8)\,0.4}{25.8} = 1.66 \text{ (O.K.)}$$

Resisting Moment about A(M_{RA})

$$= \left\{59.5 \times \frac{2}{3} \times 1.7\right\} + \left\{35 \times (1.7 + 0.25)\right\}$$

$$+ \left\{5.45 \times 2.2\right\} + \left\{7.45 \times 2.2\right\}$$

$$= 67.4 + 68.2 + 28.3 = 163.9 \text{ kN-m/m}$$

Overturning moment about A $= 9.4 \times \frac{3.5}{3} + 12.9 \times \frac{3.5}{3}$

$= 26.0 \text{ kN - m / m}$

FOS against overturning $= \frac{16.39}{2.60} = 6.30$

Net moment $163.9 - 26.0 = 137.9 \text{ t - m / m}$

Eccentricity of vertical load $= \frac{2.20}{2} - \frac{137.9}{(94.4 + 12.8)} = -0.18 \text{ m}$

$$p_{min} = \frac{W}{b}\left(1 + \frac{6e}{b}\right) = 21.0 \text{ kN / m}^2$$

$$p_{max} = \frac{W}{b}\left(1 - \frac{6 \times 0.18}{2.20}\right) = 63.9 \text{ kN / m}^2 \qquad \text{(O.K.)}$$

For working out the vertical spacing (S_z) of the reinforcement, the following equation is used

$$T = \left[\gamma H\left(K_a - K_\gamma\right) + q\left(K_a - K_\gamma\right)\right] S_z$$

where, T = Permissible tensile strength of geogrid

$= 5 \text{ kN / m}$

Here, K_a (for $\phi = 40°$) $= 0.21$

$g = 16 \text{ kN/m}^3$

Substituting the values in above equation, a curve can be drawn as shown in Fig.5B for determining the value of S_z.

The layout of reinforcement is shown in Fig. 5A.