DIRECTORATE GENERAL BORDER ROADS

TECHNICAL INSTRUCTION NO 5
(REVISION – 2022)

RIGID RETAINING STRUCTURES
(GRAVITY TYPE)

अजय भट्ट
AJAY BHATT

सत्यमेव जयते

आज़ादी का अमृत महोत्सव

रक्षा एवं
पर्यटन राज्य मंत्री
भारत सरकार
Minister of State for
Defence and Tourism
Government of India

MESSAGE

The revised DGBR Technical Instructions are unique as they combine technical content and codes with practical advice on practice of specific subjects. These Technical Instructions have definite data/content which explicitly bring out tools, process and methodology to be followed for various road construction and infrastructure development associated activities.

The literature is a repository of technical and ground experience amassed by the BRO, working over six decades in inhospitable terrain with harsh climatic conditions as well as latest technical advancements in the field of road communication infrastructure development. I personally find these technical instructors informative, exhaustive and practical in approach. These will mitigate the need for ground executers to refer various books/codes where working on various aspects of road construction and will go a long way in assisting the coming generations of BRO executives.

Jai Hind !

Ajay Bhatt
(Ajay Bhatt)

Dated : 20 Dec, 2021
New Delhi

डा. अजय कुमार
रक्षा सचिव
Dr. Ajay Kumar
Defence Secretary

भारत सरकार
रक्षा मंत्रालय
Government of India
Ministry of Defence

FOREWORD

1. I am pleased to note that the Border Roads Organisation (BRO) has revised its twenty seven Technical Instruction, after a gap of 10 years having updated technical content and IRC codes. These Technical Instructions will positively prove to be very useful and ready reckoner for the BRO ground executives while steering them towards the correct methodology and processes to be followed for diverse road construction and associated activities.

2. Since the past six decades, BRO has been developing road infrastructure in the remote regions of the nation. It has contributed immensely in nation building and ushered in prosperity and development in the border areas. A robust mechanism to channelize the road construction activities is an essential planning process and therefore, the revised Technical Instructions will facilitate in dissemination and application of engineering knowledge with updated codes and provisions, to enable the executives to construct roads with the latest specifications as laid down by IRC.

3. I congratulate Team BRO for their stupendous efforts put in for revision of the twenty seven Technical Instructions which will further strengthen their technical proficiency.

New Delhi, (Ajay Kumar)

March 3, 2022.

श्रमेण सर्वम साध्यम्

ले. जनरल राजीव चौधरी, वी एस एम
महानिदेशक सीमा सड़क एवं कर्नल कमांडेंट
सीमा सड़क संगठन

Lt Gen Rajeev Chaudhry, VSM
Director General Border Roads &
Colonel Commandant
Border Roads Organisation

FOREWORD

1. DGBR Technical instruction Number 5 (Revision 2022) on "Rigid Retaining Structures (Gravity Type)" deals with specifications, planning, design, construction and maintenance of gravity type retaining walls.

2. The Technical Instruction Number 5 has now been updated and revised incorporating the current standards and specifications laid down by Ministry of Road Transport & Highways (MoRT&H), Indian Road Congress and Bureau of Indian Standards (BIS). The specific requirement of roads under BRO, have also been kept in view while formulating the Technical Instruction.

3. For proper planning and design of gravity retaining walls and its effective functioning, the correct assessments of ground requirements and design parameters as well as selection of site and appropriate material for incorporating in the various component layers are important. The guidelines given in this Technical Instruction will facilitate ground executives to ensure proper methodology and precautions while constructing and maintaining the retaining walls in an economical way.

4. This Technical Instruction may be supplemented by the Standards and specifications of the MoRT&H, IRC and BIS for more elaborate understanding of the subject.

5. This Technical Instruction will come into force with immediate effect.

Station: New Delhi

Dated: Mar 2022

(**Rajeev Chaudhry**)
Lt Gen
Director General Border Roads

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TECHNICAL INSTRUCTION NO 5
RIGID RETAINING STRUCTURES
(GRAVITY TYPE)

PART I: GENERAL INSTRUCTION

1. INTRODUCTION

Retaining wall is a structure that are designed and constructed to withstand lateral pressure of soil or hold back soil materials. The lateral pressure could be also due to earth filling, liquid pressure, sand, and other granular materials behind the retaining wall structure. . Retaining walls also resist the effect of loads of traffic over road of structure or building etc. these can be Grouped in following two categories :-

(a) **Rigid Retaining Structures** – These structures involve permanent construction.

(b) **Flexible Retaining structure** –These are constructed on temporary basis with sheet piles, wooden piles and wire –crated stone masonry.

2. SCOPE

2.1 This technical instruction is being issued for design of rigid retaining structures like gravity type/RCC Cantilever type retaining structures. These type of retaining structure is used to support earth or other type of materials behind them, which would otherwise not stay in that position. Other type of rigid retaining structure, such as brick retaining wall, revetment walls etc. are not covered in this technical instruction. Flexible retaining structure such as gabion walls etc. are not covered in this technical instruction. These structures will be covered under a separate technical instruction.

2.2 All designs of RC retaining walls shown in the TI may be referred as sample as seismic conditions not taken into account for design. It needs to be considered as per applicable zone and as per IRC/ IS provisions for design.

3. TYPES OF RIGID RETAINING STRUCTURES

3.1 Gravity type rigid retaining structures in hills / plains are constructed for different situation/purpose. There are generally of following types. Each of these types, their purpose and mode of construction are enumerated in succeeding Para 3.2 to 3.7

(a) Retaining walls.
(b) Breast walls.
(c) Toe Walls.
(d) Return walls.
(e) Revetment walls.
(f) Check walls.

3.2 **Retaining Walls.** Retaining walls are built to resist the earth pressure of filling and the traffic loads of the road. These are commonly used in hill roads when the road goes in embankment or partly cutting and partly filling. The retaining walls are also used extensively to develop sites for building complexes. Retaining walls resist the combination of pressures.

3.3 **Breast Walls.** Breast walls are normally stone masonry walls provided to protect the slopes of cutting in natural ground from the action of weather and cut slope failure but not from impact of snow avalanches. A toe wall cannot be used to stabilize an unstable slope. This type of retaining structure is meant to protect a freshly cut or old cut surface of a natural hill face. Breast wall prevent slipping of hill side slides under action of varying weather and rain water flowing over hill slopes. Impact of snow avalanches, landslides and surcharge are not considered in the design of breast walls. Height of breast walls shall not exceed 3.00 mtrs. Breast walls are not required to be constructed where back mass comprises of rock or table strata / deposit of soil mass.

3.4 **Toe Walls.** These types of retaining structure are constructed at the toe of hill slope in soil zones to check the erosion of soil at the toe of hills.

3.5 **Return Walls**. These types of retaining structure are constructed parallel to the center line of a road or in bridge.

3.6 **Revetment Walls.** These types of retaining structure are constructed for protection of embankment slope. These structures are not meant to withstand lateral or vertical pressure. These are constructed with various thickness (minimum 30cm thick) comprising of stone pitching.

3.7 **Check Walls.** These are retaining structures constructed at various elevations below and above the road longitudinally/parallel to the center line of the road, to check erosion of soil at various elevations and help in stabilization of the hill slope.

4. GRAVITY TYPE RIGID RETAINING WALLS

4.1 This technical instruction is issued as general guideline for design of stone masonry retaining walls and RCC Cantilever retaining walls.

4.2 Details of Gravity masonry retaining structure is as under.

4.2.1 **Ordinary Retaining Wall.** At places, which are not impinged on by floods or where high degree of strength is not necessary, ordinary retaining walls of height up to 6.0 m (20 feet) will generally be constructed of dry stone masonry. These ordinary retaining wall can be placed in two categories such as-

(a) Reatining walls upto 3.0 m height shall be constructed in Random Rubble (RR) dry masonry.

(b) Retaining walls from 3 m to 6 m shall be in banded RR masonry with 1:6 cement mortars. The banded masonry in cement mortar shall be provided length wise as well as breadth wise for full length of wall to break the joints and to cover-ups the short comings in the execution of the wall.

4.2.2 **Masonry Retaining wall in Cement Mortar**. It will be constructed with stone or random rubble with cement mortar over the well laid foundation comprising of Plain Cement Concrete (PCC).

(a) Where high degree of strength is necessary in all cases of wall with height less than 6.0 m (20 feet).

(b) All retaining walls with height more than 6.0 m (20 feet).

5. SPECIFICATIONS

5.1 Based on the requirement of strength expected from a retaining structure and considering its service / utility, the masonry in the retaining structure (gravity type) may be constructed as per the specifications specified at para 5.2 & 5.3.

5.2 Ordinary retaining walls and breast walls, which are not impinged upon by floods and are not required to hold water, will have the following specifications:-

(a) **Dry stone masonry walls.** Masonry comprising rough dry stone over PCC foundation up to 3 m height.

(b) **Banded dry stone masonry wall.** Masonry comprising rough dry stone, along with masonry strengthening bands (horizontally and vertically) of stones in cement mortar, over PCC foundation of heights 3.0 m to 6.0 m.

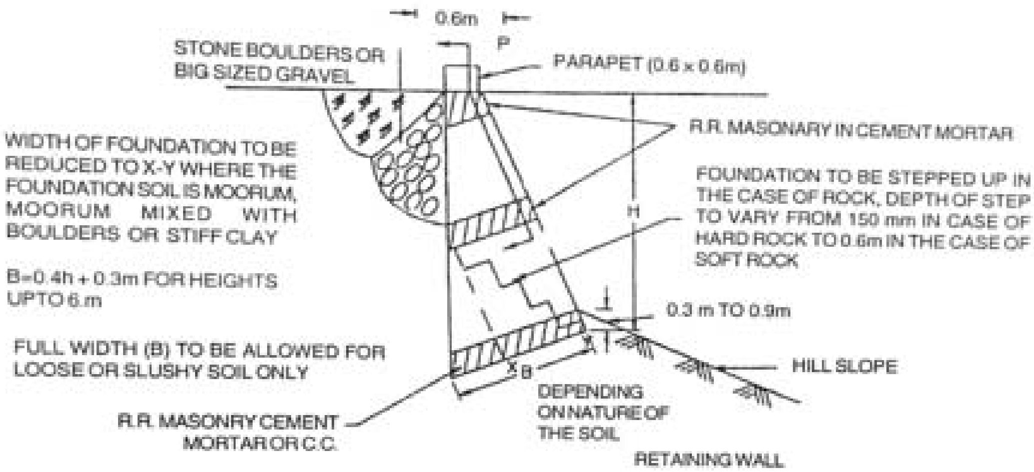


Fig. 1(a): Cross Section of Banded wall

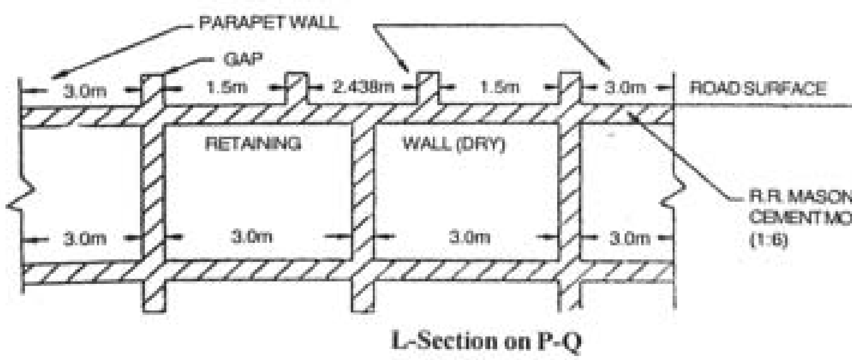


Fig. 1(b): Stones masonry in cement mortar over PCC foundation

5.3 In case site condition is such that back earth pressure is more due to pour water pressure or saturated state and surcharge due to heavy load movement on road, the complete retaining wall may be constructed in random rubble stone masonry in cement mortar 1:6 even if the height of structure is less than 3.0 m.

PART II: DESIGN OF STRUCTURES

6. DESIGN OF RIGID RETAINING STRUCTURES (GRAVITY TYPE)

6.1 Rigid retaining Structures should be properly designed considering ground data with particular reference to areas comprising weak soils, unstable strata, surcharges, fluctuations in moisture content and fluid condition etc.

7. DIMENSIONS OF RETAINING STRUCTURES IN HILLS

7.1 It is generally not possible to design each and every wall along the entire length of road. Standard design of Banded Dry Stone walls, as given in the fig. 1 is adopted for walls with height up to 6.00 m (20ft.) in low hazard zone, Broad design details are given as under:

(a) The base must be substantial and must be capable of distributing the pressure over foundation. For average dry conditions width of foundation footing be taken as 0.5 H + 0.30 m (where H is the total height of wall in m) and base width of retaining wall may be taken as 0.4 H + 0.30 m.

(b) For near fluid condition, these dimensions should be increased in direct proportion to the horizontal pressure.

(c) The front batter is given as 1 in 3 up to 4 m height and thereafter made flatter and the back face is kept vertical.

(d) Top thickness of retaining wall is generally kept as 0.6 mtr.

(e) Minimum depth of foundation of wall below the natural hill slope (or hill) is generally kept as 0.1 H + 0.3 m (where H is the height of wall in m) Fig. 2(a).

(f) Projection of any footing course (towards back fill) should not exceed half the depth of course.

(g) A trial section first chosen and analysed. If the stability checks yields unsatisfactory result, the section is changed and rechecked.

(h) For heights more than 6 m, the face slope may be given a batter from 1:3 to 1:2.5 or 1:2 max for portion deeper than 6.0 m or 8.0 m respectively below the road level depending upon ground conditions.

(j) Base slope should not be flatter than 1:6.

(k) In firm rock zone face slope can be made steeper up to 1:6.

(l) It is always advantageous to provide 0.3 m projection at the toe of retaining wall so as to reduce pressure intensity and also at every 3 m depth an offset or projection or projection of 0.3 m is desirable. (See Fig .14.)

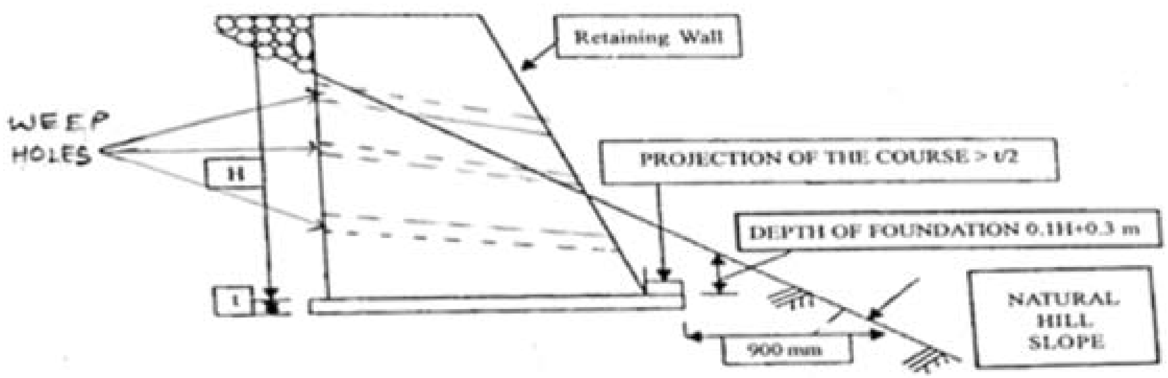


Fig. 2(a)

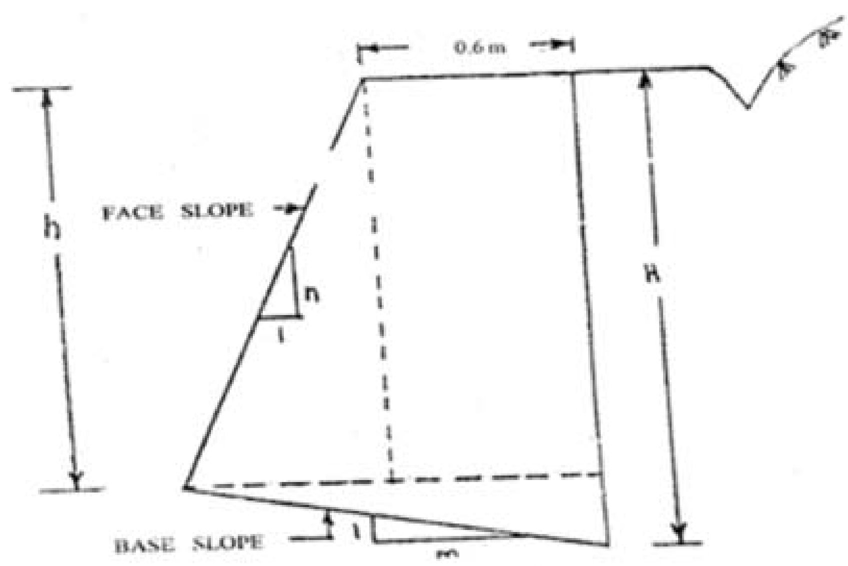


Fig.2(b)

Fig. 2: Various dimensions of retaining walls

8. DIMENSIONS OF RETAINING WALLS IN VARIOUS ROCK ZONES

Table - 1

Height of R/Walls	TYPE OF ROCK ZONES					
	Soil, Soil mixed boulder, zones fractured rock of beds dipping khud or valley side (ø = 35°)			Conglomerate homogeneous, very hard, hard, medium rock and stables fracture rock zones of stable nature (ø = 45°)		
	walls face slope	Toe Wall		Front Face Slope	Toe Wall	
		B	T		B	T
Upto 4 m	1:4	0.3 m	0.45 m	1) Vertical with HP filling 2) 1:6 for 1.5 m wide filling with hand packed stones 3) 1:4 for wider stone packed fillingas above	---	---
Above 4 m to 6 m	1:4	0.6	0.6	1) Vertical without filling 2) 1:6 for 1m wide hand packed stones filling as above 3) 1:4 for wider stone packed filling as above	0.6 m 0.6 m	0.6 m 0.6 m
Above 6 m to 8 m	1:3	0.9 m	0.9 m	1:40.6 m	0.6 m	
Above 8 m to 10 m	1:3	1.2 m	1.2 m	1:4 for top 8 m height and1:3 for lower portion	0.6 m	0.6 m

* For B & T See Fig. 14.

8.1 Vertical faced, back sloped retaining walls can be designed on economic cost considerations keeping similar top and base width and base slope. Only face slope of back and front are to be interchanged. (Refer para 12).

8.2 The proposed sections given in Table 1are based on observed behavior of existing old dry masonry retaining walls standing well for several Decades on important roads in the hills Uttrakhand.

9. BEARING CAPACITY

9.1 The allowable bearing capacity shall be Calculated in accordance with IS 6403 on the basis of soil tests data. In case of non –erodible rocks, the bearing capacity shall not exceed one–half the unconfined compression strength of the rock, if the joints are tight. Where the joints are open the bearing capacity of soils shall not exceed one-tenth the unconfined compressive strength of the rock. Bearing capacity for weak and closely jointed rock shall be assessed after visual inspection supplemented as necessary by field or laboratory tests to determine strength and compressibility.

9.2 **Assessed Bearing Capacity.** In the absence of soil test data for preliminary design the values as given in Table 2 may be adopted.

9.3 Bearing capacity of rocks may be determined in accordance with IS – 12070. In case of erodible and weak soil in Foundation (clay, loose soil etc.), **Gabion wall** shall be preferred as it can with stand high differential settlement.

9.4 When earthquake forces on all are also included in forces, the permissible increase in allowable bearing capacity shall be in accordance with clause 3.3 of IS 1893: 1984.

9.5 The value of cohesion ‘C’ and angle of internal friction Φ of soils vary for different back fill and foundation materials. These values shall be determined by experiment. However for preliminary design the value of cohesion and angle of internal friction of soil can be adopted from Table 3.

9.6 **Bearing Capacity of Soil.** As per IS-14458 (Part 2) 1997 safe bearing capacity of different type of soil as given in Table 2, same may be considered for the design of retaining structure.

Table - 2

Type of bearing material	Group Symbol	Consistency of place	Recommended value of safe bearing capacity (Ton per sq. mtr)
Well graded mixture of fine and coarse grained soil. Glacial till, hard pan boulder clay gravel, gravel-sand mixtures, boulder gravel mixture	GW-GC GC, SC GW GP SW SP	Very compact Very compact Medium compact Loose	100 80 60 40
Coarse to medium sand, Sand with little gravel	SW SP	Very compact Medium compact Loose	40 30 30
Fine to medium sand, silt or clayey medium to coarse sand	SW SM SC	Very compact Medium compact Loose	30 25 15
Fine sand, silt or clayey medium to fine sand	SP SM SC	Very Compact Medium to stiff Soft	40 20 5
Homogenous inorganic clay, sand or silty clay	CL, CH	Very stiff to hard Medium to stiff Soft	40 20 5
Inorganic silt, sandy or clayey silt varied stiff-clay-fine sand	ML, MH	Very stiff to hard Medium to stiff Soft	30 15 5

9.7 **Value of Cohesion 'C' and Angle of Internal Friction ϕ of Soils.** Value of cohesion C and angle of internal friction ϕ depend upon type of soil of foundation and backfill materials. These values shall be determined by experiment. However, keeping in view the clause of 4.3 of IS-14458 (Part II) 1997, following value of cohesion 'C' and angle of internal friction ϕ can be considered for preliminary design of walls as given in Table 3.

Table - 3

TYPICAL STRENGTH CHARACTERISTICS OF SOIL

Group Symbol	Cohesion of soil Ton per sq. mtr		Φ' in degree ffective stress envelope	Tan Φ'
	C	C -		
1	2	3	4	5
GW	0	0	>38	>0.79
GP	0	0	>37	>0.74
GM	-	-	>34	>0.87
GC	-	-	>31	>0.60
SW	0	0	38	0.79
SP	0	0	37	0.74
SM	0.50	0.20	34	0.67
SM-SC	0.50	0.15	33	0.66
SC	0.75	0.10	31	0.60
ML	0.70	0.10	32	0.62
ML-CL	0.65	0.20	32	0.67
CL	0.90	0.15	28	0.54
MH	0.75	0.21	25	0.47
CH	1.00	0.10	19	0.35

10. FOUNDATION

10.1 Foundation of retaining structure (retaining walls and Breast walls etc.) must be taken deep enough to reach sound strata (comprising solid material) and must be safe enough from scour, frost and surface water. The depth of foundation below natural ground level should normally be 0.30 m (1.00 ft) plus 1/10 of the height of all subject to minimum 0.60 m (2.00 ft) except in rocky foundation.

10.2 **Steeping of base of walls in rock strata.** In hills the rock below foundation must be cut to a downwards slope i.e., dip of base of wall towards hill side of 3:1 (horizontal: vertical) in various steps along the hill face proves economical in seismic condition. It increases factor of safety against sliding significantly and is economical. However, rock below foundation in plain areas can be cut in level steps. Refer Fig. 2(c).

11. DEPTH OF FOUNDATION

11.1 Foundation must be taken deep enough to reach on solid strata material and be safe from frost action, surface water and scour. Minimum depth of foundation (h) for stability is given by the following equation:-

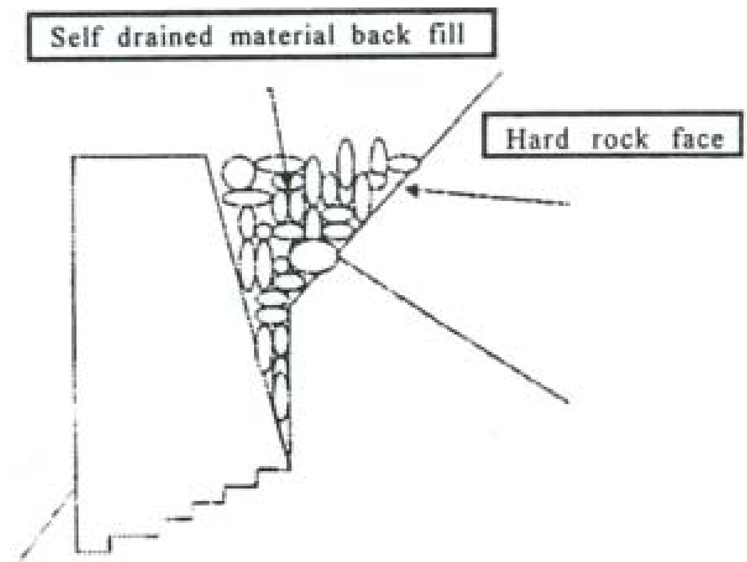


Fig. 2(c) stepping of foundation of wall on Rock Slope

$$h = \frac{w}{Aw}\left[\frac{1-\sin\Phi}{1+\sin\Phi}\right]^2$$

Where W	=	Total weight on foundation
A	=	Area of foundation
w	=	Weight of Unit Volume of Soil
Φ	=	Angle of repose of soil

12. PRESSURE ON FOUNDATION

12.1 The pressure on foundation material at the toe of the wall must not be greater than the safe working stress. By projecting the toe, the center of the base is brought nearer to the resultant and the toe pressure is very much reduced. Projection on the heal increase the maximum toe pressure and should not be given. (See Fig. 13)

The average pressure P on the foundation is given by

$$P = \frac{\text{Total vertical force or load per unit lenght}}{\text{width of head}}$$

$$= \frac{F}{B+T}$$

Where B width of base without toe projection,
T is toe projection beyond face of wall
F is total vertical force (or load) per unit length

13. DESIGN CRITERIA

13.1 The design of a retaining structure consists of two principles parts, first the evaluation of loads and pressures that are active on the structure second the design of the structure should be safe to withstand these loads and pressures.

13.2 Following forces shall be accounted for in the design of the retaining structures:

(a) Self-weight of the retaining structure.

(b) Super imposed loads, and live loads if any

(c) Active Earth pressure acting on the retaining structure

(d) Passive Earth pressure action on the retaining structure

(e) Pore Water pressure due to water table/sub-surface seepage.

(f) Passive Water pressure due to water table on toe side, if any

(g) Seismic forces

(h) Special loads, if any

13.3 The self-weight of the structure, live loads and imposed loads shall be estimated in accordance with IS 875 (Part 1 to 5). In the usual cases live load may be taken between 250 Kg/Sqm. on the top width of the wall.

13.4 The earth pressure and other seismic forces on the retaining structure shall be estimated in accordance with IS 1893:1984. For low traffic volume roads, the walls may not be designed for earthquake forces. However, in case of retaining walls for roads, earth pressure due to surcharge shall be considered.

14. CALCULATION OF EARTH PRESSURE UNDER DIFFERENT CONDITION

A. RANKINE'S METHOD

Case I : When there is no surcharge and C=0

$$Pa = \frac{1}{2} Ka\, Y\, H^2$$

$$Ka = \frac{1-\sin\Phi}{1+\sin\Phi}$$

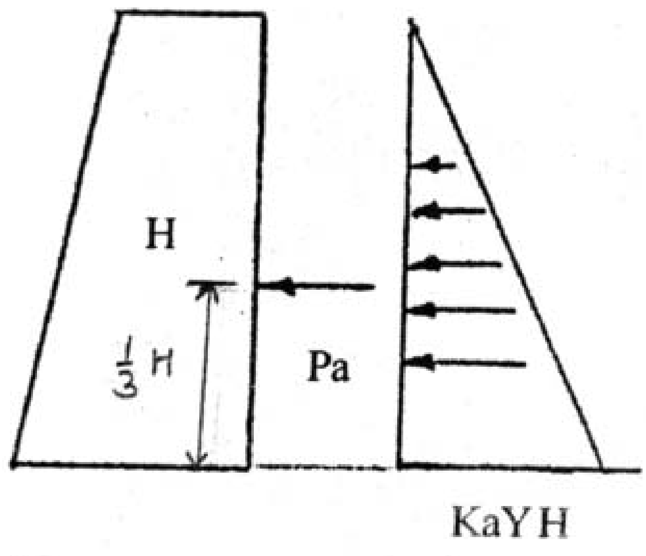


Fig.3

Case II : When there is Surcharge C = 0

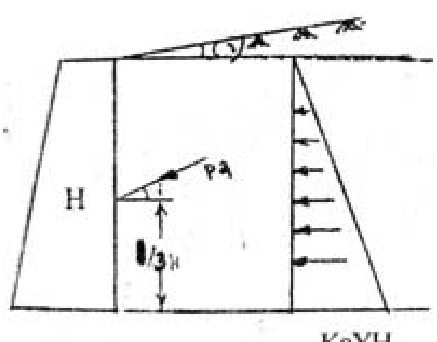


Fig. 4

$$Pa = \frac{1}{2} Ka\, Y\, H^2$$

$$Ka = \frac{\cos i + \cos i - \quad \cos^2 i - \cos^2\phi}{\cos i + \quad \cos^2 i + \cos^2 \phi}$$

Pa = is parallel to inclined surface.

C = Cohesion of soil

i = angle of surcharge

Case III : When soil is submerged and C = 0

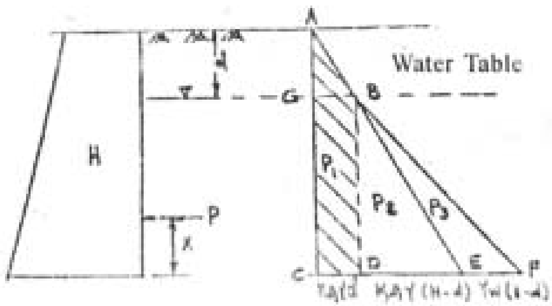


Fig. 5

Y	=	Bulk density of soil
Y^1	=	Submerged density of soil
Yw	=	density of water
Y'	=	$Y_{sat} - Yw$

Pressure at the base of foundation

P = Area (AGB + GBCD) + BDE + BEF

$P = P_1 + P_2 + P_3$

P_1 = Pressure due to bulk wt of soil

P_2 = Pressure due to submerged wt of soil

P_3 = Pressure due to water

Distance of resultant pressure is calculated by taking moment of all forces about base.

Case IV : When there is super-imposed load q and C = 0

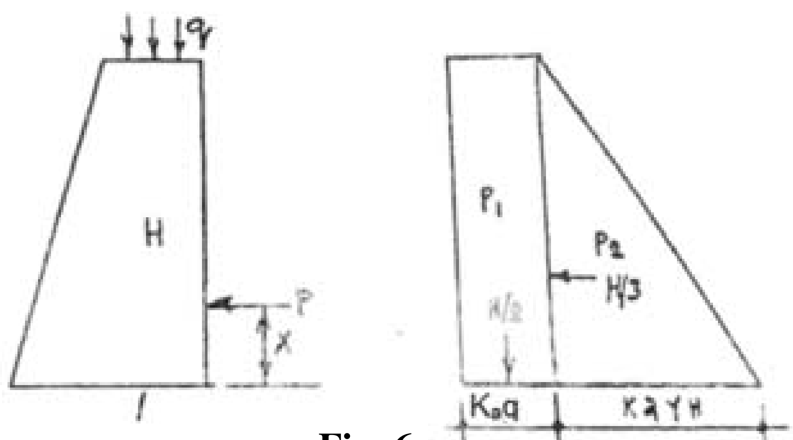


Fig. 6

$P = P_1 + P_2$

$Pa = K_aqH + 1/2Ka\ YH^2$

Note: In case of passive pressure Ka is replaced by Kp for case I to IV.

$$Kp = \frac{1}{ka}$$

Case V: Earth pressure in cohesive soil

$$P_a = KaYZ - 2C\sqrt{Ka} \quad \& \quad Ka = \frac{1-sin\phi}{1+sin\phi}$$

$$P_p = KpYZ + 2C\sqrt{Kp}$$

P_a = Earth pressure intensity at distance Z from top

$$K_p = \frac{1-sin\phi}{1+sin\phi}$$

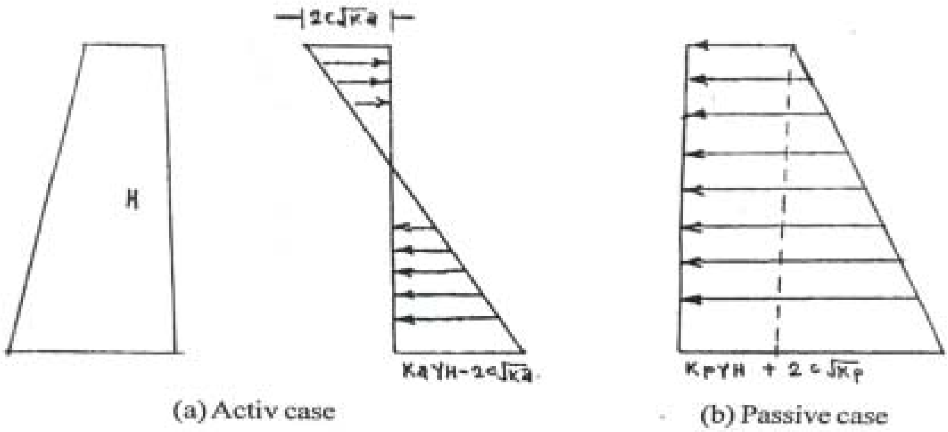


Fig. 7 : Pressure distribution

Area of respective pressure diagram gives the resultant pressure.

B. COLOMB'S ACTIVE PRESSURE IN COHESIONLESS SOIL

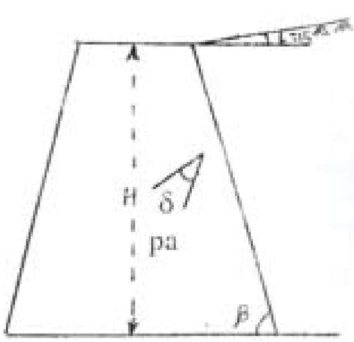


Fig. 8

Φ = Angle of repose of soil

δ = Wall friction angle

γ = density of soil

i = surcharge angle

β = Inclination of back of the wall

Where Ka= $$\left(\frac{\sin(\beta+\phi)}{\sin\beta\left\{\sqrt{\sin(\beta-\delta)}+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-i)}{\sin(\beta+i)}}\right\}}\right)^2$$

Where Kp= $$\left(\frac{\sin(\beta-\phi)}{\sin\beta\left\{\sqrt{\sin(\beta+\delta)}-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+i)}{\sin(\beta+i)}}\right\}}\right)^2$$

Table - 4

COEFFICIENT OF FRICTION AT FOUNDATION AND STONES

Sr No.	Surface	Coefficient friction μ	Remarks
1.	Stone Masonry on moist clay	0.30	
2.	Stone Masonry on dry clay	0.50	
3.	Stone Masonry on dry sand	0.50	
4.	Stone Masonry on gravel	0.60	
5.	Stone Masonry cut or hard rock	0.70	
6.	Lime stone	0.75	
7.	Brick on brick	0.70	
8.	Cement block on same	0.65	
9.	Cement concrete on clay	0.20	
10.	Cement concrete on sand	0.40	
11.	Cement concrete on gravel	0.40	
12.	Wood on same	0.46	

Table - 5

COEFFICIENT OF EARTH PRESSURE Ka FOR VARIOUS GROUND SLOPES AND BACK FACE VERTICAL

Sr No.	**Φin degree**	**Ground slope/Surcharge Angle = (i)**				**Remarks**
		i = 0	i = 5°	i = 10°	i = 15°	
1.	30	0.297	0.317	0.342	0.375	
2.	35	0.245	0.259	0.277	0.300	
3.	40	0.20	0.210	0.222	0.237	
4.	45	0.16	0.167	0.175	0.186	
5.	50	0.125	0.130	0.136	0.145	

Note:- Angle of wall friction has been taken 2/3 ϕ or 22.5° whichever is lesser as applicable for stone masonry wall.

Example 1: Determine the active pressure on the retaining wall as shown in Fig. 9

Yw = 10 KN/m^3

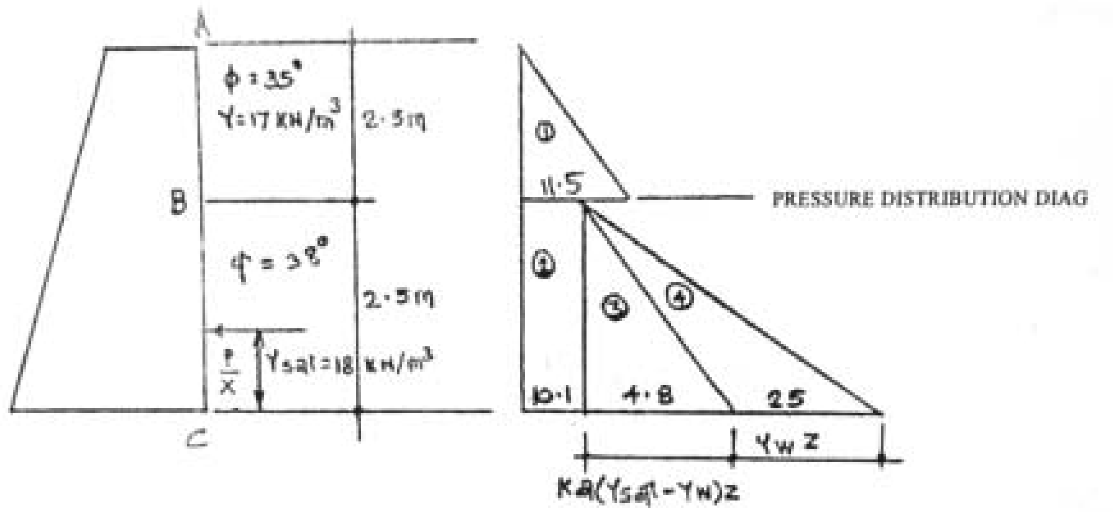


Fig. 9 : Pressure Distribution Diagrame

Soln : Ka $= \frac{1-\sin\Phi}{1+\sin\Phi}$

For upper layer Ka = $\frac{1-\sin 35^0}{1+\sin 35^0}$ = 0.271

For bottom layer Ka = $\frac{1-\sin 38^0}{1+\sin 38^0}$ = 0.238

Now at Point B

For upper layer pa1 = $Ka_1 Y_1 Z_1$ = 0.271 x 17 x 2.5 = 11.5 KN/m^2

For bottom layer pa2 = $Ka_2 Y_1 Z_1$ = 0.238 x 17 x 2.5 = 10.1 KN/m^2

Now at Point C

Pa_3 = $Ka_2 (Y_{sat} - Y_w) Z$

= 0.238 (18 – 10) x 2.5 = 4.8 KN/m^2

$$Pa_4 = \gamma_w Z$$

$$= 10 \times 2.5 = 25 \text{ KN/ m}^2$$

Now $P1 = \frac{1}{2} \times 11.5 \times 2.5 = 14.4$ KN

$P2 = 10.1 \times 2.5 \times 4.8 = 25.3$ KN

$P3 = \frac{1}{2} \times 2.5 \times 4.8 = 6.0$ KN

$P4 = \frac{1}{2} \times 25 \times 2.5 = 31.25$ KN

$P = 77.0$ KN

Now taking moment about point C

$$X = \frac{p_1x_1+p_2x_2+p_3x_3+p_4x_4}{p}$$

$$= \frac{14.4\left(2.5\frac{2.5}{2}\right)+25.3 \times \frac{2.5}{2} + 6 \times \frac{2.5}{2}\ 31.25 \times \frac{2.5}{2}}{77}$$

$$= \frac{48+31.625+5+26.04}{77} = 1.44\text{m}$$

P = 77 KN, acting at 1.44 m from bottom

Example 2: A 5 m high retaining wall is shown in Figure. Determine the Rankine active pressure on wall.

(a) Before the formation of crack

(b) After the formation of crack

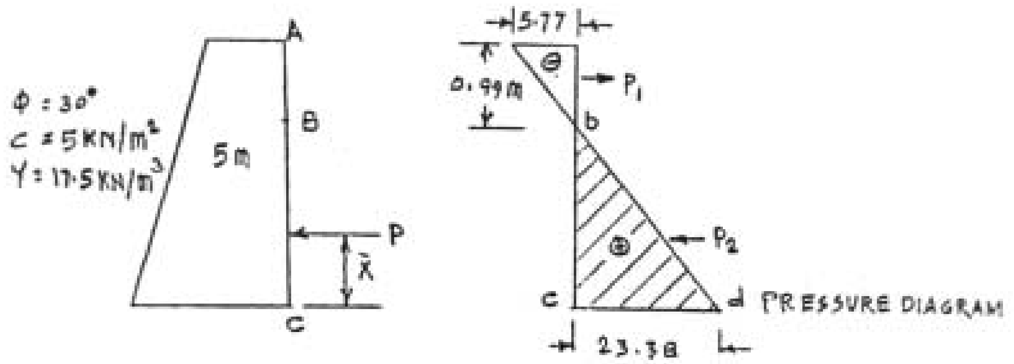


Fig. 10

Solution: $Ka = \frac{1-\sin\Phi}{1+\sin\Phi}$

$= \frac{1-\sin\Phi}{1+\sin\Phi} = 0.333$

Now $Pa = KaYZ - 2C\sqrt{Ka}$

$= 0.333 \times 17.5\ Z - 2 \times 5\sqrt{0.333}$

$Pa = 58.3\ Z - 5.77$

Now at $Z = 0$ $\quad pa = 5.77$

For $pa = 0$ $\quad Z = \frac{5.37}{5.83} = 0.99$

At $Z = 5$ $\quad pa = 5.83 \times 5 - 5.77$

$= 23.28\ KN/m^2$

(a) <u>Before the formation of crack</u>

Negative pressure $P_1 = ½ \times 0.99 \times 5.77$

$= 2.86$ KN

Positive Pressure $P_2 = ½ \times 23.38 \times 4.01$

$= 46.88$ KN

Total pressure $P = P_1 + P_2$

$= 46.88 - 2.86$

Taking moment about – c $= 44.02$ KN

$$8 = \frac{46.88\frac{4.02}{5} - 2.86\left(5 - \frac{0.99}{5}\right)}{44.02}$$

$$X = \frac{62.66-13.256}{44.02} = 1.12$$

P = 44.02 KN acting at 1.12 m from Bottom.

(b) After formation of crack

After the formation of crack, the negative pressure is eliminated and only passive pressure p_2 is considered.

$Pa = P_2 = 46.88$ KN acting at height $\frac{4.01}{2}$ = 1.336 m above the base

Example 3: Determine the coulomb active force on the retaining wall as shown below:

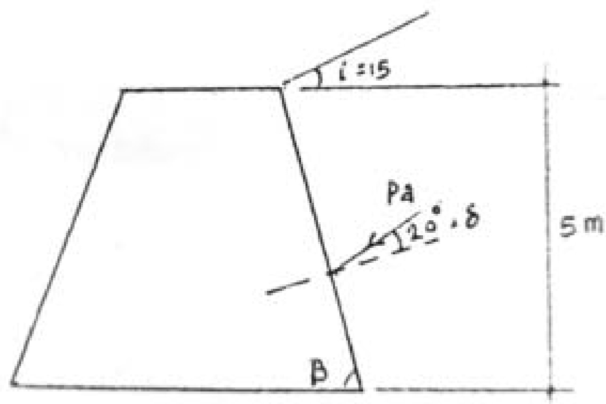


Fig. 11

$i = 15^0$

$\Phi = 30^0$

$\delta = 20^0$

$y = 17.5\ KN/m^3$

$\beta = 75^0$

$$\text{Solution } Ka = \frac{\sin^2(\beta+\beta)}{\sin^2\beta \ \text{Sin}(\beta-a)\left[1+\frac{\sin(\beta+a)\ \text{Sin}(\beta-1)^2}{\text{Sin}(\beta-a)\ \text{Sin}(\beta+1)}\right]}$$

= 0.548

$Pa = ½\ Ka\ YH^2 = ½\ 0.548 \times 17.5 \times 25 = 119.9$ KN at height 5/3 m

And inclined at 20^o to the normal in the direction shown-

15. PRINCIPLES OF THE DESIGN AND SAFETY OF RETAINING WALLS

15.1 Design Principles

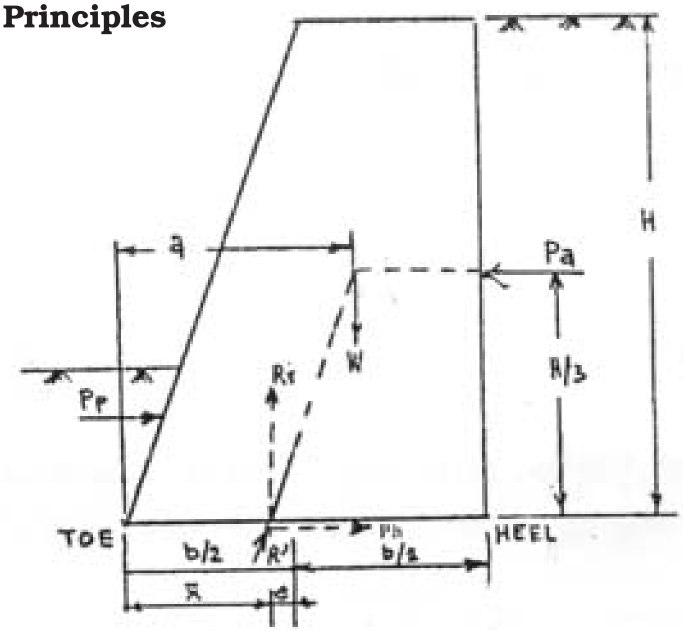


Fig. 12

Pa = active pressure acting horizontally

P_p = Passive pressure below the soil surface

Pp is small and therefore it may be neglected and gives more Conservative design.

W = Weight of wall

R = Resultant Force

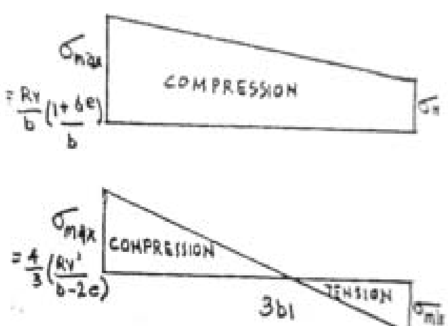


Fig. 13

(a)Pressure diagram at base when ed≥b/6

$$\sigma_{max} = \frac{RV}{b}\left(1 + \frac{6e}{b}\right)$$

(b) Pressure diagram at base when e >

b/6

b 1 + e = b/2

The portion of base-width where there is tension would become ineffective.

The width 3b1 will bear all the pressure.

From equilibrium of system

Rv = W - (1)

Rh = Pa - (2)

Taking moment of all the forces about Toe

$$Rv \times X = Wa - Pa \frac{(H)}{3}$$

$$X = \frac{W x\ a - P_a \times H/3}{Rv^2} \qquad \frac{W \times a - P_a \times H/3}{W^2} = \frac{\Sigma M}{\Sigma V}$$

$$\text{Now} \quad e + X = \frac{b}{2}$$

$e = \frac{b}{2} - X$ Where e = eccentricity of Resultant force R from Centre of base.

For a safe design following requirements must be satisfied.

15.2 **Safety in Desigh**

15.2.1 **No sliding.** The wall must have a factor of safety of 1.5 against sliding. To meet the condition tan must not exceed half the coefficient of friction of wall of the foundation material. The value of tan m can be decreased, if this proves necessary by sloping the foundation downwards, towards the backfill and taking this foundation slope as 1 in q (tan = 1/q) must be less than half the co-efficient of friction.

Where δ = angle of resultant force on the wall to the vertical

1/q = foundation slope (e.g. for slope of 1 in 8, q=8)

$$\text{Tan}\ \mu = \frac{CW\ H^2}{2V+PB} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

Where V = Vertical load per m length of wall from the vertical component of the backfill pressure and any super imposed load.

B = width of base without toe projection

P = Toe pressure on base B in Kg/sqm

$\frac{1-\sin\Phi}{1+\sin\Phi}$ For level back fill

$$Fs = \frac{\mu R_v}{R_h}$$

Fs = Factor of safety against sliding (See IS-1904)

Where Rv = vertical component of Resultant force R

R_h = Horizontal component of Resultant force R

μ = Coefficient of friction between base of wall and soil (Tan ä)

I. = for static loads - Fs> 1.5

II. = For static load with earthquake forces -Fs> 1.0

Sometimes to achieve minimum factor of safety and thereby resist sliding, it may be necessary to increase the base area or to add concrete keys monolithic with foundation slab or to provide piles.

15.2.2 **No Overturning.**

$$Fo = \frac{\Sigma M_R}{\Sigma M_O}$$

Fo = Factor of safety against overturning (See IS – 1904)

Where,

ΣM_R = Sum of resisting moment about toe

ΣM_O = Sum of overturning moment about toe

I. $F_o \geq 2.0$ Under normal or static load condition as per IS 1904.

II. $Fo \geq 1.5$ Under earthquake forces as per IS 1893-1981

In any case section should not be lower than that required for normal condition.

15.2.3 **No bearing capacity failure.**

$$Fb = \frac{q_{na}}{\delta \max}$$

Fb = Factor of safety against bearing capacity.

Where q_{na} = allowable bearing pressure

δ max = Maximum pressure at Toe cause by Rv

$$\delta \max = \frac{Rv}{b} \left(1 + \frac{6a}{b}\right)$$

i. $F_b \geq 3$ Under normal load condition

ii. $F_b \geq 1.5$ under earthquake load provided the settlement is also within allowable limit.

15.4 **No Tension.**

There should be no tension at base of wall

$$ie = \frac{Rv}{b} \left(1 + \frac{6a}{b}\right) \geq 0$$

b/6 d $\leq$ e (middle third rule)

i.e. e should be within middle third of base

16. PROVISION OF TOE WALL

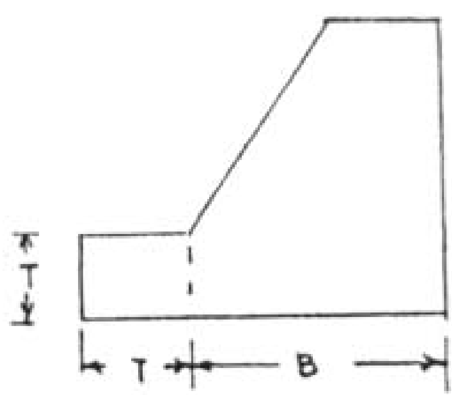
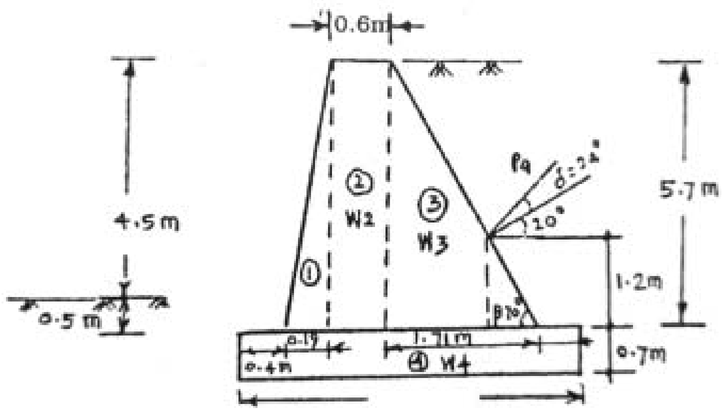


Fig. 14

If the Toe pressure exceeds the allowable limit, a toe wall "T" can be projected or provided in the following proportions to reduce the effect on the pressure. The provision of these has to be made as per requirement of the condition.

For	T	= 0	Toe pressure ä max becomes δ max
	T	= B/6	0.62 δ max
	T	= B/5	0.56 δ max
	T	= B/4	0.48 δ max
	T	= B/5	0.37 δ max

Example 4:- Check the stability of gravity Retaining wall as shown in following diagram. Take allowable soil Pressure equal to 600 KN/Sqm.



Ys = 19 KN/m^3

Φ = 24.3^0

δ = 24^0

Yc = 24KN/m^3

Fig. 15

Calculation of active Earth pressure

$$Ka = \frac{1-\sin\Phi}{1+\sin\Phi} = \frac{1-\sin24.3}{1+\sin24.3} = 0.417$$

$$Pa = \tfrac{1}{2} KaYH^2 = \tfrac{1}{2} \times 0.417 \times 19 \times (5.7)^2 = 128.7 \text{ KN}$$

Total pressure acts at an inclination 24^o to the normal

Now P_U = Pa Cos (20+24)^o = 128.7 x 0.719 = 92.58 KN

P_V = Pa Sin(20+24)^o = 128.7 x 0.6946 =89.40 KN

Remarks: - In gravity masonry R/Wall Wt of soil coming on wall is not taken into consideration.

TABLE - 6

CALCULATION

Sr No.	Description	Forces KN		Lever Arm (m)	Moment about To (KN-m)	
		Vertical KN	Horizontal KN		Clockwise (+ve)	Anticlock wise (-ve)
1.	Weight W1½ x 5 x 0.19 x 24	11.4	-	0.53	6.0	
2.	Weight W25 x 0.6 x 24	72	-	0.89	64.08	
3.	Weight W3½ x 5 x 1.17 x 24	102.6	-	1.76	180.576	
4.	Weight W4 3.3 x 0.7 x 24	55.44	-	1.65	91.476	
5.	Vertical component of Pa = Pv	89.4	-	2.36	210.98	
6.	Horizontal component ofPa = PH	-	92.58	1.9	-	175.9
		ΣR_V= 330.84	ΣR_H= 92.58		ΣM_R= 553.112	ΣM_O= 175.9

ΣM= 377.212KN

(a) **No sliding**

$$Fs = \frac{\mu R_V}{R_H} = \frac{\tan 24^o\ 330.84}{92.58} \quad \mu$$

$$= 1.59 \text{ (Safe)}$$

(b) **No Overturning**

$$Fo = \frac{\Sigma M_R}{\Sigma M_O} = \frac{553.112}{175.90} = 3.14 \text{ (safe)}$$

(c) **No bearing capacity failure**

$$F_b = \frac{q_{na}}{\delta max} \qquad q_{na} = 600KN/m^2$$

$$\delta\ max = \frac{Rv}{b}\left(1+\frac{6a}{b}\right)$$

$$e = \frac{b}{2} - x \quad \text{Where } x = \frac{\Sigma M}{\Sigma R_V} = \frac{377.212}{330.84} = 1.14m$$

$$e = \frac{3.3}{2} - 1.14 = 0.51m$$

$$\delta max = \frac{330.84}{3.3}\left[1 + \frac{6 \times 0.51}{3.3}\right] = 193.22 \text{ KN/m}^2$$

$$F_b = \frac{600}{193.22} = 3.11 \quad \text{(safe)}$$

(d) **No Tension** :-

Since $e \leq b/6$

Hence – no Tension

17. FRONT VERTICAL FACED RETAINING WALLS

17.1 In normal practice retaining walls are being provided with a front batter of 1: 4 and back vertical. The structures are becoming quite high due to sloping valley face on which these are built resulting in increased cost of construction. Instead of keeping front face vertical a steeper front batter of 1: 10 can be provided with projection of 15 cm at the toe.

17.2 Vertical faced retaining wall has got several advantages as compared to flatter faced retaining wall enumerated below:

(a) Height of structure and consequently its volume and weight gets reduced.

(b) Cost of structure and construction time gets reduced.

(c) Toe pressure results in better stability of the valley side slope (See Appendix A).

(d) Front faced vertical walls provide wider road width.

17.3 **Design consideration**

(a) For an average case the angle of repose of soil has been taken as 33^0.

(b) The effort of seismic forces to be considered in the design as per IS 1893 – 2002 “Criteria for Earthquake Resistant Design of

Structures" for severest case that is Zone V.

(c) The increase in permissible stresses for seismic effect to be taken as per clause 706.1.2 of IRC- 78- 2014, while working out factor of safety.

(d) Saturated conditions have not been considered as sufficient weep holes and adequate filter media are provided for efficient drainage.

(e) Live load surcharge to be taken as per clause 214.1.1.3 of IRC-6-2014.

(f) The safety of structure to be checked against sliding, overturning, bearing capacity as per codal provision.

Foundation concrete has been provided in ratio 1:3:6 and of 30 Cm thickness.

Example 5 :- Design a retaining wall having 50^0 natural valley slope for convention as well as modified method and compare the quantities.

Assumption

i.	áv	=	0.04	$\alpha h = \beta\ I\ a_0$
ii.	Φ	=	33^0	= 1.00 x 1.00 x 0.08
iii.	λ	=	4.4	=0.08
iv.	a	=	12^0	$\lambda = \tan^{-1} \frac{1\ \alpha h}{1+\alpha v}$
v.	δ	=	27^0	$\tan^{-1} \frac{0.08}{1+0.04}$
vi.	e	=	0^0	$\tan^{-1} 0.076\ 923 \quad = 4.4^0$
vii.	Density of earth = 1.6 ton/cum			
viii.	Density of stone mass			= 2.40 ton/cum
ix.	Co-efficient of friction (i)			= 0.50
x.	Angle of repose			= 33^0

xi.	Permissible bearing capacity of foundation soil	
(a)	Height of R/wall up to 5.00 mtr	= 15 + 50% for Earthquake = 22.50 ton/m^2
(b)	Height of R/wall above 5.00 mtr	= 20 + 50 % for earthquake
xii.	Seismic Zone with horizontal	= 30.00 ton/m^2
xiii.	Seismic co-efficient a^h	= 0.08
xiv.	Surcharge load due to traffic equivalent Height of earth fill	= 0.6 m

Ref: Clause 214.1.2.1 of IRC 6 -2014

$$Ca = \frac{(1+av)\cos^2(\phi-\lambda-\alpha)}{\cos\lambda \cos^2\alpha \cos(\delta+\alpha+\lambda)} \text{ x } \left[\frac{1}{1+\left\{\frac{\sin(\phi+\delta)\sin(\phi-i-\lambda)}{\cos(\alpha-i)\cos(\delta+\alpha+\lambda)}\right\}^{1/2}}\right]^2$$

$$= \frac{(1+0.04)\cos^2(33-4.4-12)}{\cos 4.4 \cos^2 12 \cos(27+12+4.4)} \text{ x } \left[\frac{1}{1+\left\{\frac{\sin(33+27)\sin(33-0-4.4)}{\cos(12-0)\cos(27+12+4.4)}\right\}^{1/2}}\right]^2$$

$$= \frac{1.04\times\cos^2 16.6}{\cos 4.4 \cos^2 12 \cos 43.4} \text{ x } \left[\frac{1}{1+\left\{\frac{\sin 60 \sin 28.60}{\cos 12 \cos 43.4}\right\}^{1/2}}\right]^2$$

$$= \frac{1.04\times 0.918}{0.997\times 0.957\times 0.727} \text{ x } \left[\frac{1}{1+\left\{\frac{0.866 \text{ x } 0.479}{0.978 \text{ x } 0.727}\right\}^{1/2}}\right]^2$$

$$= 1.376 \left[\frac{1}{1+(0.584)^{1/2}}\right]^2$$

$$= 1.376 \left[\frac{1}{1+0.764}\right]^2$$

$$= 1.376 \text{ x } 0.321 \quad = 0.442$$

$$\text{cs} = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cos(\delta+\alpha)} \times \left[\frac{1}{1+\left\{\frac{\sin(\phi+\delta)\sin\phi}{\cos\alpha\cos(\delta+\alpha)}\right\}^{1/2}}\right]^2$$

$$= \frac{\cos^2(33-12)}{\cos^2 12\cos(27+12)} \times \left[\frac{1}{1+\left\{\frac{\sin(33+27)\sin 33}{\cos 12\cos(27+12)}\right\}^{1/2}}\right]^2$$

$$= \frac{\cos^2 21}{\cos^2 12\cos 39} \times \left[\frac{1}{1+\left\{\frac{\sin 60\ \sin 33}{\cos 12\cos 39}\right\}^{1/2}}\right]^2$$

$$= \frac{0.872}{0.957 \times 0.777} \times \left[\frac{1}{1+\left\{\frac{0.866 \times 0.545}{0.978 \times 0.777}\right\}^{1/2}}\right]^2$$

$$= 1.173 \times \left[\frac{1}{1+\{0.621\}^{1/2}}\right]^2$$

$$= 1.173 \times \left[\frac{1}{1+0.788}\right]^2$$

$$= 1.173 \times 0.313 \quad = 0.367$$

Recommended Section of Retaining Walls in Steep Valley slopes

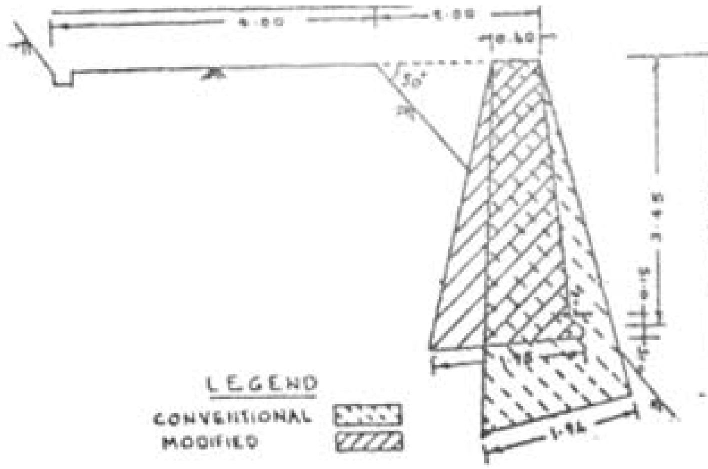


Fig. 16

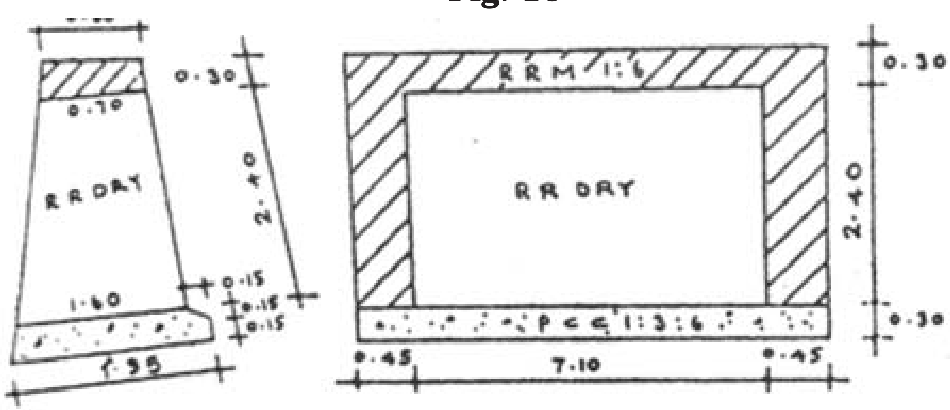


Fig. 17 Calculation of quantities (Modified)

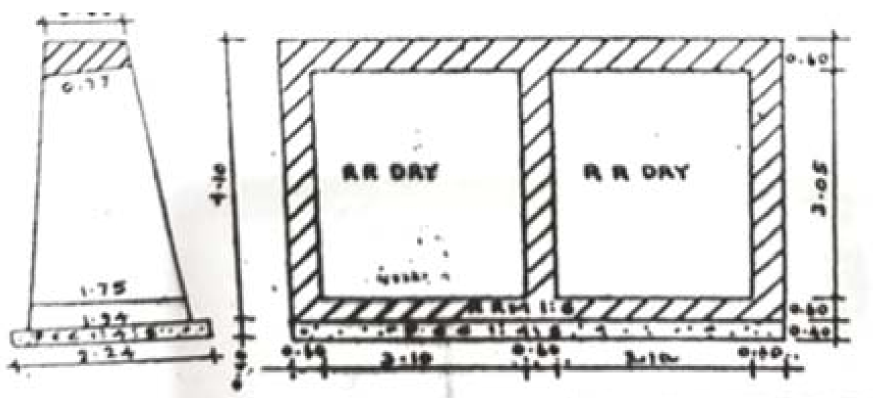


Fig. 18 Calculation of Quantities Conventional

(A) MODIFIED RETAINING WALL (50^0 Angle)
CHECK AGAINST SEISMIC EFFECT

Height of R/Wall = 3.45 m or 3.64

h = 3.64 + 0.6 = 4.24 m

$\frac{h}{3}$ = 1.41 m and = $\frac{h}{2}$ = 2.12 m

Ps = ½ wh^2 Cs

= ½ x1.60 x $(4.24)^2$ x 0.367

= ½ x 1.60 x 17.98 x0.367

= 5.28 Ton acting at 1.41 m from heel

P_d = ½ x wh^2 cd

= ½ x 1.6 $(4.24)^2$ x 0.442

= ½ x 1.6 x 17.98 x 0.442

= 6.36 Ton

PD_1 = P_d – Ps = 6.36 – 5.28 = 1.08 Ton acting at 2.12 from heel

$\delta + \alpha$ = 27 + 12 = 39^0

Cos 39^0 = 0.777

Sin 39^0 = 0.656

Table - 7

Sr No.	Horizontal Component				Vertical Component		
	Force	**Force in ton**	**Level Arm**	**Moment**	**Force in ton**	**Letter Arm**	**Moment**
1.	P_S	5.28 X 0.777=4.10	1.41	5.78	5.28 X 0.656=3.46	0.34	1.08
2.	W_1				10.56	1.04	10.98
3.	W_2				2.50	0.30	0.75
SEISMIC EFFECT							
4.	PD_1	1.08 X 0.777 = 0.84	2.12	1.78	1.08 X 0.656 = 0.71	0.48	0.34
5.	W_1	10.56 X 0.08 = 0.84	1.46	1.23			
6.	W_2	2.50 X 0.80 = 0.20	2.44	0.49			
Total		5.98		=9.28	17.23		13.15

Total moment = 9.28 + 13.15 = 22.43 Tm

Resultant load normal to base

Bed slope is 1: 10 i.e. 5^0

$Cos5^0$ = 0.997 and Sin 5^0 = 0.087

= 5.98 x 0.087 + 17.23 x 0.997

= 0.52 + 17.18

= 17.70 Ton

$$Z = \frac{22.43}{17.70} = 1.27 \text{ Mtr}$$

$$2/3 \text{ of base} = \frac{2}{3} \times 1.95 = 1.30 \text{ mtr}$$

Resultant passes in middle third of base hence structure is not in tension

$$e = Z \frac{b}{2} = 1.27 - \frac{1.95}{2} = 0.295 < \frac{b}{6} \text{ ie } 0.325 \text{ hence safe}$$

Check against Sliding.

Resultant force causing sliding against base:

4.98 x 0.997 - 17.23 x 0.087 = 5.96 - 1.50 = 4.46

Force resisting the sliding = 0.5 x W

= 0.5 x 17.70

= 8.85

Factor safety against sliding = 8.85/4.46

= 1.98 > 1.25 hence safe

Check against toe pressure.

$$Po = \frac{17.70}{1.98} = 9.08 \text{ Ton/Mtr}^2$$

$$P_T = Po\left(1+\frac{6e}{b}\right)$$

$$= 9.08\left(1+\frac{6 \times 0.295}{1.95}\right)$$

$= 9.08\ (1 + 0.91)$

$= 9.08 \times 1.91$

$= 17.32 < 20$ hence safe

$$P_M = Po\left(1-\frac{6e}{B}\right)$$

$$= 9.08\ \frac{1-6 \times 0.295}{1.95}$$

$= 9.08 \times (1 - 0.91)$

$= 9.08 \times 0.09$

$= 0.82$ is not negative hence safe

Check against overturning

<u>Over turning moment</u>

Ps	=	5.28 x 0.777 x 1.20	=	4.92
PD_1	=	1.08 x 0.777 x 1.90	=	1.59
W_1	=	10.56 x 0.08 x 1.24	=	1.05
W_2	=	2.50 x 0.08 x 2.22	=	<u>0.44</u>
				8.00

<u>Stabilizing moment</u>

Ps	=	5.28 x 0.656 x 1.60	=	5.54
PD_1	=	1.08 x 0.656 x 1.44	=	1.02
W_1	=	10.56 x 0.90	=	9.50
W_2	=	2.50 x 1.64	=	<u>4.10</u>
				20.16

Factor of safety against overturning = 20.16/8

= 2.52 < 1.5 hence safe

W1 = $\left(\frac{0.61+1.80}{2} \times 3.64 \times 2.40\right) + 0.08 = 10.48 + 0.08 = 10.56$

		V	H
A	-	1.24	0.90
B	-	1.46	1.04

W2 = ½ x 3.64 x 0.86 x 1.60 = 2.50

		V	H
A	-	2.22	1.64
B	-	2.44	0.30

(B) Conventional Retaining Wall

Height of R/Wall = 4.10 mtr or 4.62 mtr

h = 4.62 + 0.6 = 5.22 mtr

$\frac{h}{3}$ = 1.74 mtr and $\frac{h}{2}$ = 2.61 mtr

Ps = $\frac{1}{2} wh^2 C_s$

= $\frac{1}{2} \times 1.6 \times (5.22)^2 \times 0.367$

= 0.80 x 27.25 x 0.367

= 8.00 Ton acting at 1.74 mtr from heel

Pd = $\frac{1}{2} \times wh^2 C_d$

= $\frac{1}{2} \times 1.6 \times (5.22)^2 \times 0.442$

= 0.82 x 27.25. 0.442

= 9.64 Ton acting at 2.61 mtr from heel

PD_1 = $P_d - P_s$ = 9.64 – 8.00 = 1.64 Ton acting at 2.61 m from heel

$\delta + \alpha$ = 27 + 2 = 29^0 Cos 29^0 = 0.875

Cos 29^0 = 0.875 Sin 29^0 = 0.485

Table - 8

Sr No.	Horizontal Component				Vertical Component		
	Force	Force in ton	Level Arm	Moment	Force in ton	Letter Arm	Moment
1.	P_S	8.00 x 0.875 = 7.00	1.74	12.18	8.00 x 0.485 = 3.88	0.04	0.16
2.	W_1				14.08	0.78	10.98
3.	W_2				0.67	0.02	0.01
SEISMIC EFFECT							
4.	PD_1	1.64 x 0.875 = 1.44	2.61	3.76	1.64 x 0.485 = 0.80	0.10	0.08
5.	W_1	14.08 x 0.08 = 1.13	1.78	2.01			
6.	W_2	0.67 x 0.08 = 0.05	3.14	0.16			
Total		9.62		18.11	19.43		11.23

Total moment = 18.11 + 11.23 = 29.34 Ton m

Resultant load normal to base

Bed slope is 1: 4 i.e. 12^0

Cos 120 = 0.978 and Sin 12^0 = 0.208

= 9.62 x 0.208 + 19.43 x 0.978

= 2.00 + 19.00

= 21.00

$$Z = \frac{29.34}{21} = 1.40 \text{ m}$$

$$2/3 \text{ of base} = \frac{2}{3} \times 1.94 = 1.29 \text{ m}$$

Resultant passes in middle third of base hence structure is not in tension

$$e = z - \frac{b}{2} = 1.40 - \frac{1.94}{2} = 0.43 > \frac{b}{6} \text{ ie } 0.32 \text{ hence safe}$$

Check against Sliding

Resultant force causing sliding against base:

9.62 x 0.978 – 19.43 x 0.208 = 9.41 – 4.04 = 5.37

Force resisting the sliding = 0.5 x W = 0.5 x W = 0.5 x 21 = 10.50

Factor of safety against sliding =10.50/5.37 =1.96 > 1.25 hence safe

Check against toe pressure

$$P = \frac{21.00}{1.94} = 10.82 \text{ Ton/Mtr}^2$$

$$P_T = Po\left(1 + \frac{6e}{b}\right)$$

$$= 10.82\left(1 + 6 \text{ x } \frac{0.43}{1.94}\right)$$

$$= 10.82 \ (1+1.33) = 10.82 \ (2.33)$$

$$= 25.21 > 22.50 \text{ hence safe}$$

$$P_M = Po\left(1 \quad \frac{6e}{b}\right)$$

$$= 10.82\left(1- \frac{6\text{x } 0.43}{1.94}\right)$$

$$= 10.82 \ (1- 1.33)$$

$$= 10.82 \text{ x } (-) \ (0.33)$$

$$= (-) \ 3.57 \text{ is negative hence not safe}$$

Check against overturning

Over turning moment

Ps	=	8.00 x 0.875 x 1.22	=	8.54
PD_1	=	1.64 x 0.875 x 2.10	=	3.01
W_1	=	14.08 x 0.08 x 1.26	=	1.42
W_2	=	0.67 x 0.08 x 2.62	=	0.14
				13.11

(x) Stabilizing moment

Ps = 8.00 x 0.485 x 1.76 = 6.83

PD_1 = 1.64 x 0.485 x 1.72 = 1.37

W_1 = 14.08 x 1.08 = 15.21

W_2 = 0.67 x 1.84 = <u>1.23</u>

24.64

Factor of safety against overturning = 24.64/13.11

= 1.88 > 1.5 hence safe

$$W_1 = \frac{0.60+1.94}{2} \times 4.62 \times 2.40 = 14.08$$

$$A = \frac{V}{1.26} \qquad \frac{H}{1.08}$$

B = 1.78 0.78

W2 = ½ x 0.18 x 4.62 x 1.60 = 0.67

A = 2.62 1.84

B = 3.14 0.02

CALCULATION OF QUANTITIES (CONVENTIONAL)
(NOT TO SCALE)

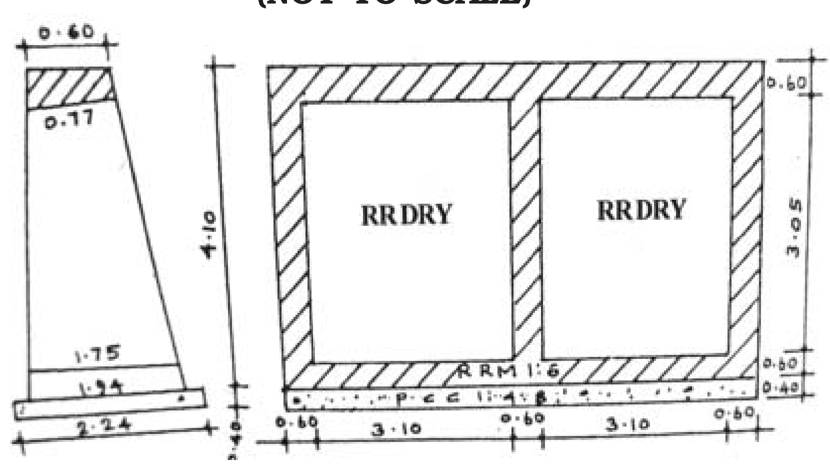


Fig. 19

Table – 9

Item of work	Nos.	L (M)	B(M)	H(M)	Qty
Excvn in trenches	01	08	$\frac{2.24}{2}$	$\frac{0.6+4.75}{2}$	24.01 cum
PCC 1 : 4 : 8	01	08	2.24	0.4	7.17 cum
RRM 1 : 6 (BB)	01	08	$\frac{1.94+1.75}{2}$	0.6	8.86 cum
(TB)	01	08	$\frac{0.6+0.77}{2}$	0.6	3.29 cum
(VB)	03	$\frac{1.75+0.77}{2}$	0.60	3.05	6.9219.07– 1.07=18.00 cum
RR Dry	02	3.10	$\frac{1.75+0.77}{2}$	3.05	23.44-1.36= 22.08 cum
B/Filling	01	8	$\frac{0.15+1.4}{2}$	4.50	27.90 cum
Bond Stone Total face are of wall	01	8	-	4.25	34 sq.m
Nos of B/Stone Average length of B/S	2 Nos /sqm		$\frac{(1.94+0.60)\text{x } 1.25}{2}$		68 Nos1.59 mtr.
PCC for B/Stone	68	1.59	0.15	0.15	2.43 cum
Area of Dry wall face	02	3.1	-	3.05	18.91 cum
Nos of B/Stone	2 Nos /sqm		-		38 Nos
Qty in RR Dry (B/S)	38	1.59	0.15	0.15	1.36 cum
Qty of B/S in RRM 1 : 6Centering/ Shuttering	1.43 – 1.36 =68/10 x 2 x 1.59 x 0.15				=1.07 cum 3.24 sqm

CALCULTION OF QUANTITIES (MODIFIED)

(NOT TO SCALE)

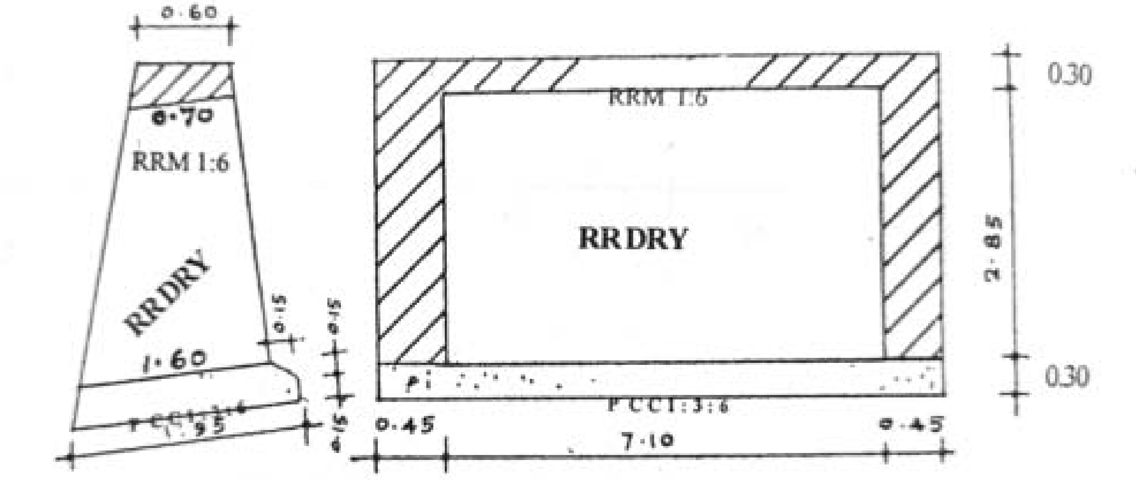


Fig. 20

Table - 10

Item of Work	**A/U**	**L**	**B**	**H**	**Qty**
Excvn in trenches	01	8	$\frac{1.95}{2}$		14.43 cum
PCC 1 : 3 ; 6	01 01 01	8 8 8	$\frac{1.80+1.60}{2}$ 0.15 0.15	0.30 0.15 0.15/2	4.08 0.18 0.09 = 4.35 cum
RRM 1 : 6 (Vert)(Hor)	02 01	$\frac{0.7+1.60}{2}$ 8	0.45 $\frac{0.6+0.7}{2}$	2.85 0.30	2.95 1.56 4.51-0.31= 4.20 cum
RR Dry	01	7.10	$\frac{0.7+1.60}{2}$	2.85	23.27-1.24 22.03 cum
B/Filling	01 01	8 8	$\frac{0+1.40}{2}$ 15/2	3.10 0.30	17.36 0.18 17.54

Item of Work	A/U	L	B	H	Qty
Total Area of wall face	01	8	-	3.15	25.20 Sqm
Nos of B/S@2 Nos/Sqm		=25.20	X 2 = 50.4	Say	50 Nos
Average length of B/S			1.60+060 2	+25%	1.38 Mtr
PCC for B/Stone	50	1.38	0.15	0.15	1.55 cum
Area of wall face Nos of B/stone	1@2 Nos./ Sqm for	7.10	-20.24x2	2.85= 40.48 Say	20.24 Sqm 40 Nos
Qty of PCC in RR Dry	40	1.38	0.15	0.15	1.24 cum
Qty of PCC in		1.55-1.24			0.31 cum
RRM 1 : 6					
Centering/Shutte ring		=50 x 2 x 1.38 x 0.15			2.07 Sqm

COMPARASION OF QUANTITIES

Table - 11

Sr No.	Item	A/U	Quantity	
			Conventional	**Modified**
1.	Excvn in trenchs	Cum	24.01	14.43
2.	PCC 1:4:8	“	7.17	-
3.	PCC 1:3:6	“	-	4.35
4.	RMM 1:6	“	18.00	4.20
5.	RR Dry	“	22.08	22.03
6.	PCC 1:2:4	“	2.43	1.55
7.	B/Filling	“	28.52	17.54
8.	Centering/Shuttering for B/Stone	Sqm	3.24	2.07

17.4 **Factor of Safety.** A comparison of factor of safety with front face slope flatter (modified Design) and that of existing practice is given in Appendix 'A'. It is seen that there is improvement in the factor of safety as compared to existing practice. The back filling is generally done with boulders/hand packed stone fill, due to ample availability of boulders in the hill. This hand packed stone fill retains some back pressure by itself. Moreover, retained soil is not homogenous and is mixed with boulders, gravel etc., which have large angle of repose as compared to assumption of 33. Also the actual filling width is generally lesser than the theoretical width of the soil wedge which in other words means that the theoretical earth pressure as assumed in design calculating is not getting developed. This is illustrated in Fig. 21 & 22.

17.5 **Reduction in resources and Construction time**

(a) The Comparison of height and per meter cost between conventional and modified retaining walls is given at Appendix 'B'. It is seen that there is considerable reduction in height and cost.

(b) The reduction in cost due to modified proposal will work out to 35% on an average.

17.6 **Recommended Section of Retaining Wall in steep valley with natural slopes.** Retaining wall designed as per modified method makes the structure lighter, time saving and cost effective. Recommended section as retaining wall as per modified method is shown in Fig. 23 & 34.

Recommended Sections of Retaining Walls in Steep Valley Slopes

(a) Natural valley Slope 55^0

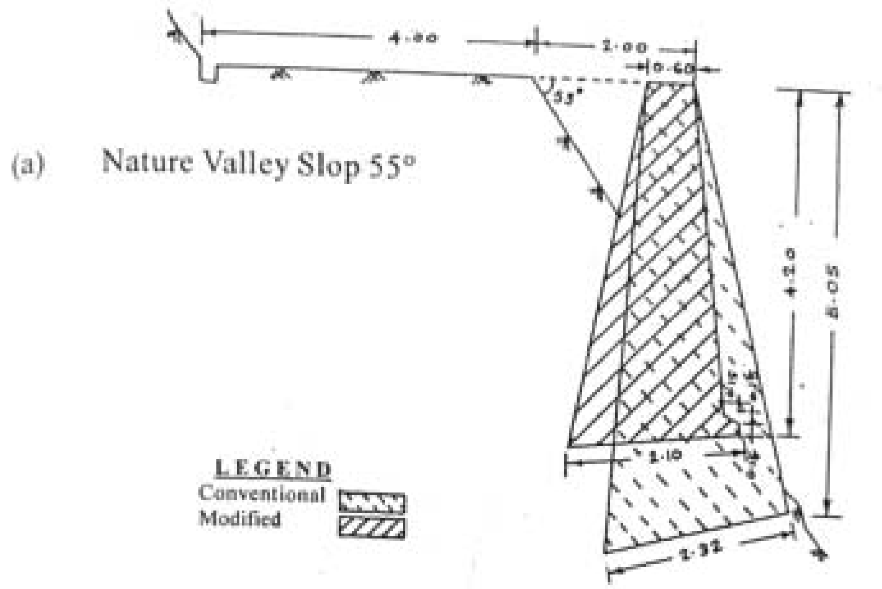


Fig. 21

(b) Natural Valley Slope 60^0

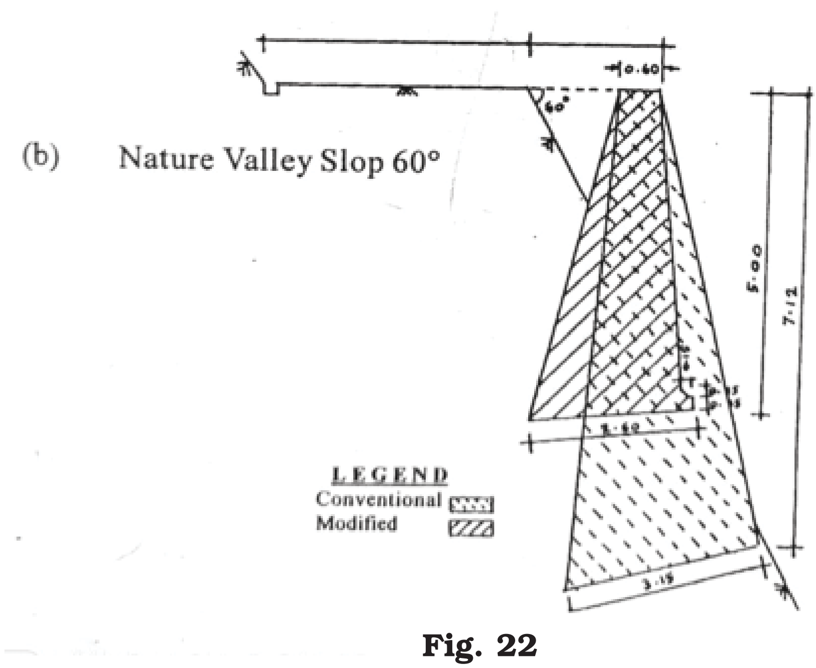


Fig. 22

Appendix 'A'

COMPARISON OF SAFETY FACTORS OF MODIFIED DESIGN WITH CONVENTIONAL DESIGN AGAINST SEISMIC EFFECT

Sr No	Valley side slope	Height of Wall		Factor of Safety														
				Conventional							Modified							
		Conv-entio-nal	Modi-fied	Against over turning		Against over sliding		Maximum toe pressure at foundation		Whether tension develops	Against over tension		Against sliding		Maximum toe pressure		Whether tension develops
				Allow-able	Act-ual	Allow-able	Act-ual	Allow-able	Act-ual		Allow-able	Act-ual	Allow-able	Act-ual	Allow-able	Act-ual	
1	45^0	3.35	2.90	1.50	1.95	1.25	2.11	22.50	21.57	Yes	1.50	2.80	1.25	2.18	22.50	11.23	No
2	50^0	4.10	3.45	1.50	1.88	1.25	1.96	22.50	25.21	Yes	1.50	2.52	1.25	1.98	22.50	17.34	No
3	55^0	5.05	4.20	1.50	2.11	1.25	2.24	30.00	28.53	Yes	1.50	2.26	1.25	1.90	30.00	21.53	No
4	60^0	7.12	5.00	1.50	1.92	1.25	2.10	30.00	48.96	Yes	1.50	2.32	1.25	1.97	30.00	25.95	No

Appendix 'B'

SAVINGS IN COST DUE TO MODIFIED PROPOSAL OVER CONVENTIONAL

Sr No.	Valley side slope	Height of Wall (in m)	% saving per m cost due to modified proposal
1.	45^0	2.90	26.46
2.	50^0	3.45	38.11
3.	55^0	4.20	35.55
4.	60^0	5.00	50.35

(A) Recommended Sections of Retaining Walls in Steep Valley Slopes – Natural Valley Slope 45^0

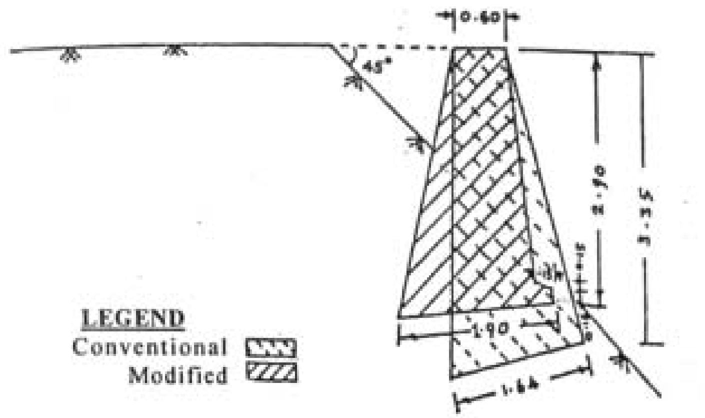


Fig. 23

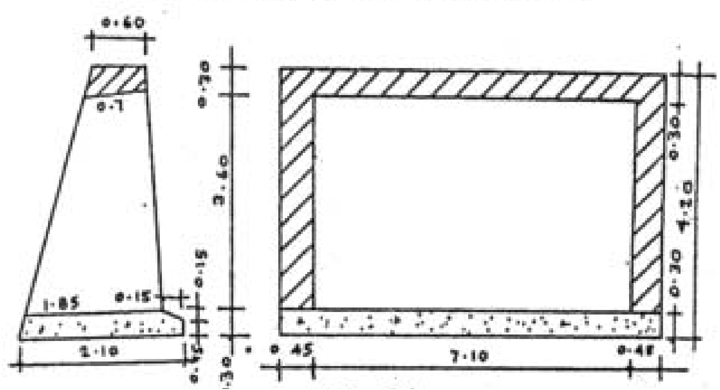


Fig. 24: Calculation of Quantities (Modified)

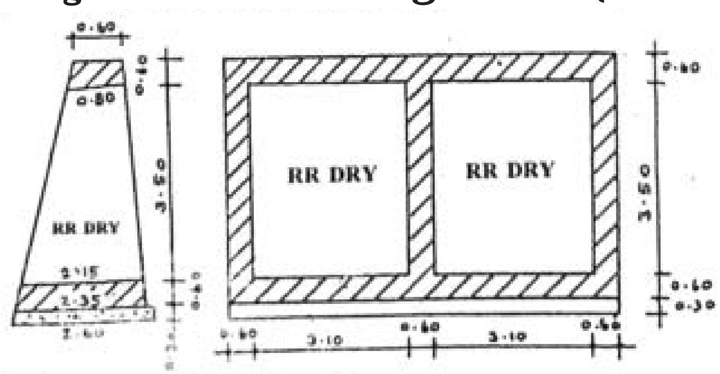


Fig. 25: Calculation of Quantities (Conventional)

(B) Recommended Section of Retaining Walls in Steep Valley Slopes – Natural Valley Slope 50^0

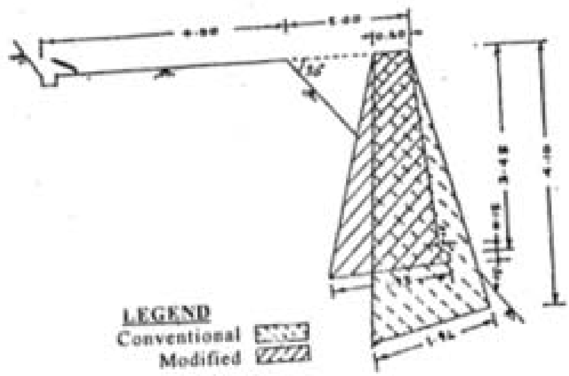


Fig. 26

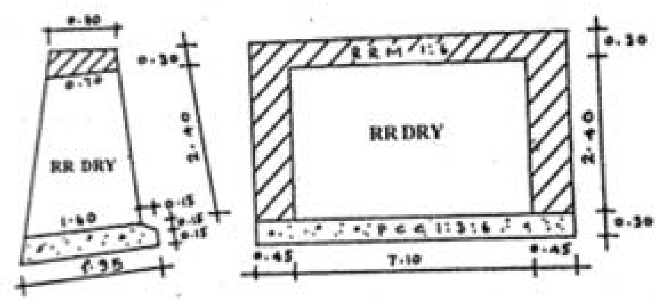


Fig. 27: Calculation of Quantities (Modified)

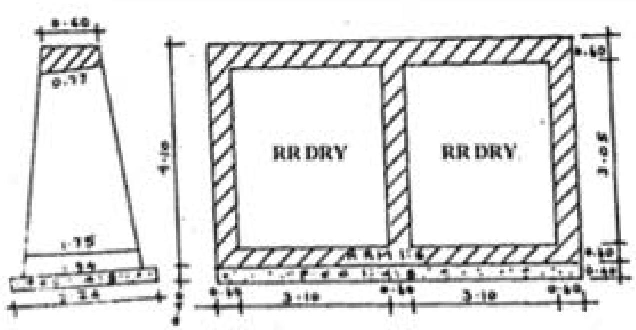


Fig. 28: Calculation of Quantities (Conventional)

(C) Recommended Section of Retaining Walls in Steep Valley Slopes – Natural Valley Slope 55°

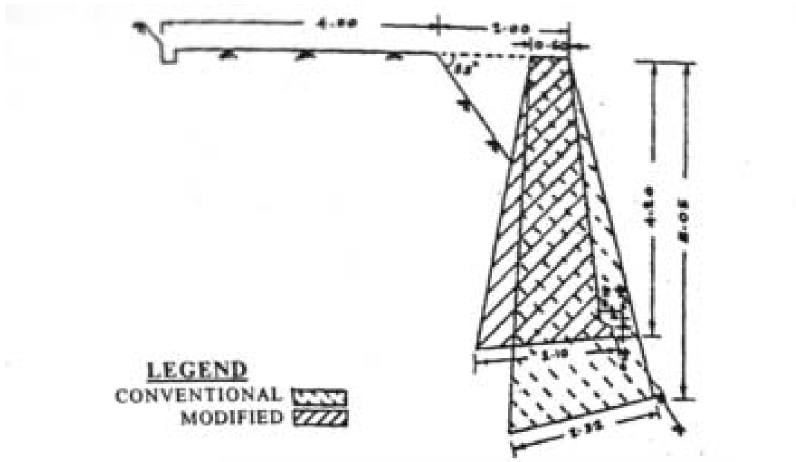


Fig. 29

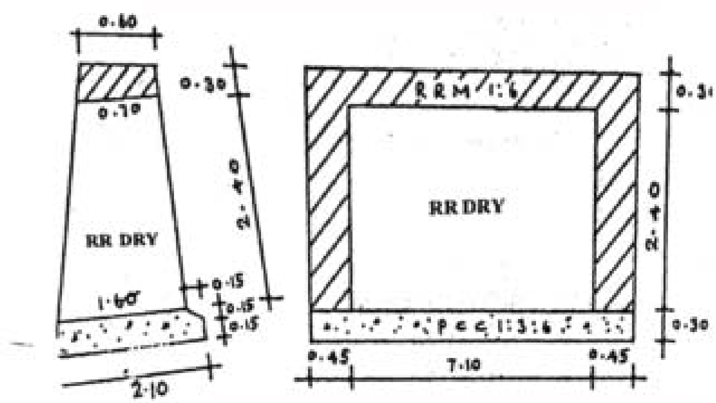


Fig. 30: Calculation of Quantities (Modified)

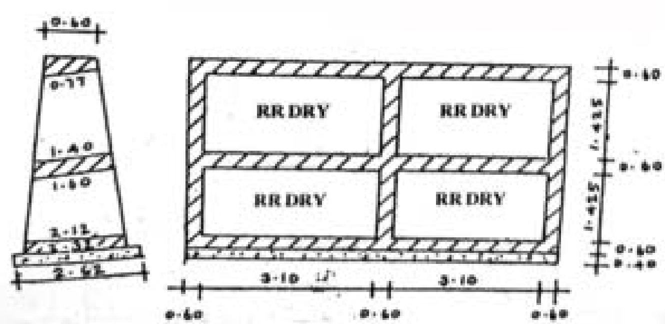


Fig. 31: Calculation of Quantities (Conventional)

(D) Recommended Section of Retaining Walls in Steep Valley Slopes – Natural Valley Slope 60^0

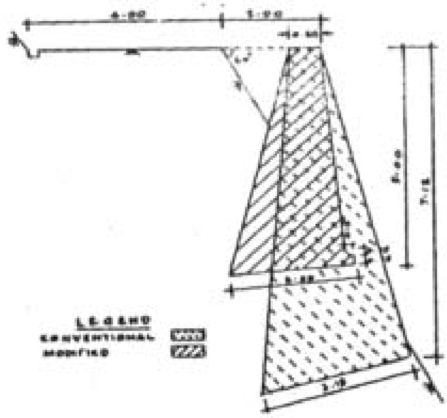


Fig. 32

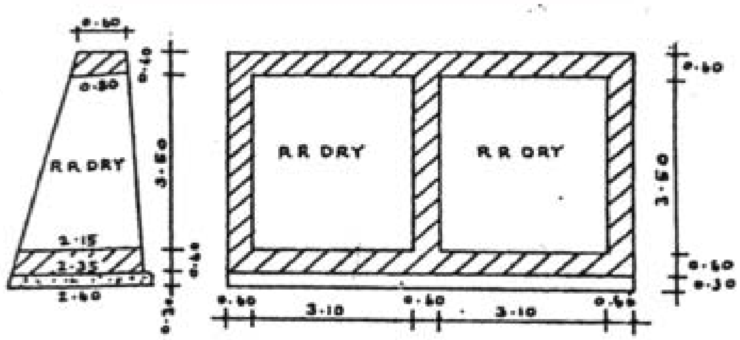


Fig. 33: Calculation of Quantities (Modified)

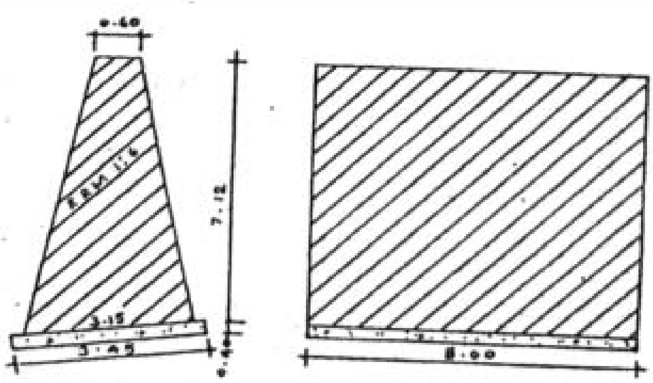


Fig. 34: Calculation of Quantities (Conventional)

18. CANTILEVER TYPE RCC RETAINING WALL

18.1 The cantilever R.C.C retaining wall resists the horizontal earth pressure as well as other vertical pressures by way of bending of various components acting as cantilever. The common form of cantilever Retaining wall is the inverted T-shaped wall, shown in fig-35.

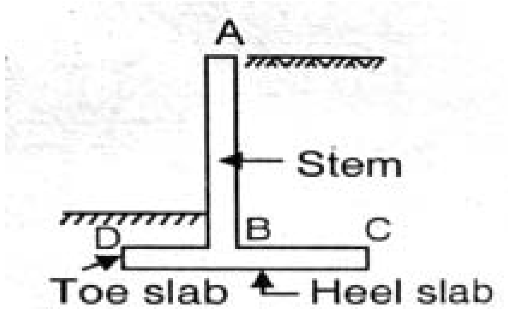


Fig 35

Cantilever RCC retaining wall can be preferred for 5 mtr to 6 mtr height over the Gravity retaining wall.

18.2 **Stability of cantilever Retaining wall:-**

A cantilever R.C.C retaining wall is subjected following forces:

(a) Weight w_1 of the stem AB.

(b) Weight w_2 of the base slab DC

(c) Weight w_3 of the column of soil supported on heel slab BC.

(d) Horizontal force Pa, equal to active earth pressure acting at H/3 above the base.

The following are the mode of failure of retaining wall. These criteria are to be satisfied for safe design.

18.2.1 **Overturning about the toe.**

Overturning moment due to active earth pressure at toe

$$M_o = P_a \times \frac{H}{3} = \frac{1}{2} KaYH^2 . \frac{H}{3}$$

$$= \frac{K_a Y H^3}{6}$$

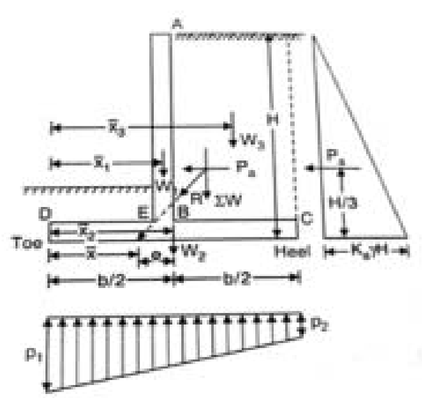


Fig 36

Resisting moment is due to weightw_1, w_2& $w_3$and neglecting passive earth pressure because of soil weight above toe slab.

$M_R = w_1x_1 + w_2x_2 + w_3x_3$

Factor of safety due to overturning $F_o = \dfrac{M_R}{M_o}$

Factor of safety against overturning of wall should not be less than 2.

18.2.2 **Sliding.** The Horizontal force Pa tends to slide the wall away from the fill. The tendency to resist is achieved by the friction at the base.

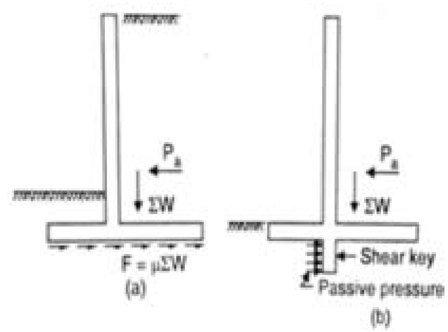


Fig 37

The force of Resistance $F = \mu \Sigma W$

μ= Coefficient of friction between soil & concrete.

ΣW = Sum of vertical forces

Factor of safety due to sliding $F_s = \dfrac{\mu \Sigma W}{Pa}$

The factor of safety against sliding should not be less than 1.5.

If wall is found to be unsafe against sliding, shear key below the base should be provided.

The following value of ì may be adopted for different soil.

Sr No.	**Type of soil**	μ
a.	Coarse grained soil without silt	0.55
b.	Coarse grained soil with silt	0.45
c.	Silt	0.35

18.2.3 **Soil Pressure Distribution**

If ΣW is the sum of vertical forces and Pa is the horizontal active earth pressure, the resultant R will strike the base slab at a distance of e from the middle point of the base.

Net moment at the toe.

$$\Sigma M = w_1x_1 + w_2x_2 + w_3x_3 - P_a\frac{H}{3}$$

Then Distance of point of application of resultant

$$X = \frac{\Sigma M}{\Sigma W}$$

Hence $$e = \frac{b}{2} - X$$

The Pressure distribution at toe and heel of base slab as follows.

$$P_1 = \frac{\Sigma W}{b}\left(1 + \frac{6a}{b}\right) \text{ at Toe edge}$$

$$P_2 = \frac{\Sigma W}{b}\left(1 - \frac{6a}{b}\right) \text{ at Heel edge}$$

18.2.4 **Bending failure.** There are three distinct part of inverted T shaped cantilever retaining wall, the stem AB, the heel slab BC and toe slab DE.

The stem AB will bend as cantilever, thereby tensile will be towards the backfill and the critical section will be at B, so cracks may occur at the inner face,If it is not properly reinforced.

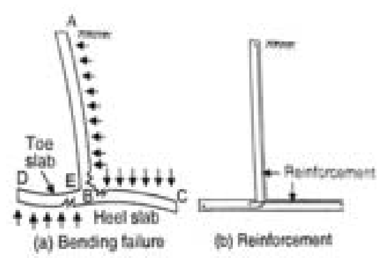


Fig 38

The heel slab will have net pressure acting downwards, and will bend as cantilever having tensile face upwards the critical section will be at B, where crack may occur if it is not reinforced properly at the upper face as shown in figure.

The net pressure on toe slab will act upward, and hence it must be reinforced at bottom face.

The thickness of stem, heel, slab and toe slab must be sufficient to withstand

Compressive stressed due to bending.

19. PRINCIPLES OF DESIGN OF CANTILEVER TYPE RCC RETAINING WALL

The design of cantilever retaining wall consisting of following

(a) Fixation of base width
(b) Design of stem
(c) Design of hill slab
(e) Design of toe slab

19.1 **Fixation of base width**

19.1.1 **Determination of base width considering Stres.** The base width of the retaining wall so chosen that the resultant of the forces of retaining wall remain within middle third and ratio of length of toe slab to the base width should be such that pressure P_1 at toe does not exceed the safe bearing capacity of soil.

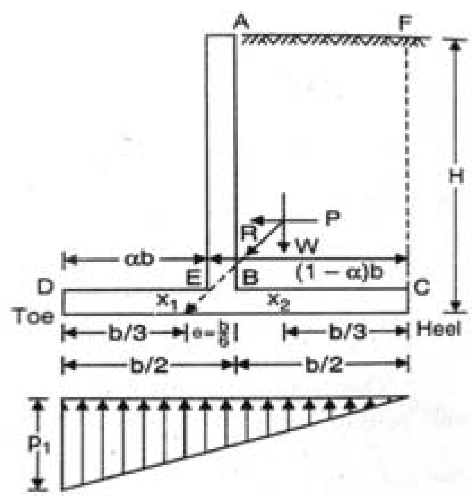


Fig 39

(a) In case of No sloping backfill with surcharge angle β = 0

$$\text{Base width (b)} = 0.95\,H\sqrt{\frac{Ka}{(1-\alpha)(1+3\alpha)}}$$

(b) In case of sloping backfill with surcharge angle â

$$\text{Base width } b = H\sqrt{\frac{Ka\cos}{(1-\alpha)(1+3\alpha)}}$$

Where- b = base width of base slab

H = Height of Retaining wall

$Ka = \frac{1-\sin\Phi}{1+\sin\Phi}$

α = Ratio of length Toe slab to the total base width

$\alpha = 1 - \frac{q_o}{2.2YH}$ (in case of No surcharge)

$\alpha = 1 - \frac{q_o}{2.2YH}$ (sloping backfill with surcharge angle â)

q_o = Safe bearing capacity of soil

Y = unit weight of soil .

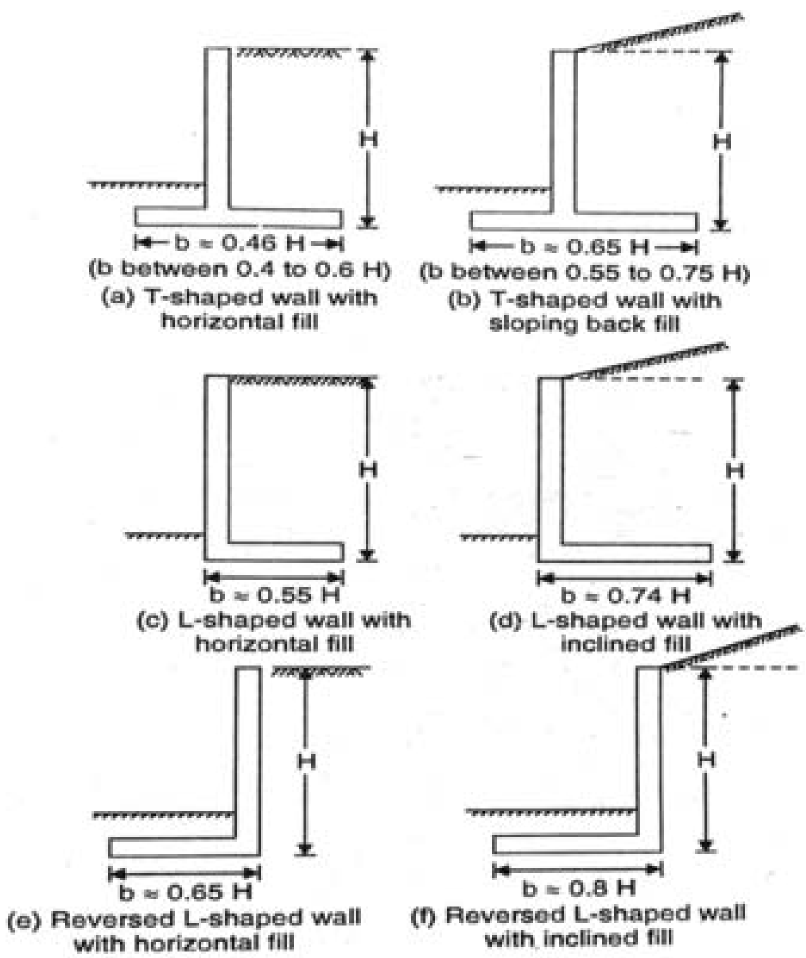


Fig 40

19.1.2 **Determination of base width form the consideration of sliding –**

Base width is also determined considering sliding criteria.

The factor of safety against sliding should be minimum 1.5.

$$F.S = \frac{\pi W}{Pa}$$

$$W = 1.1\ YbH\ (1\text{-á})$$

$$Pa = Ka\ Y\ \frac{H^2}{2}$$

Base width $b = \frac{0.7\ H}{1-\alpha} \quad \frac{Ka}{\mu}$

Ka, α can be determined as per equation given previously.

19.2 **Design of Stem**

The vertical stem AB is designed as a cantilever triangular loading

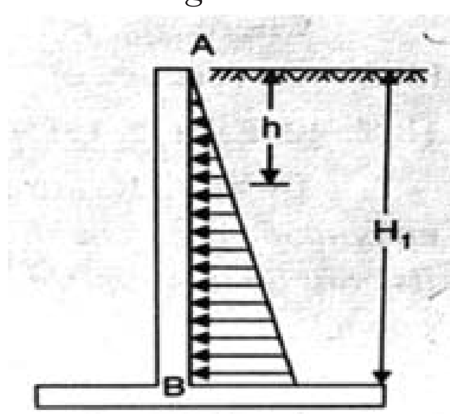


Fig 41

(a) At any section h below the top point A, the force = ½ $KaYh^2$

(b) It's bending moment about the section $= Ka \frac{Yh^2}{b}$

(c) The thickness at B is maximum

(d) The thickness at point A will be minimum, its vary from 20 to 30 cm depending upon the height of wall.

(e) Reinforcement is provided towards the inner face of back fill i.e. towards side of backfill.

(f) The reinforcement towards the top of stem can be curtailed since Bending Moment varies as h^3.

(g) Distribution reinforcement is provided @.15% of the area of cross section along the length of Retaining wall at inner face.

(h) Similarly temperature reinforcement @ 0.15% of area of cross section is provided in both horizontal as well as vertical direction, at the outer face of stem.

19.3 **Design of heel slab.** The heel is also to be designed as a cantilever. It has both downward pressure due to weight of soil & self-weight and upward pressure due to soil reaction. However net pressure act downward and hence reinforcement is provided at upper face BC.

19.4 **Design of Toe slab.** Neglecting the weight of the soil above it, the toe slab will bend upwards as a cantilever due to upward soil reaction.

(a) Hence the reinforcement is provided at bottom face.

(b) Normally the thickness of both toe slab and heel slab is kept the same determined on the basis of greater of the cantilever bending moment.

19.5 **Depth of foundation.** The height H_2 of retaining wall, above ground level is fixed on the basis of height of backfill to be retained.

The depth of foundation y should be such that good quality of soil to bear the induced pressure is available.

Minimum depth of foundation as per Rankine's formula –

$$Y_{min} = \frac{q_0}{y}\left[\frac{1-\sin\Phi}{1+\sin\Phi}\right]$$

Where-q_o = safe bearing capacity of soil.

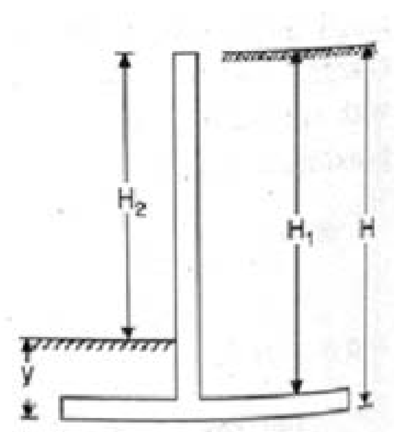


Fig 42

DESIGN OF CANTILEVER RETAINING WALL WITH HORIZONTAL BACKFILL AND TRAFFIC LOAD

Example – Design of cantilever RCC Retaining wall for road for the following requirement :

1. Height of wall from bottom of base to top of stem = 6m
2. Superimposed load due to traffic = 18 KN/m^2
3. Unit weight of earth fill – 18 KN/m^3
4. Angle of Internal friction for fill material = 30^0
5. Allowable bearing pressure on ground = 160 KN/m^2
6. Co-efficient of friction between concrete and ground = 0.4

Also provide a parapet wall 0.5 mtr height on the top of stem. Use M.20, concrete Fe 415 steel.

Solution-

I. **Design Constant –**

For M-20 concrete, and Fe- 415 steel reinforcement

σ_{abc} = 7 N/mm^2 $\quad$ σ_{st} = 230 N/mm^2

M = 13.33 $\quad$ K_e = 0.289

l_e = 0.904 $\quad$ R_e = 0.914

II. **Dimension of base –**

- Assume that horizontal force Q = 2 KN/m length of parapet wall well act because person standing near the parapet.
- Due to surcharge/live load equivalent height of fill given by

$$h_e = \frac{W}{r} = \frac{18}{18} = 1m$$

$$\text{then}H_2 = 6 + 1 = 7m$$

$$\alpha = 1 - \frac{q_0}{2.2YH} = 1 - \frac{160}{2.2\times18\times7} = 0.42$$

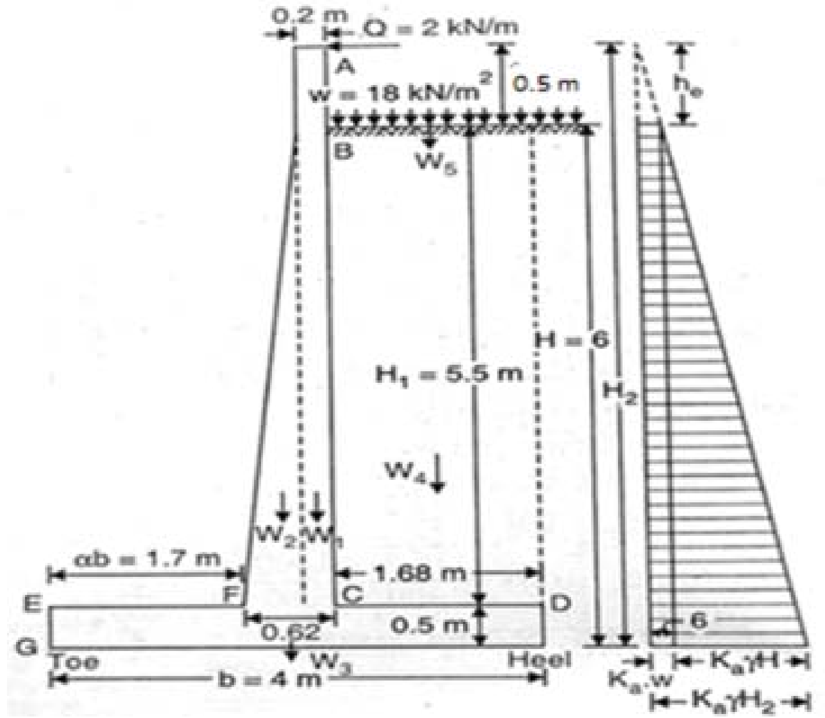


Fig 43

$$\text{width of base } b = 0.95\,H\sqrt{\frac{Ka}{(1-\alpha)\,(1+3\alpha)}}$$

$$Ka = \frac{1-\text{Sin } 30^0}{1+\text{Sin } 30^0} = 0.33 = \frac{1}{3}$$

$$b = 0.95 \times 7 = \sqrt{\frac{1}{3(1-0.42)\,(1+3\times0.42)}} = 3.36 \text{ m}$$

However, keep min^m base width between 0.5 to 0.6 H2

Hence adopted b = 4 m

Note- for the safer side 4m base width adopted, if soil Bearing capacity is better in that case it can also be taken as 3.5m.

Length of toe slab = αb = 0.42 × 4 = 1.68 m

Adopt αb = 1.70 m

For preliminary calculation let thickness of base = 0.5 m

III. **Thickness of stem -**

Height of stem H_1 = 6 – 0.5 = 5.5 m

Total Bending moment at bottom of stem at C

$$M = Q (H_1 + 0.5) + Ka\ w \frac{H_1^2}{2} + Ka \frac{H_1^2}{2}$$

$$= 2 (5.5 + 0.5) + \frac{1}{3} \times 18 \times \frac{5.5^2}{2} + \frac{1}{3} \times \frac{18}{} \times \frac{5.5^2}{6}$$

$$= 12.00 + 90.75 + 166.35 = 269.1 \times 10^6 \text{ N-mm.}$$

$$d = \sqrt{\frac{269.1 \times 10^6}{1000 \times 0.914}} = 543 \text{ mm}$$

adopt d = 560 mm

Considering 60 mm effective cover, total thickness of stem

D = 560 + 60 = 620 mm

Reduce total thickness at B = 200 mm and B to A continue 200 mm thickness constant.

IV. **Stability of wall –**

Length of heel slab CD = 4 – 1.7 -0.62 = 1.68 m

Vertical forces –

Let W_1 = weight of rectangular portion of stem = 0.2 × 6× 1 × 25 = 30 KN

W_2 = weight of triangular portion of stem = ½ ×0.42 5.5× 1 × 25 = 28.9 KN

W_3 = weight of base slab = 0.5 × 4 × 1 × 25 = 50 KN

W_4 = weight of soil above heel slab = 1.68 × 5.5 × 1 × 18 = 166.40 KN

W_5 = Total super imposed road traffic load over heel slab = 1.68 × 1 × 18 = 30.24 KN

Horizontal forces

Pa_1 = Total earth pressure at base $Pa_1 = KaY \frac{H^2}{2} = \frac{1}{2} \times 18 \times \frac{6^2}{2} = 108$ KN

Q_1 = Horizontal force due to parapet wall = 2KN

Pa_2 = Horizontal pressure caused by live load = $K_a wH$

$= \frac{1}{3} \times 18 \times 6 = 36$ KN

SI No.	Force Designation	Force (KN)		Level Arm (Mtr)	Moment at toe (KN-M)	
		Vertical	Horizontal		Resisting	Overturning
1.	W_1	30		2.22	66.60	
2.	W_2	28.9		1.98	57.17	
3.	W_3	50		2.00	100.00	
4.	W_4	166.4		3.16	525.82	
5.	W_5	30.2		3.16	95.56	
6.	Pa_1		108.00	2		216.00
7.	Pa_2		36.00	3		108.00
8.	Q_1		2.00	6.5		13
	Total	ΣW = 305.5	ΣP = 146		ΣM_R = 845.15	ΣM_o = 337

Factor of safety against overturning = $\frac{\Sigma M_R}{\Sigma M_o} = \frac{845.15}{337} = 2.50 > 2$

Hence safe against overturning

Factor of safety against sliding = $\frac{\mu \Sigma W}{\Sigma P} = \frac{0.4 \times 305.5}{146} = 0.84 < 1.5$

Hence unsafe therefore special shear key is to be provided to make it safe.

Pressure Distribution –

Point of application of resultant = X = $\frac{\Sigma M}{\Sigma W}$

$$= \frac{845.15-337}{305.5} = 1.66 \text{ m}$$

Eccentricity e = $\frac{b}{2}$ = X $\frac{4}{2}$ – 1.66 = 0.34 m < $\frac{b}{6}$ = $\frac{4}{6}$ = 0.67

Hence no tension will develop

Pressure at toe $P_1 = \frac{\Sigma W}{b} = \left(1+ \frac{6e}{b}\right) = \frac{305.5}{4} \left(1 + \frac{6\times 0.34}{4}\right)$

P_1 = 115.3 KN/m² < 160 KN/m²

Pressure at heel $P_2 = \frac{\Sigma W}{b} = \left(1- \frac{6e}{b}\right) = \frac{305.5}{4} \left(1 - \frac{6\times 0.34}{4}\right)$

Pressure at heel P_2= =

= 37.43 KN/m² < 160 KN/m²

Pressure P at Junction with stem and toe slab

$$= 115.3 - \frac{115.3-37.43}{4} \times 1.70 = 82.20 \text{ KN/m}^2$$

Pressure P' at Junction with stem and heel slab

$$= 115.3 - \frac{115.3-37.43}{4} \times 2.32 = 70.13 \text{ KN/m}^2$$

5. **Design of toe slab –** If the weight of soil above the toe slab is neglected thus only two forces acting on it

I. Upward soil pressure

II. Downward weight of slab

Downward weight of slab per unit area = 0.5 × 1 × 25 = 12.5 KN/m²

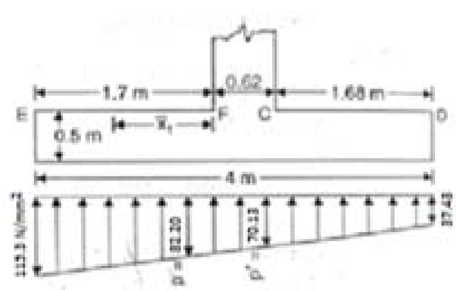


Fig 44

Net pressure intensity at toe edge E = 115.3 – 12.5 = 102.8 KN/m^2

Net pressure intensity at point F = 82.2 – 12.5 = 69.7 KN/m^2

Total force = shear force at F = ½ (102.8 + 69.7) × 1.7 = 146.6 KN

$$\text{C.B of force from point F} = X_1 = \left(\frac{69.7+2\times 102.8}{69.7+102.8}\right) \times \frac{1.7}{3} = 0.904 \text{ m}$$

B.M at point F = 146.6× 0.904 = 132.52 KN-m

=132.52× N-mm

$$d = \frac{132.52\times 10^6}{1000\times 0.914} = 380.77 = 383 \text{ mm}$$

Take d = 440 mm and total depth 500mm with effective cover 60 mm

Reduce the total depth to 200 mm at the edge.

$$\text{Area of reinforcement } A_{st} = \frac{132.52\times 10^6}{230\times 0.904\times 440} = 1449 \text{ mm}^2$$

20 mm ϕ bar @ 120 mm c/c may be provided

$$\text{Total } A_{st} = \frac{1000}{120}\times 314 = 2616.68 \text{ mm}^2$$

$$\text{Shear stress } \tau_v = \frac{148.07\ 10^3\times 100}{1000\times 440} = 0.337 \text{ N/mm}^2$$

$$\%\text{ of steel} = \frac{A_3}{bd}\times 100 = \frac{2616.68\times 100}{1000\times 440} = 0.594\ \%$$

From table t_c = 0.31 N/mm^2

$\tau_v > \tau_c$ which is slightly unsafe

Hence reinforcement increases to 0.7 %

$$\text{Then } A_{st} = \frac{0.7\times bd}{100} = \frac{0.7\times 1000\times 440}{100} = 3080 \text{ mm}^2$$

20 mm ϕ bar spacing $s = \dfrac{1000 \times 314.10}{3083} = 101 = 100$ mm/c

Hence Finally 20 mm ϕ bar @ 100 mm c/c provided

Distribution steel:-

$$\text{Distribution steel} = \frac{0.12}{100}\left(\frac{460+200}{2}\right) \times 1000 = 396\ mm^2$$

using 8 mm ϕ bars having $A_p = 50.3\ mm^2$

$$\text{spacing} \quad s = \frac{1000\ 50.3}{396} = 127\ mm$$

hence provided 8 mm ϕ bar @ 120 mm c/c

6. **Design of heel slab** –

There are 4 forces are acting on it

I. Downward weight of soil = 1.68 × 1 × 5.5 × 18 = 166.32 KN
Acting at 0.84 m from C

II. Live load/Surcharge of traffic = 18 × 1 × 1.68 = 30.24 KN
Acting at 0.84 m from C

III. Weight of heel slab = 1.68 × 0.5 × 25 = 21 KN
Acting at 0.84 m from C

IV. Upward soil reacting = ½ (70.13 + 37.43) × 1.68 = 90.35 KN

$$\text{Acting at} \left(\frac{70.13+2\ 37.43}{70.13+37.43}\right) \times = \frac{1.68}{3}\ 0.75 \text{ m from c}$$

Total shear force at cF = 166.32 + 30.24 +21 – 90.35

= 127.21 KN

BM at point C = 217.56 ×0.84 - 90.35×0.75 = 114.98 KN-m

= 114.98× N-mm < 133.9 ×N-mm

Hence keep thickness of slab as that of toe slab

d = 440 mm D = 550 mm effective cover = 60 mm

Reduce total depth to 200 mm at edges

$$A_{st} = \frac{114.98\ 10^6}{230 \times 0.904 \times 440} = 1256\ mm^2$$

Using 20 mm ϕ bar spacing $= \dfrac{1000 \times 314}{1256} = 250$ mm c/c

$$\iota_v = \frac{127.21 \times 10^2}{1000 \times 440} = 0.289\ N/mm^2$$

From table % reinforcement needed to get $\iota_v = 0.29\ N/mm^2$ is equal to 0.49%

$$A_{st} = \frac{0.49bd}{100} = \frac{0.49 \times 1000 \times 440}{100} = 2156\ mm^2$$

Hence finally provided 20 mm ϕ bar @ 140 mm c/c

$$\text{Actual } A_{st} = \frac{1000 \times 314}{140} = 2242.8\ mm^2$$

Distribution steel $= \dfrac{0.12bd}{100} = \dfrac{0.12}{100}\left(\dfrac{460 + 200}{2}\right) \times 1000 = 396\ mm^2$

8 mm ϕ bar @ spacing $s = \dfrac{1000 \times 50.3}{396} = 127$ mm c/c

8 mm ϕ bar @ 120 mm c/c provided.

7. **Reinforcement in the stem –**

Area of steel $A_{st} = \dfrac{269.10 \times 10^6}{230 \times 0.90 \times 560} = 2321\ mm^2$

Using 20 mm ϕ bars. $A_\phi = 314.10\ mm^2$

Then spacing $s = \dfrac{1000 \times 314.1}{2320} = 131$

Provide 20 mm ϕ bar @ 120 mm c/c may be provided

Total Reinforcement $= \dfrac{1000 \times 314.1}{120} = 2618\ mm^2$

Bend all the bars in the toe slab to serve as the reinforcement there However in order to make toe slab safe in shear required A_{st} =3080 mm^2&spacing = 100 mm

Hence finally 20 mm x bar @ 100 mm c/c provided

Total shear force at point C = $Q + KaW + KaY \frac{H_1^2}{2}$

$$= 2 + \frac{1}{3} \times 18 \times 5.5 + \frac{1}{3} \times 18 \times \frac{5.5^2}{2}$$

$$= 125.75 \text{ KN}$$

$$\iota_v = \frac{125.75 \times 10^2}{1000 \times 560} = 0.225 \text{ N/mm}^2$$

$$\% \text{ of steel} = \frac{100 \text{ Ast}}{bd} = \frac{100 \times 3080}{1000 \times 560} = 0.55\%$$

For .55% of steel, ι_c = 0.31 N/mm^2> 0.225 N/mm^2

Hence safe

Now curtail the reinforcement between point B and C

At depth h = 0.65 H_1 = 0.65 × 5.5 = 3.575 m below point B

Check whether 50% (half of bar) reinforcement can be curtailed or not

$$BM = Q(h+0.5) + KaW\frac{h^2}{2} + KaY\frac{h^2}{6}$$

$$= 2(3.575+.5) + \frac{1}{3} \times 18 \times \frac{3.575}{2} + \frac{1}{3} \times 18 \times \frac{3.575^2}{6}$$

= 8.15 + 38.34 + 45.69 = 92.18 × N-mm

$$d = 140 + \left(\frac{560 - 140}{5.5}\right) \times 3.575 = 413 \text{ mm}$$

$$A_{st} = \frac{92.18 \times 10^6}{230 \times 0.904 \times 413} = 1073 \text{ mm}^2 < 1540 \text{ mm}^2$$

(required) (Provided)

Hence half the bars can be curtailed at this depth. However bar should be extended up to 12 ϕ = 240 mm or d = 413 mm beyond this point

Hence h = 3.575 – 0.413 = 3.162 m

Hence curtail half the bar at 3.10 m below point B.

Similarly, half of bar can be curtain for remaining part at

h = 0.65 × 3.575 = 2.32 m below point B

$$BM = 2\,(2.32 + 0.5) + \frac{1}{3} \times 18 \times \frac{2.32^2}{2} + \frac{1}{3} \times 18 \times \frac{2.32^3}{6}$$

= 5.64 + 16.15 + 12.49 = 35.28 KN-m = 34. 28 × 10^6 N-mm

$$d = 140 + \left(\frac{560\text{-}10^6}{5.5}\right) \times 2.32 = 317 \text{ mm}$$

$$A_{st} = \frac{34.28 \times 10^6}{230 \times 0.904 \times 317} = 520 \text{ mm}^2 < 770 \text{ mm}^2$$

So half bar can be curtailed at below B h = 2.32 – 0.317 = 2.00 m

Rest of the bar will be continued up the top of parapet.

Distribution Reinforcement –

$$\text{Area of distribution steel} = \frac{0.12}{100} \times \left(\frac{620 + 200}{2}\right) \times 1000$$

= 492 mm^2

Using 10 mm ϕ bar, $A_{\varnothing}$ = 78.5 mm^2 spacing = $\frac{1000 \times 78.5}{492}$ = 159 mm

Hence 10 mm ϕ bar @ 150 mm c/c provided

Temperature Reinforcement –

Total area = 492 mm^2

Provide 10 mm ϕ bar @ 300 mm c/c both way on the outer face.

8. **Design of shear key –** The wall is unsafe in sliding hence shear key is to be provided.

Passive pressure intensity will develop in front of shear key to resist sliding. This pressure intensity depends upon the soil pressure P just in front of shear key.

$P_p = K_p .P \qquad K_p = \dfrac{1+\sin\Phi}{1-\sin\Phi} = 3$

$P_p = 3 \times 82.20 = 246.6\ KN/m^2$

Then total passive force $\mathbf{P_p} = P_p a = 246.6\ a$

Taking a = 600m = 0.6 m

$\mathbf{P_p} = 246.6 \times 0.6 = 147.96$ KN

Weight of soil between the base and

level IJ = $0.6 \times 4 \times 1 \times 18 = 43.2$ KN

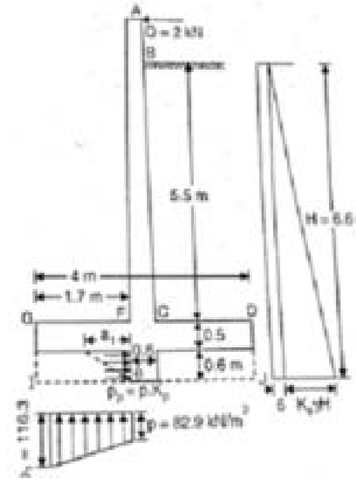


Fig 45

Total vertical load $\Sigma W \quad = 305.5 + 43.2$ KN

$\qquad = 348.7$ KN

Hence factor of safety against sliding = $\dfrac{\mu\Sigma + P_p}{\Sigma P_a}$

Total sliding force at bottom level of key

$\Sigma P_a = 2 + \left(\frac{1}{3}\ 18 \times 6.6 + \frac{1}{3} x\ 18 \times 6.6^2\right) = 172.3$ KN

Hence $\qquad F.S = \dfrac{\mu\Sigma W + P_p}{\Sigma P_a} = \dfrac{04 \times 348.7 + 147.96}{172.3} = 1.66 > 1.5$

Hence safe in sliding

The Shear key is to be casted monolithically with base.

Construction Joint – A construction joint in the form of key is to be provided at the Junction of stem with the base slab the width of key is kept equal to

$\dfrac{d}{4} = \dfrac{560}{4} = 140mm$

Note- Reinforcement Detail of RCC retaining wall shown in figure.

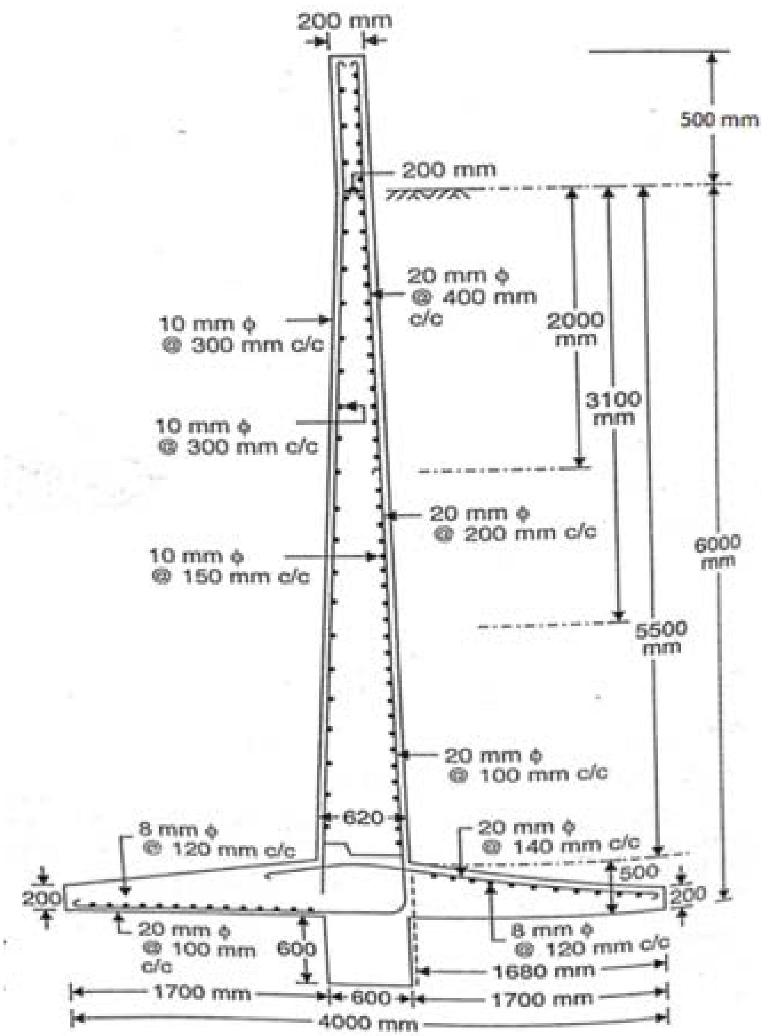


Fig 46

Design of cantilever retaining wall with horizontal backfill :–

I. Height of wall from the bottom of base to top of stem = 5m

II. Super imposed load due to road traffic = 18 KN/m^2

III. Unit weight of fill = 18 KN/m^3

IV. Angle of internal friction for fill material = 30^0

V. Allowable bearing pressure on ground = 160 KN/m^2

VI. Coefficient of friction between concrete and ground = 0.4

VII. Also provide a parapet wall 0.5 m high on the top of stem use M-20 concrete and Fe-415 steel.

Solution –

I. **Design constant –**

For M-20 concrete and Fe-415 steel

σ_{abc} = 7 N/mm^2 σ_{st} =230 N/mm^2 m = 13.33

K_c = 0.289 J_c = 0.904 & R_c = 0.914

II. **Dimension of base –**

Assume that a horizontal force Q= 2 KN/m length of parapet wall will act because of person standing near parapet wall.

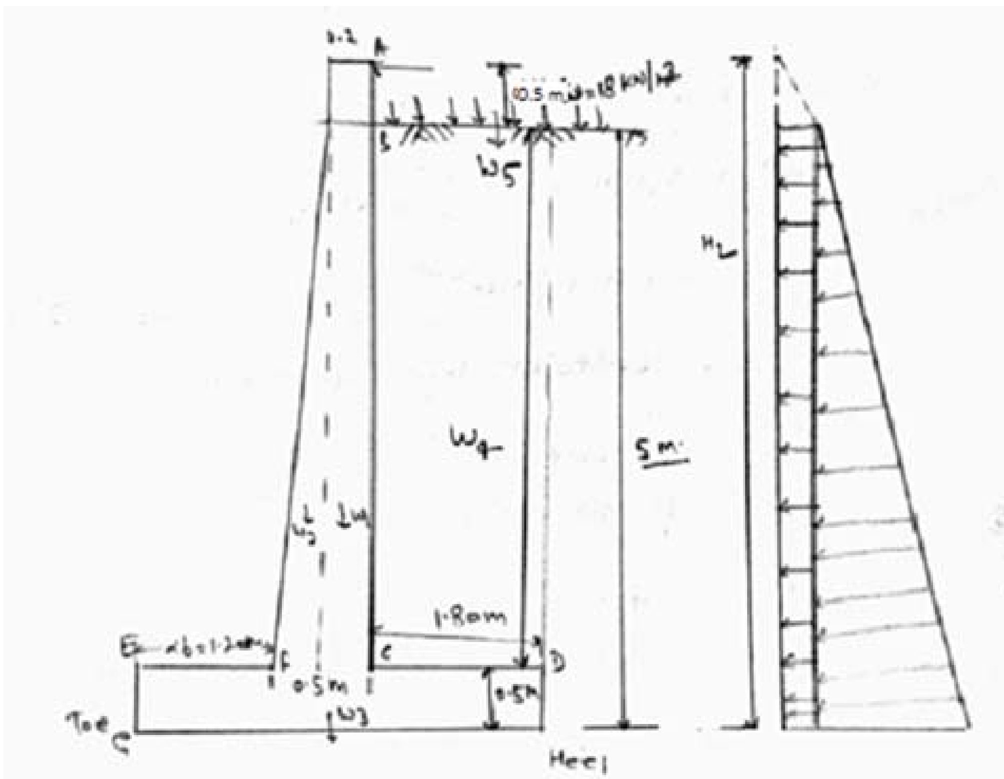


Fig 47

Due to surcharge equivalent height of fill given by-

$$H_e = \frac{W}{Y} = \frac{18}{18} = 1\text{m}$$

Hence in determining value of base width b and á use height H_2 = 5 + 1 =6.00 m

$$\alpha = 1 - \frac{160}{2.2 \times 18 \times 6} = 0.33$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1}{3}$$

$$\text{Width of base } b = 0.95 \times H_2 \sqrt{\frac{Ka}{(1-\alpha)\ (1+3\alpha)}}$$

$$= 0.95 \times 6\sqrt{\frac{1/3}{(1-0.33)\ (1+3 \times 0.33)}} = 2.85$$

Now base width adopted 3.5 m (As b shall be bet^n 0.5 to 0.6 H_2)

Length of toe slab = áb = 0.33 × 3.5 = 1.155

Adopt length of toe slab = 1.20 m

Let take thickness of base slab = 0.5 m = 500 mm

3. **Thickness of stem –**

Height H_1 = 5 – 0.5 = 4.5 m

The total bending moment at point C

$$Bm = Q(H_1 + 0.5) + KaW \frac{H_1^2}{2} + KaY \frac{H_1^3}{6}$$

= 2 (4.5 +0.5) + × 18 × + × 18 × = 10 + 60.75 + 91.125

= 161.875 KN-m = 161.87 × N-mm

$$d = \frac{161.87 \times 10^6}{1000 \times 0.914} = 421 \text{ mm}$$

D = 421 + 60 = 487 mm

D = 500 mm effective cover = 60 mm

d = 500 – 60 = 440 mm

Reduce total thickness 200 mm at point B.

So that effective depth at B = 200 – 60 = 140 mm

Continue uniform thickness of 200 mm from B to A.

4. **Stability of wall –**

Length of heel slab CD = 3.5 – 0.5 – 1.20 = 1.8 m

Vertical forces –

W_1 = weight of rectangular portion of stem = 0.2 × 5.0× 1 × 25 = 25 KN

W_2 = weight of triangular portion of stem = ½ × 0.3 × 4.5 × 25 = 16.88 KN

W_3 = weight of base slab = 0.5 × 3.5 × 1 × 25 = 43.75 KN

W_4 = weight of soil above heel slab = 1.80 × 4.5 × 1 × 18 = 145.8 KN

W_5 = total super imposed traffic load over heel slab= 1.80 × 1 × 18 = 32.4 KN

Horizontal forces –

Pa_1 = Total earth pressure at base = KaY = × 18 × = 75 KN

Q_1 = Horizontal wall force due to parapet = 2 KN

Pa_2 = Horizontal pressure caused by live load = KaWH = × 18 × 5 = 30 KN

Sr No.	Force Designation	Force (KN)		Level Arm (Mtr)	Moment at toe (KN-M)	
		Vertical	Horizontal		Resisting	Overturning
1.	W_1	25	-	1.60	40	-
2.	W_2	16.88	-	1.40	23.63	-
3.	W_3	43.75	-	1.75	76.56	-
4.	W_4	145.8	-	2.60	379.08	-
5.	W_5	32.4	-	2.60	84.24	-
6.	Pa_1	-	75	1.67	-	125.25
7.	Pa_2	-	30	2.5	-	75.00
8.	Q_1	-	2	5.5	-	11
	Total	ΣW= 263.83	ΣP = 107.00		ΣM_R = 603.51	ΣM_o = 211.25

Factor of safety against overturning = $\frac{\Sigma M_g}{\Sigma M_0} = \frac{603.51}{212.25}$ = 2.84> 2

Hence safe against overturning

Factor of safety against sliding = $\frac{\mu \Sigma W}{\Sigma P_\alpha} = \frac{0.4 \;\; 263.83}{107}$

= 0.986 <1.5 unsafe

Unsafe, therefore special shear key is to be provided to make it safe.

Pressure Distribution –

Point of application of resultant $X = \frac{\Sigma M}{\Sigma W}$

$$X = \frac{603.51-211.25}{263.83} = 1.48 \text{ m}$$

Eccentricity $e = \frac{b}{2} - X = \frac{3.5}{2} \;\; 1.48 = 0.27 < \frac{b}{6} = 0.58$

$e < \frac{b}{6}$ Hence no tension will develop

Pressure at toe P1 = $\frac{\Sigma W}{b} = \left(1+\frac{6e}{b}\right) = \frac{263.83}{3.5}\left(1+\frac{6 \times 0.27}{3.5}\right)$

= 110.27 KN/m^2< 160 KN/m^2

Hence safe

Pressure at heel P2 = $\frac{\Sigma W}{b} = \left(1+\frac{6e}{b}\right) = \frac{263.83}{3.5}\left(1+\frac{6 \times 0.27}{3.5}\right)$

= 40.49 KN/m^2< 160 KN/m^2

Pressure P at Junction with stem and toe slab –

$$= 110.27 - \left(\frac{110.27-40.49}{3.5}\right) \times 1.20 = 86.34 \text{ KN/m}^2$$

Pressure P' at Junction with stem and heel slab

$$= 110.27 - \left(\frac{110.27-40.49}{3.5}\right) \times 1.70 = 76.37 \text{ KN/m}^2$$

5. **Design of toe slab –**

If the weight of soil above the toe is neglected, in this case only two forces acting on it.

I. Upward soil pressure

II. Downward weight of slab

Downward weight of slab per unit area = 0.5 × 1 × 1 × 25 = 12.5 KN/m²

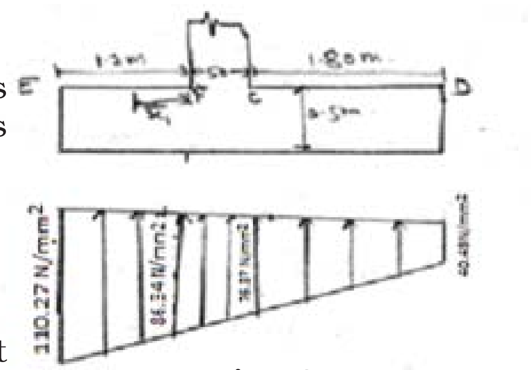


Fig 48

Net pressure intensity of toe edge E = 110.27 – 12.5 = 97.77 KN/m²

Net pressure intensity at point F = 86.34 – 12.5 = 73.84 KN/m²

Total force = shear force from point F = ½ (97.77 +73.84) × 1.20

= 102.97KN

$$\text{C.G of force from point F X} = \left(\frac{73.84+2 \times 97.77}{73.84+ 97.77}\right) \times \frac{1.20}{3}$$

= 0.627 m from point F.

B.M at point F = 102.97× 0.627 = 64.56 KN-m

= 64.56 × 10^6 N-mm

$$d = \sqrt{\frac{64.56 \times 10^6}{1000 \times 0.914}}$$

$$m = Qbd^2$$

$$d = \sqrt{\frac{m}{Qb}}$$

d = 265.77< 440 mm taken

with 60 mm effective cover total depth adopted 500 mm

Reduce the total depth to 200 mm at the edge

Area of reinforcement $A_{st} = \dfrac{64.56 \times 10^6}{230 \times 0.904 \times 440} = 706 \text{ mm}^2$

20 mm Φ bar 150 mm c/c may be provided-

Total $A_{st} = \dfrac{1000 \times 314}{150} = 2093 \text{ mm}^2$

Shear stress $\iota_v = \dfrac{104.08 \times 10^2}{1000 \times 440} = 0.236 \text{ N/mm}^2$

% of steel = × 100 = $\dfrac{2093 \times 100}{1000 \times 440} = 0.475$ %

From table % of steel = 0.475

Then $\iota_c = 0.30 - \dfrac{0.3-0.22}{0.25} \times (05-0.475)$

%	t_c
0.25	0.22
0.50	0.30

$\iota_c = 0.292 \text{N/mm}^2$

$\iota_v < \iota_c$ hence safe against sliding

Hence 20 mm Φ bar @ 150 mm c/c provided

<u>Distribution steel</u> = $\dfrac{0.12}{100} \times \dfrac{500+200}{2} = 420 \text{ mm}^2$

If 8 mm Φ bar provided spacing = $\dfrac{1000 \times 50.3}{420} = 120$ mm

8mm Φ bar @ 120 mm c/c provided.

6. **Design of heel slab –**

There are 4 forces acting on it

I. Downward weight of soil = 1.80 × 4.5 × 1 × 18 = 145.8 KN
Acting 0.9 m from point C

II. Live load/surcharge of traffic = 18 × 1 × 1.80 = 32.4 KN
Acting 0.9 m from point C

III. Weight of heel slab = 1.80 × 0.5 × 25 = 22.5 KN
Acting 0.9 m from point C

IV. Upward soil reaction = ½ (77.09 + 40.87) × 1.80 = 106.164

$$\text{Acting at} = \frac{76.37+2 \times 40.49}{76.37+40.49} \times = 0.80 \text{ m from point C}$$

Total shear force at C F = 145.8+ 32.4 + 22.5 – 106.16 = 94.54 KN

BM at point C = (145.8 + 32.4 + 22.5) × 0.90 – 106.16 × 0.8

= (200.7) ×0.90 - 106.16 × 0.8 = 95.70 KN-m

= 95.70×10^6 N-mm

$$d = \sqrt{\frac{95.70 \times 10^6}{1000 \times 0.914}} = 323 < 440 \text{ mm}$$

d = 440 already provided in toe slab

hence, keep thickness d = 440 & D = 500 mm as that of toe slab. Reduce total depth to 200 mm at edges.

$$A_{st} = \frac{95.70 \times 10^6}{230 \times 0.904 \times 440} = 1046 \text{ mm}^2$$

Using 20 mm Φ bar spacing $s = \frac{1000 \times 314}{1046} = 300$ mm c/c

$$\iota_v = \frac{94.54 \times 10^2}{1000 \times 440} = 0.214 \text{ N/mm}^2$$

If 20 mm Φ bar @ 150 mm c/c may be provided

$$\text{Then total steel Ast} = \frac{1000 \times 314}{150} = 2093 \text{ mm}^2$$

$$\% \text{ of steel} = \frac{2093 \times 100}{1000 \times 440} = 0.475 \%$$

For 0.475 % steel ι_c = 0.292 N/mm²

Hence $\iota_v < \iota_c$ safe against sliding

Hence finally 20 mm Φ bar @ 150 mm c/c provided.

Distribution steel –

Area of distribution reinforcement = $\frac{0.12 \times b \times d}{100}$ =

$$\frac{0.12 \times 1000}{100} \times \left(\frac{500+200}{2}\right) = 420 \text{ mm}^2$$

8mm Φ bar @ 120 m c/c provided

7. **Reinforcement in the stem –**

Area of reinforcement A_{st} = $\frac{161.87 \times 10^6}{230 \times 0.904 \times 440}$ = 1769 mm^2

Using 20 mm Φ bars A_{Φ} = 314.10 mm^2 then spacing

$$S = \frac{1000 \times 314.10}{1769} = 177.55 \text{ mm}^2$$

20 mm Φ bar @ 140 mm c/c Provided

Bend all the bars in the toe slab to serve as reinforcement there in order to make toe slab safe in shear.

Total actual reinforcement A_{st} = $\frac{1000 \times 314.10}{140}$ = 2243 mm^2

Total shear force at point C = $Q + KaWH_1 + KaY\frac{{H_1}^2}{2}$

$$= 2 + \frac{1}{3} \times 18 \times 4.5 + \frac{1}{3} \times 18 \times \frac{4.5^2}{2} \quad = 89.75 \text{ KN}$$

$$\iota_v = \frac{89.75 \times 10^3}{1000 \times 440} = 0.203$$

% of steel = $\frac{2243 \times 100}{1000 \times 440}$ = 0.50 %

For 0.50 % of steel $\tau_c = 0.30 N/mm^2$

Therefore $\tau_v < \tau_c$ Hence safe in shear

Now curtail the reinforcement between point B and C at depth h = 0.65 H_1 = 0.65 × 4.5 = 2.925 m below point B

Check whether 50 % (half of bar) can be curtailed or not

$$B = Q\,(h + 0.5) + Kaw\,\frac{h^2}{2} + KaY\,\frac{h^2}{2}$$

$$= 2\,(2.925 + .5) + \frac{1}{3} \times 18 \times \frac{2.925^2}{2} + \frac{1}{3} \times 18 \times \frac{2.925^2}{6}$$

= 6.85 + 25.67 + 25.02 = 57.54 KN-m

= 57.54 × 10^6 N-mm

$$d = 140 + \frac{440\text{-}140}{4.5} \times 2.925 = 335 \text{ mm}$$

$$A_{st} = \frac{57.54 \times 10^6}{230 \times 0.904 \times 335} = 826 \text{ mm}^2 < \frac{2243}{2} = 1121.5 \text{ (provided)}$$

Hence half of the bar can be curtailed at this depth. However, bar should be extended up to 12 Φ= 240 mm or d = 335 mm beyond this point.

Hence h = 2.925 – 0.335 = 2.59 m

Hence curtail half bar at 2.59 m below point B.

Similarly half of bar can be curtailed for remaining part at h = 0.65 × 2.925 = 1.90 m below point B

$$B = 2 \times (1.90 + 0.5) + \frac{1}{3} \times 18 \times \frac{1.90^2}{2} + \frac{1}{3} \times 18 \times \frac{1.90^2}{6}$$

= 4.8 + 10.83 + 6.86 = 22.89 KN-m

= 22.89 × 10^6 N-mm

$$d = 140 + \left(\frac{440\text{-}140}{4.5}\right) \times 1.90 = 266.67$$

$$A_{st} = \frac{22.89 \times 10^6}{230 \times 0.904 \times 266.67} = 412.84\ mm^2 < \frac{2243}{4} = 560.75\ mm^2$$

Hence half bar can be curtailed a below h = 1.90 – 0.267 = 1.633 m below point B.

Rest of the bar can be continued up to the top of parapet.

Distribution Reinforcement –

$$\text{Area of distribution steel} = 0.12 \times \left(\frac{500+200}{2}\right) \times 1000$$

$$= 420\ mm^2$$

Using 10 mm Φ bar $A_{ö}$ = 78.5 mm²

Spacing = = 186.90 mm²

Hence 10 mm Φ bar @ 150 mm c/c provided

Temperature reinforcement –

Total area = 420 mm²

Provide 10 mm Φ bar @ 300 mm c/c both way on the outer face.

8. **Design of shear key –**

The Wall is unsafe against sliding Hence to prevent the sliding failure shear key has to be provided.

Passive pressure intensity will develop in-front of shear key to resist sliding. This pressure intensity depends upon the soil pressure P just in-front of shear key.

$P_P = K_p P \quad K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3$

$P_P = 3 \times 86.34 = 259.02\ KN/m^2$

Total passive force

$\mathbf{P_p} = P_p a \quad$ keeping a = 0.5m

= 259.02 × 0.5

= 129.51 KN

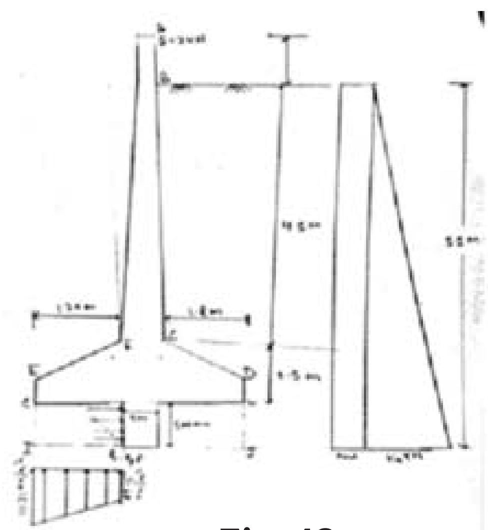


Fig. 49

Total sliding force at bottom level of key

$$\Sigma P = 2 + \frac{1}{3} \times 18 \times 5.5 + \frac{1}{3} \times 18 \ \frac{5.5^2}{2}$$

= 2 + 33 + 90.75 = 125.75 KN

Weight of soil between base and level IJ

= 3.5 × 0.5 × 1 × 18 = 31.5 KN

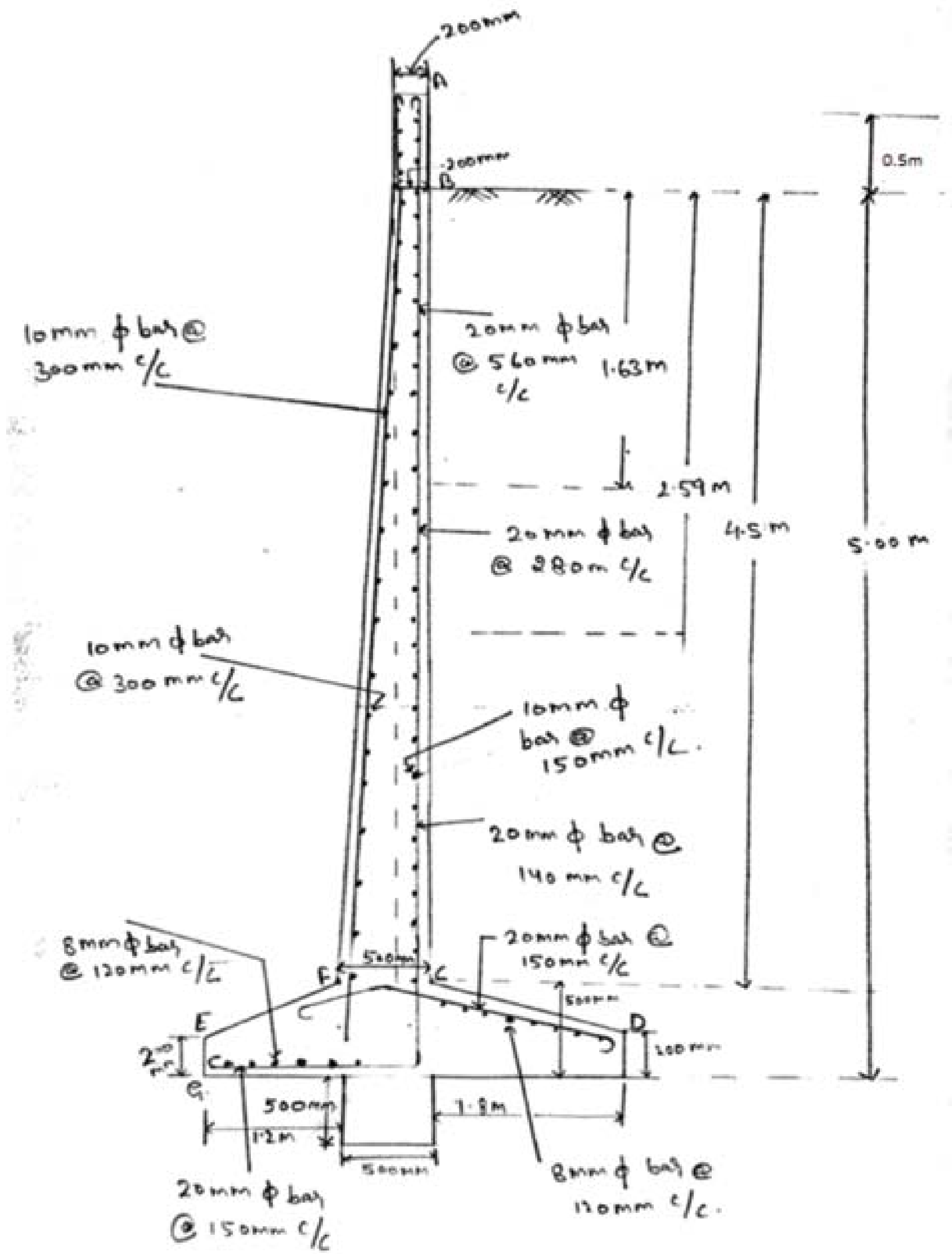
Total vertical load = 263.83 + 31.5 = 295.33 KN

Hence factor of safety against sliding = $\underline{\pm}$

$$= \frac{0.4 \times 295.33+129.5}{125.75} = 1.97 > 1.5 \quad \text{hence safe}$$

The Shear key is to be casted monolithically with base.

Note- Reinforcement Detail of RCC retaining wall shown in figure.



200mm
A
0.5m
200mm
B
10mm ϕ bar @
300mm c/c
20mm ϕ bar
@ 560mm
c/c
1.63m
2.59m
20mm ϕ bar
@ 280m c/c
4.5m
5.00m
10mm ϕ bar
@ 300mm c/c
10mm ϕ
bar @
150mm c/c.
20mm ϕ bar @
140 mm c/c
8mm ϕ bar
@ 120mm c/c
20mm ϕ bar @
150mm c/c
F
500mm
C
E
D
200mm
G
500mm
1.2m
1.8m
500mm
20mm ϕ bar
@ 150mm c/c
8mm ϕ bar @
120mm c/c.

Fig. 50

QUANTITY CALCULATION OF CANTILEVER RCC RETAINING WALL OF HEIGHT 6 M

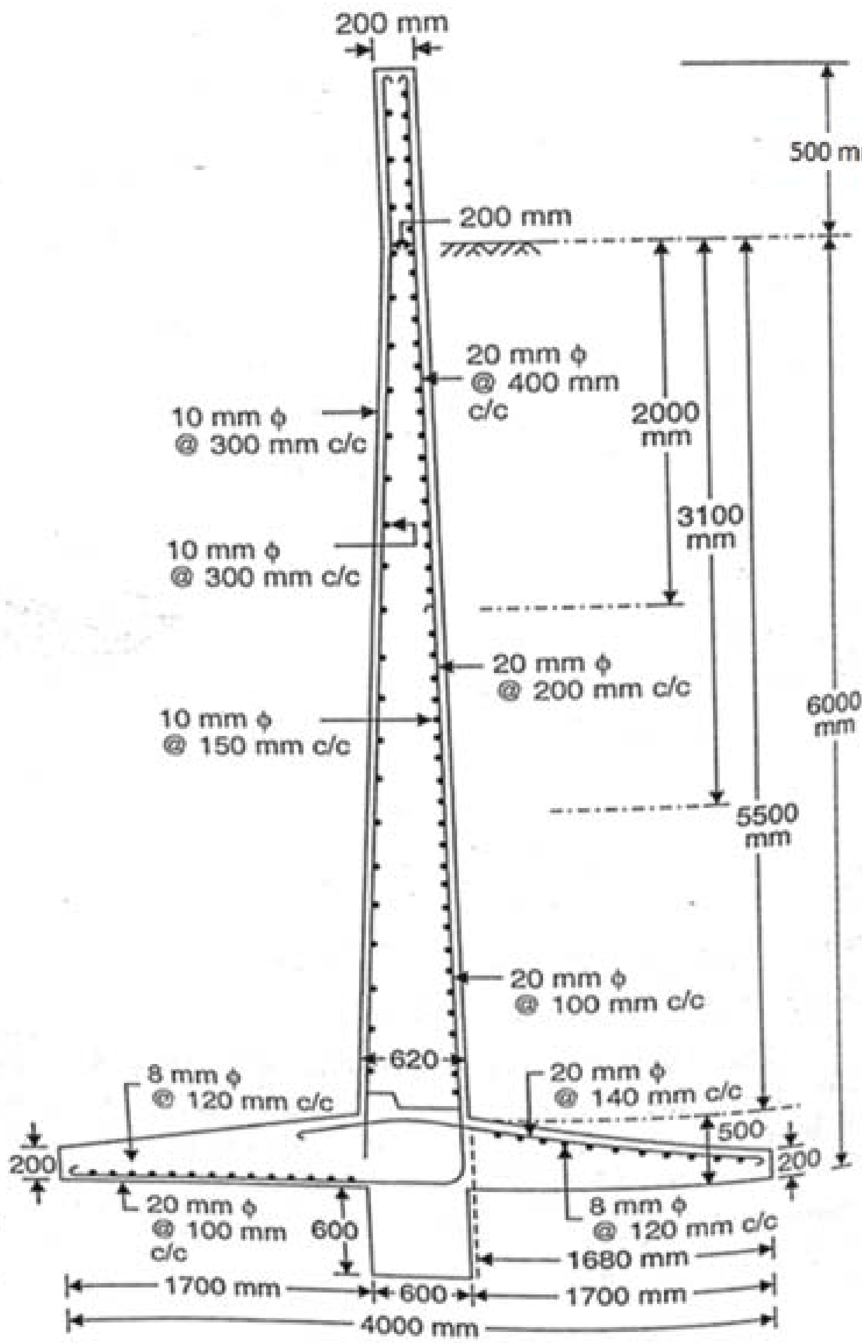


Fig. 51

Sr No	ITEM OF WORKS	A/U	Nos	LEN -GTH	WIDTH	DEPTH/ HEIGHT	QTY	REMARKS
1	Excavation in trenches	Cum	1	8.3	4.3	1.5	53.535	Depth 1.5 m considered
	for bottom key	"	1	8.3	0.9	0.7	5.229	10 cm extra taken for leveling course
							Total	58.764
2	PCC 1:3:6 for leveling course	Cum	1	8	4.3	0.1	3.44	
3	PCC 1:1.5:3 for key	Cum	1	8	0.6	0.6	2.88	
4	RCC 1:1:2 (M-25 Grade) For Stem	Cum	1	8	(.62+.2) /2	6	19.68	
	"	"	1	8	0.2	1	1.6	
	For bottom trapezoidal portion Hill side	"	1	8	1.68	(.50+.2) /2	4.704	
	For bottom trapezoidal portion valley side	"	1	8	1.7	(.50+.2) /2	4.76	
	Total	"					30.744	
5	**Centering/ Shuttering**							
	For bottom key	Sqm	2	8		0.6	9.6	
	For bottom key sides	"	2	0.6		0.6	0.72	
	For Stem hill side	"	1	8		5.5	44	
	For top straight portion	"	2	8		1	16	
	For top straight portion sides	"	2	0.2		1	0.4	
	For Stem valley side	"	1	8		5.51	44.08	
	For sides	"	2	(.62+0.2)/2		6	4.92	
	For trapezoidal portion sides	"	2	8		0.2	3.2	
	For trapezoidal portion sides hill side	"	2	1.68		(.50+.2) /2	1.176	
	For trapezoidal portion sides valley side	"	2	1.7		(.50+.2) /2	1.19	
	Total	"					125.286	

Sr No	ITEM OF WORKS	A/U	Nos	LEN -GTH	WIDTH	DEPTH/ HEIGHT	QTY	REMARKS
6	**Reinforcement**							
	20 mm dia bar up to top @ 400 mm c/c	Kgs	21			9.52	493.8024	L=7- 2x.04+0.58 +1.66+0.36
	20 mm dia bar up to 4 mtr @ 200 mm c/c	Kgs	20			7.36	363.584	L=4-1x.04+0.58+ 1.66+0.36+0.80
	20 mm dia bar up to 2.9 mtr @ 100 mm c/c	Kgs	40			6.26	618.488	L=2.9- 1x.04+0.58+ 1.66+0.36+0.80
	20 mm dia bar for hill side fdn @ 140 mm c/c	Kgs	58	3.83			548.6858	L=1.73 1x.04+0.62+ 0.8+2x0.36
	Total 20 mm	Kgs					2024.56	
	Add 10% for wastage & lap length	Kgs					202.456	
	G/Total 20 mm						2227.016	
	10 mm dia bar hill side binder @ 150 mm c/c	Kgs	44	8.17			222.8776	L=8-2x.04+0.25
	10 mm dia bar valley side vertical @ 300 mm c/c	Kgs	27	7.22			120.8628	L=7.01-1x.04 +0.25
	10 mm dia bar valley side binder @ 300 mm c/c	Kgs	22	8.17			111.4388	L=8-2x.04+0.25
	Total 10 mm	Kgs					455.1792	
	Add 10% for wastage & lap length	Kgs					45.51792	
	G/Total 10 mm						500.6971	
	8 mm dia bar for fdn hill side binder @ 120 mm c/c	Kgs	15	8.22			48.7035	L=8-2x.04+2x.15
	8 mm dia bar for fdn valley side binder @ 120 mm c/c	Kgs	15	8.22			48.7035	L=8-2x.04+2x.15
	Total 8 mm						97.407	
	Add 10% for wastage & lap length						9.7407	
	G/Total 8 mm						107.1477	
7	Back filling with stone	Cum	1	8	(0.15+	6.5 0.60)/2	19.5	

QUANTITY CALCULATION OF CANTILEVER RCC RETAINING WALL OF HEIGHT 5 M

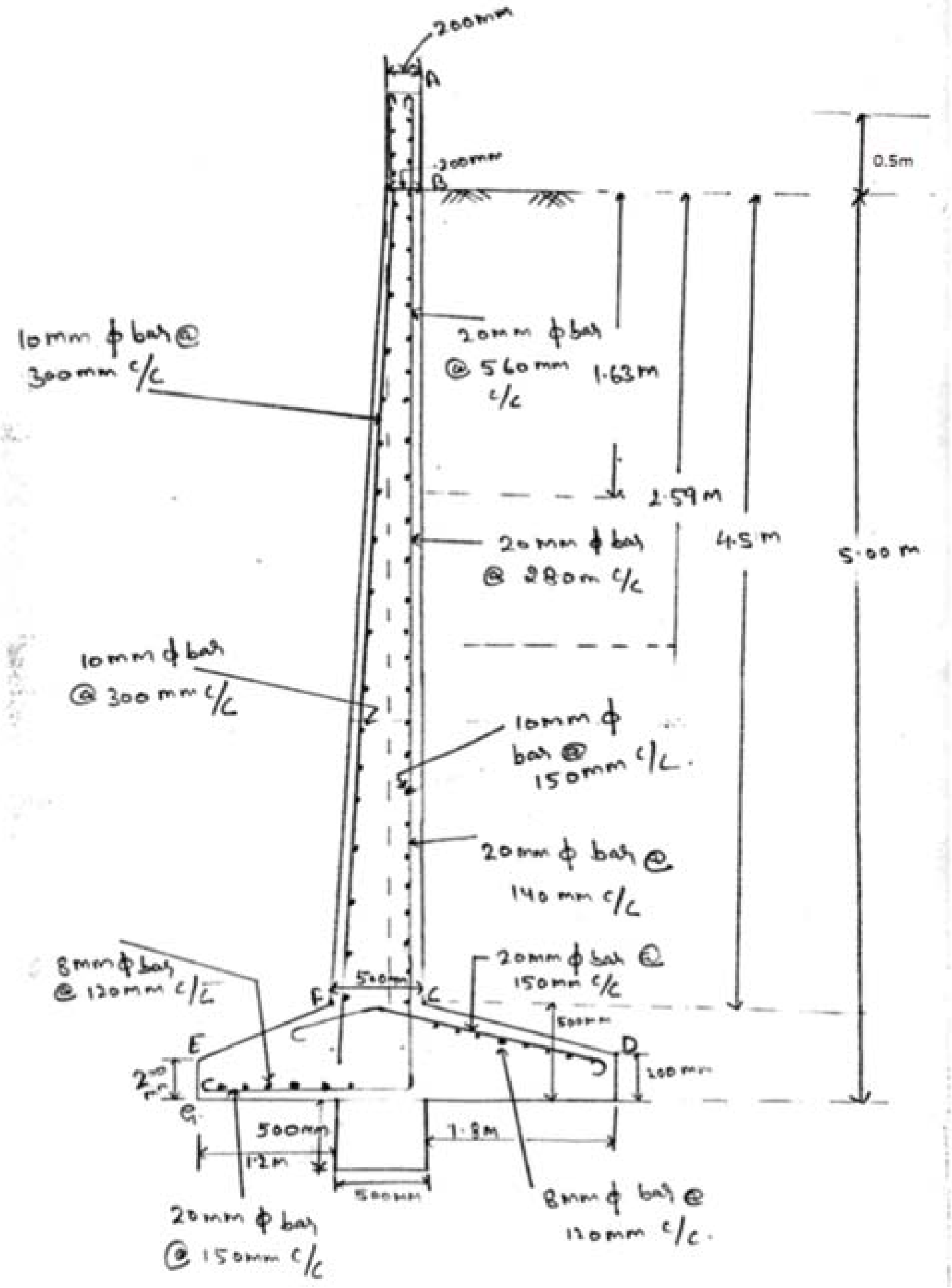


Fig. 52

Sr No	ITEM OF WORKS	A/U	Nos	LEN -GTH	WIDTH	DEPTH/ HEIGHT	QTY	REMARKS
1	Excavation in trenches	Cum	1	8.3	3.8	1.5	47.31	Depth 1.5 m considered
	for bottom key	"	1	8.3	0.8	0.7	4.648	10 cm extra taken for leveling course
	Total						51.958	
2	PCC 1:3:6 for leveling course	Cum	1	8	3.8	0.1	3.04	
3	PCC 1:1.5:3 for key	Cum	1	8	0.5	0.5	2	
4	RCC 1:1:2 (M-25 Grade) For Stem	Cum	1	8	(0.5+.2) /2	5	14	
	"	"	1	8	0.2	1	1.6	
	For bottom trapezoidal portion Hill side	"	1	8	1.8	(.50+.2) /2	5.04	
	For bottom trapezoidal portion valley side	"	1	8	1.2	(.50+.2) /2	3.36	
	Total	"					24	
5	**Centering/ Shuttering**							
	For bottom key	Sqm	2	8		0.5	8	
	For bottom key sides	"	2	0.5		0.5	0.5	
	For Stem hill side	"	1	8		4.5	36	
	For top straight portion	"	2	8		1	16	
	For top straight portion sides	"	2	0.2		1	0.4	
	For Stem valley side	"	1	8		4.51	36.08	
	For sides	"	2	(0.50+ 0.2)/2		5	4.1	
	For trapezodial portion side	"	2	8		0.2	3.2	
	For trapezodial portion sides hill side	"	2	1.8		(.50+.2) /2	1.26	
	For trapezodial portion sides valley side	"	2	1.2		(.50+.2) /2	0.84	
	Total	"					106.38	

S. No	ITEM OF WORKS	A/U	Nos	LEN -GTH	WIDTH	DEPTH/ HEIGHT	QTY	REMARKS
6	**Reinforcement**							
	20 mm dia bar up to top@ 560 mm c/c	Kgs	15			7.9	292.695	L=6-2x.04+0.46+ 1.16+0.36
	20 mm dia bar up to 3.37mtr @ 280 mm c/c	Kgs	14			6.11	211.2838	L=3.37- 1x.04+0.46+ 1.16+0.36+0.80
	20 mm dia bar up to 2.41 mtr @ 140 mm c/c		Kgs	28		5.15	356.174	L=2.41- 1x.04+0.46+ 1.16+0.36+0.80
	20 mm diabarfor hill side fdn @ 150 mm c/c	Kgs	54	3.78			504.1764	L=1.80- 1x.04+0..50+ 0.8+2x0.36
	Total 20 mm Add 10% for wastage & lap length	Kgs Kgs					1364.329 136.4329	
	G/Total 20 mm						1500.762	
	10 mm dia bar hill side binder @ 150 mm c/c	Kgs	37	8.17			187.4198	L=8-2x.04+0.25
	10 mm dia bar valley side vertical @ 300 mm c/c	Kgs	19	6.22			73.2716	L=6.01- 1x.04+0.25
	10 mm dia bar valley side binder @ 300 mm c/c	Kgs	22	8.17			111.4388	L=8-2x.04+0.25
	Total 10 mm	Kgs					372.1302	
	Add 10% for wastages & lap length	Kgs					37.21302	
	G/Total 10 mm						409.3432	
	8 mm dia bar for fdn hill side binder @ 120 mm c/c	Kgs	16	8.22			51.9504	L=8-2x.04+2x.15
	8 mm dia bar for fdn valley side binder @ 120 mm c/c	Kgs	11	8.22			35.7159	L=8-2x.04+2x.15
	Total 8 mm						87.6663	
	Add 10% for wastage & lap length						8.76663	
	G/Total 8 mm						96.43293	
7	Back filling with stone	Cum	1	8	(0.15+ 0.60)/2	5.5	16.5	

Details of RCC Retaining wall of 5m & 6m height

Sr No.	Component		A/U	RCC Retaining wall 5m height	RCC Retaining wall 6m height	Remark
1.	Base Slab	Width	Meter	3.50	4.00	
		Thickness	Meter	0.50 to 0.20	0.50 to 0.20	
2.	Toe slab	Length	Meter	1.20	1.70	
		Main reinforcement		20 mm Φ bar @150 mm c/c	20 mm Φ bar @100 mm c/c	At the bottom of Toe slab
		Distribution reinforcement		8 mm Φ bar @120 mm c/c	8 mm Φ bar @120 mm c/c	At the bottom of Toe slab
3.	Heel slab	Length		1.80	1.68	
		Main reinforcement		20 mm Φ bar @150 mm c/c	20 mm Φ bar @140 mm c/c	At the top of Heel slab
		Distribution		8 mm Φ bar	8 mm Φ bar	At the top of
		reinforcement		@120 mm c/c	@120 mm c/c	Heel slab
4.	Stem	Height	Meter	4.50	5.50	
		Thickness	Meter	0.50 to 0.20	0.62 to 0.20	Bottom to Top
		Main reinforcement		20 mm Φ bar @140 mm c/c (Upto 1.91 m from bottom of stem)	20 mm Φ bar @100 mm c/c (Upto 2.4 m from bottom of stem)	Inner side of wall i.e backfill side
				20 mm Φ bar @ 280 mm c/c (Between 1.91 m & 2.87 m from bottom of stem)	20 mm Φ bar @200 mm c/c (Between 2.4 m & 3.5 m from bottom of stem)	Inner side of wall i.e backfill side
				20 mm Φ bar @ 560 mm c/c (Above the 2.87 m from bottom of stem)	20 mm Φ bar @400 mm c/c (Above the 3.5 m from bottom of stem)	Inner side of wall i.e backfill side
		Distribution reinforcement		10 mm Φ bar @150 mm c/c	10 mm Φ bar @150 mm c/c	Inner side of wall
		Temperature Reinforcement		10 mm Φ bar @300 mm c/c	10 mm Φ bar @300 mm c/c	Outer side of wall in both ways

20. BREAST WALLS

20.1 Masonry or RCC structures supporting the uphill slopes along a road are termed Breast Walls. For protection against instability, slopes of cutting would have to be very flat, which are not economically practicable. At some places such slopes fail by slumping, sliding, toe failures or in the worst cases by failures far below the formation level, causing entire road to be washed away. Thus, weak spots which are chronic by way of hill slides have to be protected by breast walls. Such walls perform the following functions:

(a) They would keep the road edge defined and also protect the drain up to some extent.

(b) The hill slope to the extent of Breast Wall height will remain protected from slips. Any slide above this height will flow over the top of the breast wall.

(c) It would not allow continuity of the flowing mass of soil and would thus facilitate the clearance of slides.

(d) Assistance in drainage from hill-slope through weep-holes in breast wall on to the slide drains in front of the wall.

20.2 **Materials of construction.** Since these walls are usually provided against wet hill slopes, they have to be stronger than Retaining Walls and are usually comparatively less stable. They are usually provided in stone masonry in cement mortar though banded and dry masonry construction may be done where hill slope is not wet. Weep-holes have to be provided liberally in breast walls.

20.3 In very unstable situation, even stones enclosed in wire crates have been successfully tried which have the advantage of re-building in case of failure and act as flexible structures.

20.4 **Design.** The breast wall design has to consider:

(a) Of necessity, the height of these walls is kept low i.e. about 1.5m to 3 m since they occupy lot of useable space in the cross section and generally the height is kept as 1.5 m.

(b) Front batter is usually kept 1: 3 as against 1:4-1:6 for retaining walls except that earth pressure is computed as a saturated coarse backfill with a 60^0 slope of hill.

20.5 A typical breast wall is given in Fig. 54.

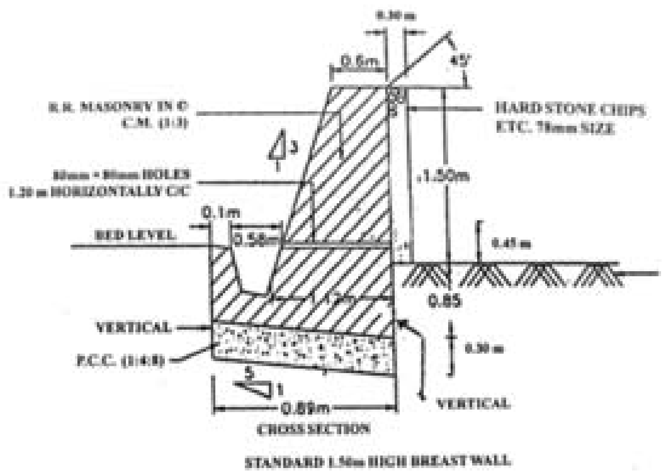


Fig. 53

21. REINFORCED SOIL RETAINING WALLS

21.1 The concept of reinforced soil is not new rather it is as old as civilization. Example of reinforcing techniques in nature is seen in three routes around stream beds and in birds nest etc. During the last three decades since Henry Vidal successfully demonstrated the construction of reinforced soil in 1966, recognition and interest in the subject have increased rapidly. Hundreds of retaining walls have been constructed and many countries have incorporated this technique in their standards and formulated guidelines. In some countries has almost replaced the conventional reinforced concrete and gravity walls. In India, this technique is being adopted in express way and National highways.

21.2 Reinforced earth technologies for retaining walls offers following advantage over alternate/conventional design and construction.

(a) **Flexibility.** making it possible to be directly on weak to very weak foundation soils or on unstable slopes.

(b) **Very high resistance.** To both static and dynamic loads.

(c) **Ease of installation.** From the use of completely prefabricated and reinforcing elements.

(d) **Excellent appearance.** Since precast elements are of the finished adaptable a great variety of structure architectural finishes.

(e) **Considerable saving.** In both construction time and material.

21.3 Reinforced soil retaining wall is a composite construction material in which the strength of fill is enhanced through addition of inextensible tensile reinforcement in form of strip, sheets, grids or geo textiles. The technique of construction is quite suitable for hilly areas mainly due to following reasons.

(a) The fill material which consists of mainly granular material is easily available in all parts of hilly areas from cutting hill side during the construction of road.

(b) This involves minimum alteration in natural slopes since the emphasis is on avoiding the cutting of natural slope.

(c) The land width or actual embankment width required is less.

(d) This is cost effective and environment friendly.

(e) This is self-draining and no weep hole is required.

21.4 Reinforced soil Retaining walls are flexible walls. The main components are :

(a) Back fill which is granular soil.

(b) Reinforcing material -thin and wide strips or Geo synthetics placed at regular intervals.

(c) Facing galvanized steel plates or pre-cast concrete panels – In case of geo textiles, the same can be wrapped around earthwork and covered with stone masonry etc. In case of inclined wall the facing can be covered with earth and turfing.

21.5 A typical reinforced earth retaining wall is shown in Fig. 55 and 55(A) to 55(E). Reinforcing ties are placed with horizontal spacing of 'h' center to center and vertical spacing 'V' center to center. With a conservative design, 0.5 mm thick galvanized steel plate would be enough to hold a wall about 15 m high. The use of pre-cast slab as a cover on front face is also common. The slabs are grooved to fit into each other so that soil does not flow out between the joints. Panel size configuration, surface texture and color can be varied to provide aesthetically pleasing finished structures. When metal plates are used, they are bolted together and reinforcing is placed between the plates.

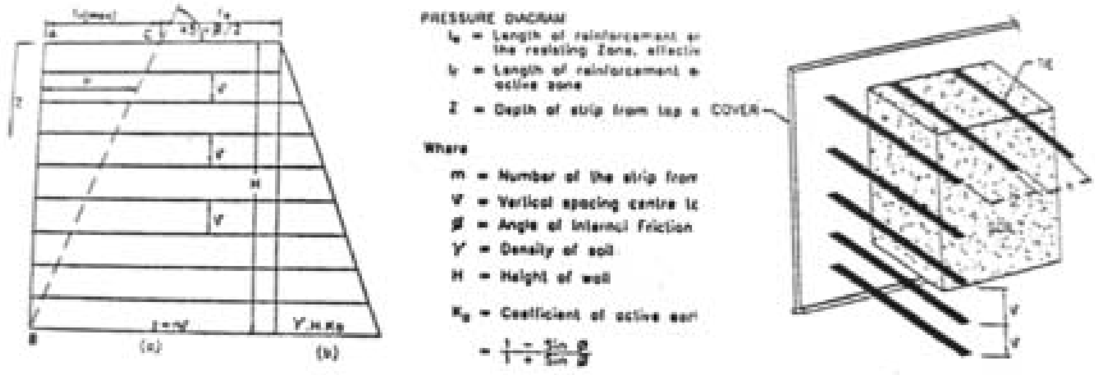


Fig. 54

21.6 Design Principle and Analysis

21.6.1 Design Principle:-

(a) Reinforced earth structure behaves like a gravity wall addition to its own weight. The block transfers the effect of surcharge and earth pressure to the foundation and distributes them evenly over entire width of its base.

(b) Due to flexibility of wall, earth pressure corresponds to the active state.

(c) At the base of structure, vertical stress in embankment is higher in proximity to the facing and tensile stresses within the reinforcing strips and max" at certain distance behind the facing. The line joining the points of maximum tensile force separates the active zone in which the reinforcing strips retain the fill from passive zone.

(d) The reinforcing strips transfers the shear force in planes lying midway between them. The horizontal stress can be expressed as $\delta_h = k\delta_v$

Where

δ_h = horizontal stress
δ_v = vertical stress
K = practically equal to the active earth pressure coefficient Ka

The value of K is higher from the top of the walls and up to approximately 6m in depth.

(e) Tensile stress levels at the connections to the concrete facing panels may each 85 percent of maximum tensile stress and up to 100 percent at base of the wall.

(f) An objective of incorporating soil reinforcement is to absorb tensile loads or shear stresses, thereby reducing the loads which might otherwise cause the soil to fail in shear or by excessive deformation.

21.6.2 **Analysis.** The design is based on the principle of limit state. The two limit states considered in the design are ultimate limit state and serviceability limit state. The margins of safety against attending the limit states are provided by use of material factor and partial load factors.

The following stability analysis is done for the reinforced soil structure:

(a) **External stability**

(a) Overturning of the reinforced soil block.

(b) Bearing capacity failure of the foundation soil.

(c) Sliding of the structure on the foundation soil.

(d) Global slip surface failures.

(b) **Internal stability**

(a) Tensile rupture at any point along the length of reinforcement.

(b) Tensile/shear failure at the connection between the reinforcement and facing element.

(c) Loss of frictional bond (adherence) between the reinforcement and the soil fill.

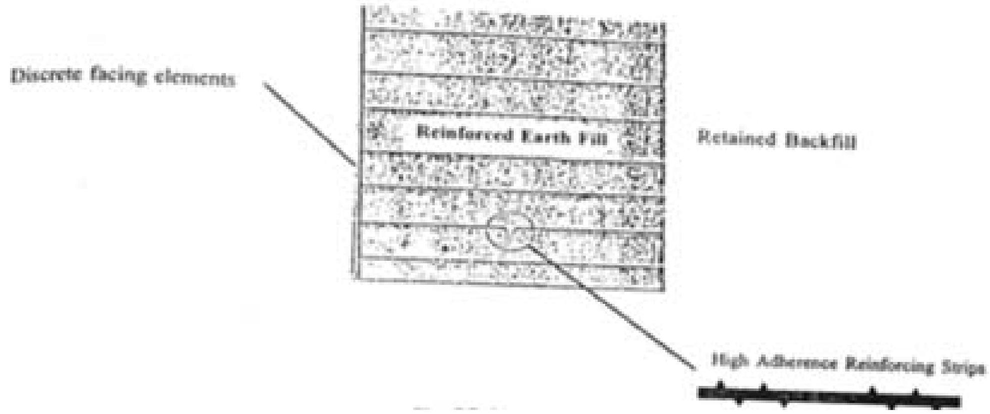


Fig. 55A: Basic concepts forming a Rcinforced Earth Structure

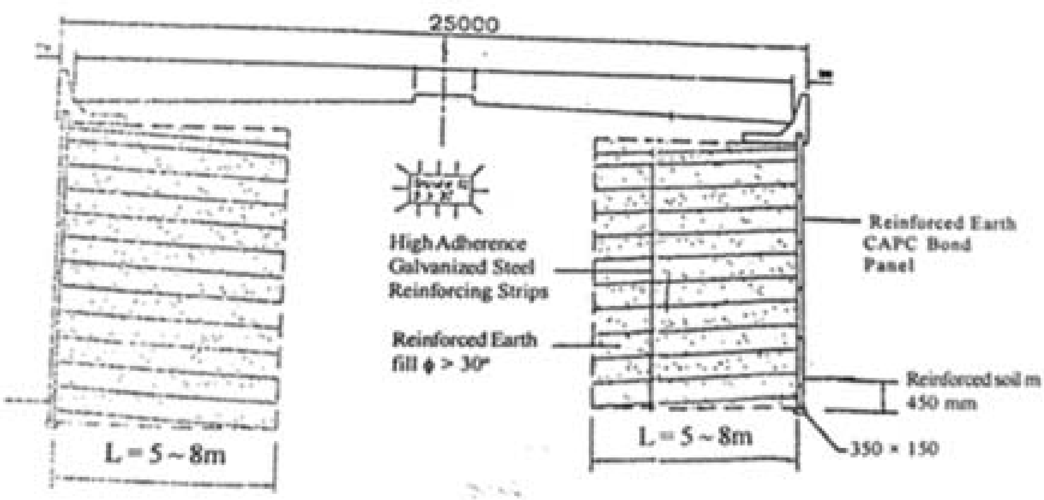


Fig. 55B: Typical cross section

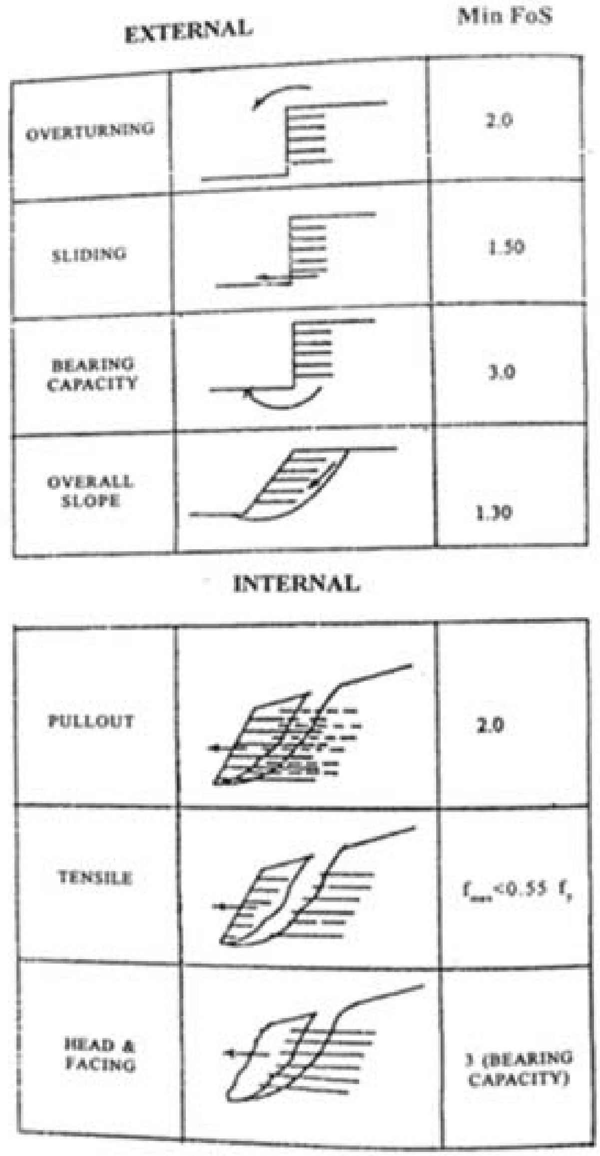

EXTERNAL
Min FoS
OVERTURNING
2.0
SLIDING
1.50
BEARING CAPACITY
3.0
OVERALL SLOPE
1.30
INTERNAL
PULLOUT
2.0
TENSILE
f max<0.55 f y
HEAD & FACING
3 (BEARING CAPACITY)

Fig. 55C

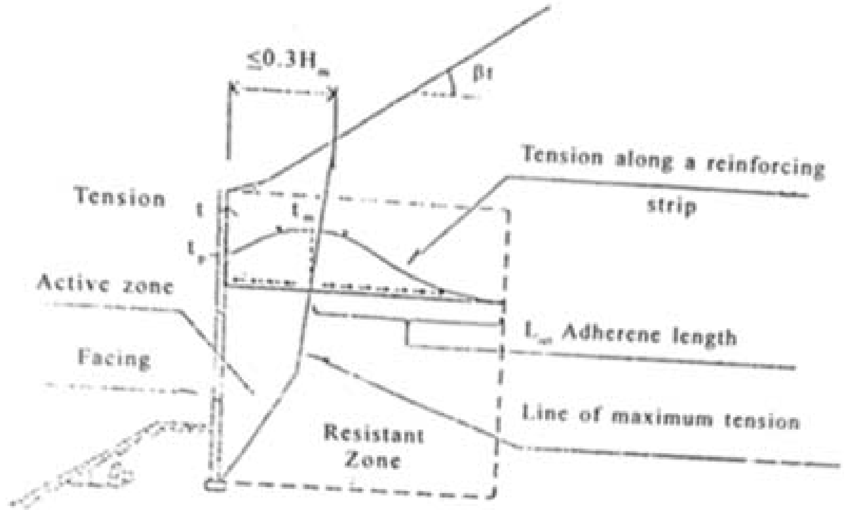


Fig. 55D: Line of maximum tension, active zone, rsistant zone

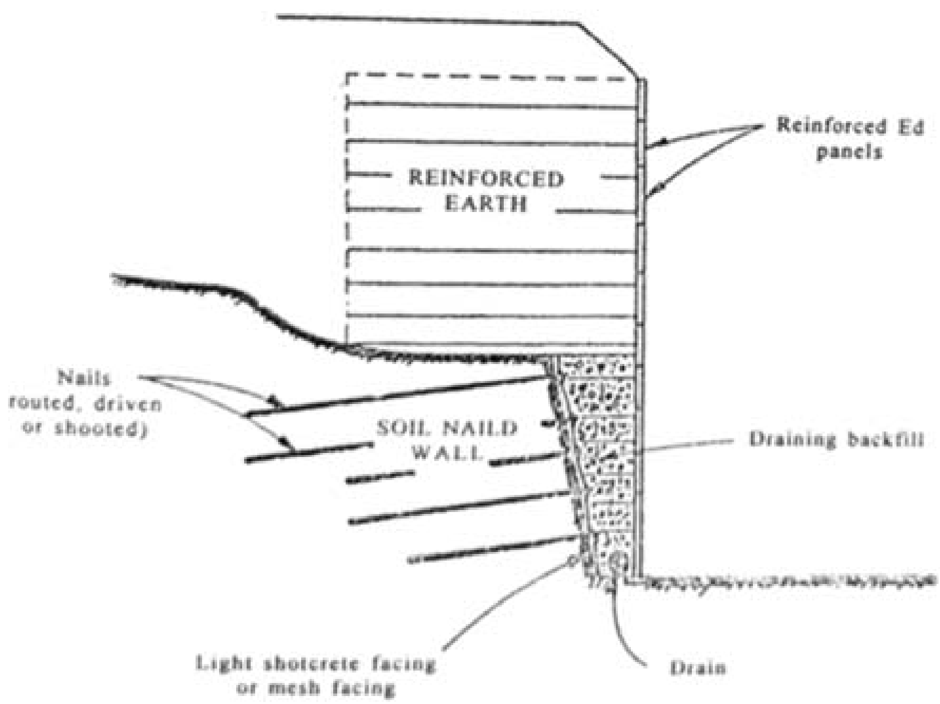


Fig. 55E: Combination of Reinforced Earth and Soil Nailing

PART III: PLANNING, CONSTRUCTION AND MAINTENANCE OF RETAINING WALLS

22. PLANNING

Alignment of retaining wall must be finalized in office based on ground data and keeping in view the overall geometrics of road. Following steps should be taken:-

(a) **Longitudinal Section.** Longitudinal section of road for a stretch minimum 100 m on either side should be plotted ad grade line as per road geometrics (allowing flat gradient in stretches with curves, approaches to bridge/culverts etc.) be marked.

(b) **Plan indicating layout of road.** Plan indicating existing road should be drawn for a stretch minimum 100 m on both side of proposed site of retaining wall. Keeping in view the norms for geometrics of road, the center line of road (and formation after due consideration for extra width for curves etc.) should be marked. The stretch/area for requiring improvement by further cutting on hill side and widening (by filling or construction of wall) on valley side should be identified/marked on plan clearly.

(c) **Layout of culvert.** Layout of bridge/culvert should be marked on the road plan. Layout of approaches to the bridge/culvert in accordance with accepted geometrics should be marked on road plan.

(d) **Improvement in existing formation.** Keeping in view the required geometrics in Vertical and Horizontal profile, the required improvement should be planned/ carried out.

(e) **Further improvement of formation.** After improvement of formation (in accordance with the planning as above) fresh detailed survey should be done to cross check whether the required geometrics of road (both in vertical and horizontal profile) has been achieved or not. If necessary, further improvement of formation should be done.

(f) **Fixing top level of wall.** Top level of retaining wall should be 15 cm lower than the proposed level of road at central line (after surfacing) considering the camber of carriageway. Top level of retaining Wall (with reference to the road level) should be fixed for the stretches of retaining wall at 4 m interval.

(g) **Fixing top level of wall in stretch of road in curve.** Top level of outer and inner edge of road formation should be worked out keeping in view the requirement of super elevation in curve. Top level of wall should be 15 cm lower than the formation level at the edge of road formation.

(h) **Alignment of Retaining Wall.** Alignment of retaining wall indicating clearly top portion of finished retaining wall should be clearly drawn/indicated over the plan of rod for the entire length of wall. Alignment, as drawn on plan, should be checked at every 4 mtr interval for the geometrics of road in horizontal profile.

23. FIXING LAYOUT OF RETAINING WALL ON GROUND

23.1 It is a very important stage. Any error in fixing the outer and inner line of the trench foundation of retaining wall could cause unnecessary excavation or error in alignment of finished wall. Sketch has been drawn below indicating the various steps. Following steps following steps should be taken for fixing out and inner line of trench foundation excavation.

(a) **Fixing Layout of Top of wall on ground.** Initially the layout of top of wall (according to the alignment of wall drawn on plan of road) be marked on ground (maintaining slope of wall in longitudinal direction as per level arrived from longitudinal section) with the help of scantlings/bamboos/ballies (ABC) and tying mason thread over it (at B in sketch) indicating outer edge of top of wall. (See Fig. 56).

(b) **Measurement of height of wall.** Required height of wall is required to be ascertained for small stretches of wall (at least at every 4 m intervals) for designing wall section in accordance with ground profile, after taking into consideration the wall deep enough below natural ground. Following steps be taken to ascertain the height of Retaining wall :-

(i) Drop the mason weight tied with mason thread (see FC in Fig. 56) taking into consideration the front batter of retaining wall (1:4).

(ii) Mark the position and measure the height.

(iii) Workout overall height of wall considering the wall will be taken at least one tenth of height of wall below natural ground.

(iv) Workout the width of wall at base and width of foundation concrete.

(v) Accordingly give layout of the proposed excavation line of trench for foundation concrete.

(vi) Mark on ground outer and inner of excavation by means of line of either lime or excavation marks on ground.

(vii) Do not disturb the mason thread or the scantling/ bamboo/ballies. It will be regular for checking the layout after completion of excavation for foundation concrete and before allowing concreting.

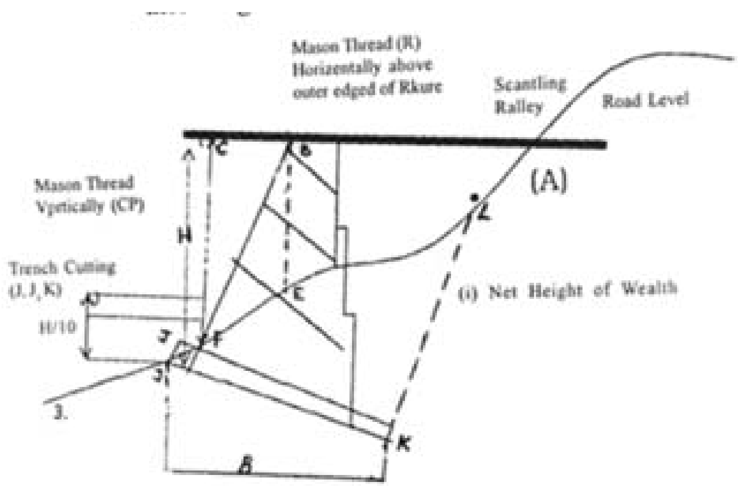


Fig. 56

24. MASONRY WORKS

24.1 The retaining structures should be made in rubble masonry consisting of hammer dressed hard-stone brought to courses every 0.60 m (2.00 ft) approximately. All the masonry courses must be normal to the batter (surface) in the front face of wall. The back of the wall can be left rough. Adequate interlocking of stones in masonry should be ensured. During construction stage, masonry work should proceed to a uniform level. Interstices between stones should be properly filled with small pieces of stone, gravels etc. Detailed guidelines, as given below in Para 24.1 to 24.3 &Para 25 to 31 be followed for masonry works.

24.2 **Stones in Masonry.** Normally no stone should be used in masonry which is less than 20 x 15 x 10 cm in size (1/35 cu mt). Bed of each stone should not normally be less than 1- ½ times its height. Approximately half the stones in masonry should tail into the wall by twice the height. Stone shall be place in such a manner so that it must break joint by half the height of course in walls.

24.3 **Dry Rubble Wall.** In case of dry rubble wall the voids and interstices between stones should be filled up properly with sand, small pieces of stone, gravel etc. so that load coming over the wall is distributed uniformly. It is advisable to bed each course in stone-dust or earth to help spread the load and to increase the frictional resistance between stones in all the courses of masonry particularly where shale slab are used in the walls.

24.4 **Coping.** The coping should consist of large stones laid and pointed in cement mortar or PCC 5.0 to 7.5 cm thick. The top of the coping should be weather sloped towards valley side. Coping, preferably should be with stones on edge, so that these are not easily dislodged. Parapet with weather slope may be provided on retaining wall in lieu of coping over-wall.

25. BOND AND PLUM STONES (HEADERS) IN MASONRY

25.1 For achieving proper interlocking of structural units in the retaining structure and to ensure adequate stability of masonry, bond be provided laterally as well as vertically. Lateral bond is created by use of bond or through stones and vertical bond is created by plum or pin headers.

25.2 In the case of random rubble masonry in cement mortar, the interlocking and adhesion between the cement mortar and stone surface is responsible for the basic strength of stone masonry. Similarly, in the case of dry rubble masonry, mainly the mass of stones imparts the basic strength. However, certain broad principles in laying, bonding, breaking of joints and finish have to be complied with in order that the masonry develops adequate strength and presents and neat appearance.

25.3 Creation of a bond between the face and back of any masonry wall thereby the whole of interior thickness for a particular effective area) is the most important aspects in stone masonry construction.

25.4 The bond affords interlocking of structural units in a wall to ensure stability. This bond can be provided both laterally as well as vertically. The lateral bond is created by use of bond or through stones while the vertical bond is created by plum or pin headers.

25.5 Following specifications shall be adopted in bond stones and plum stones which will be provided in case of RR masonry both coursed and uncoursed Masonry and dry rubble masonry brought to courses.

(a) **Bond stones or though stones:** Through bond stones shall be provided in masonry up to 60 cm thickness and in case of masonry above 60 cm thickness, a set of two or more bond stones overlapping each other at least by 15 cm shall be provided in a line from face to back. In case of highly absorbent types of stones (porous lime stone and sand stones, etc.) the bond stone shall extend only about two-thirds into the wall, as through stones in such cases may give rise to penetration of dampness and, therefore, for all thickness for such masonry a set of two or more bond stones overlapping each other by at least 15 cm shall be provided. Each bond stone or a set of bond stones shall be provided in every 0.5 sq. meter area of the wall surface.

(b) **Plum stones or pin headers:** Plum stones are selected long stones embedded vertically in the interior of masonry to from a bond between successive courses of masonry and shall be provided at interval of 90 cm.

26. CONSTRUCTION JOINTS IN RETAINING AND BREAST WALLS

26.1 Retaining walls are designed and constructed to with stand the pressure from backfill. The geometric section depends on the forces acting on the wall, self-weight of the wall, bearing strength of the foundation strata and the specification of the wall itself viz gravity walls, cantilever walls or counter-fort walls. A particular section is valid only for those set of conditions which are considered in the design of the wall. Variations in any of the design factors are likely to change the section as required. Hence, to ensure safe and economical sections, it is necessary to design and construct such walls as independent units, the section adopted for each unit dictated by the forces that are to be with stood by the walls, foundation bearing strength, etc, This can be achieved by provision of joints in the walls. Construction joints also minimize the harmful effects of temperature changes and also the practical problem that only small length can be constructed in one continuous operation. In retaining walls on hill roads, construction joints have an added function that the damages that occur due to slides, subsidence, etc. are localized and affect only small lengths of the wall.

26.2 Construction joints are required to be provided right through from top of the wall to the bottom of the foundation running through full section of wall to act as an independent section.

26.3 Construction joints are required to be provided at following points:-

(a) At locations where the type and nature of foundation strata changes.

(b) Sections where steps occur in the foundation or in the top of the wall as these are relatively weak planes.

(c) At locations where the section of wall is changed due to the following factors:-

(i) Nature and type of backfill.

(ii) Change in direction of retaining wall.

26.4 Subject to the above conditions, construction joints should be provided at intervals of not more than 8 meters i.e., length of wall not to exceed 8 meters in any one continuous stretch.

26.5 Construction joints shall ordinarily be provided 25 mm wide and will be open joints without keys or fillers.

26.6 This applies to breast walls and toe walls also. Joints are to be provided even when specification for wall as adopted is masonry in mortar, banded masonry, dry masonry, wire mesh crated masonry, plain or reinforced cement concrete.

27. BACK FILL

27.1 The back fill layer of 60 cm immediately behind the wall should consist of stone or some granular materials as shown in Fig. 57 & 58. Balance portion of the backfill should be rammed in 15 cm layers slopping away and downwards from the back of the wall. Impervious silty soil layer or back-fill of about 30 cm thickness shall be Provided in the top layer of backfill to prevent seepage of any rain/surface water in the backfill or into the foundation of building on terraces (See figure 71). However, the backfill shall be of self-draining material (coarse sand, Gravel and boulder), free of fines.

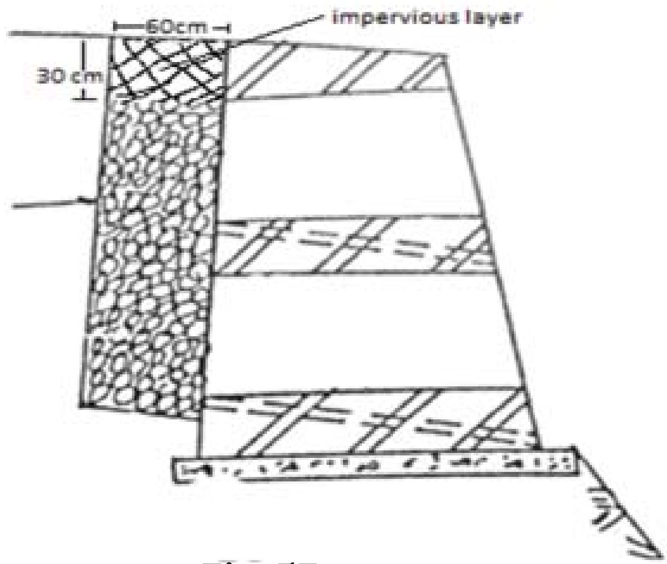


Fig. 57

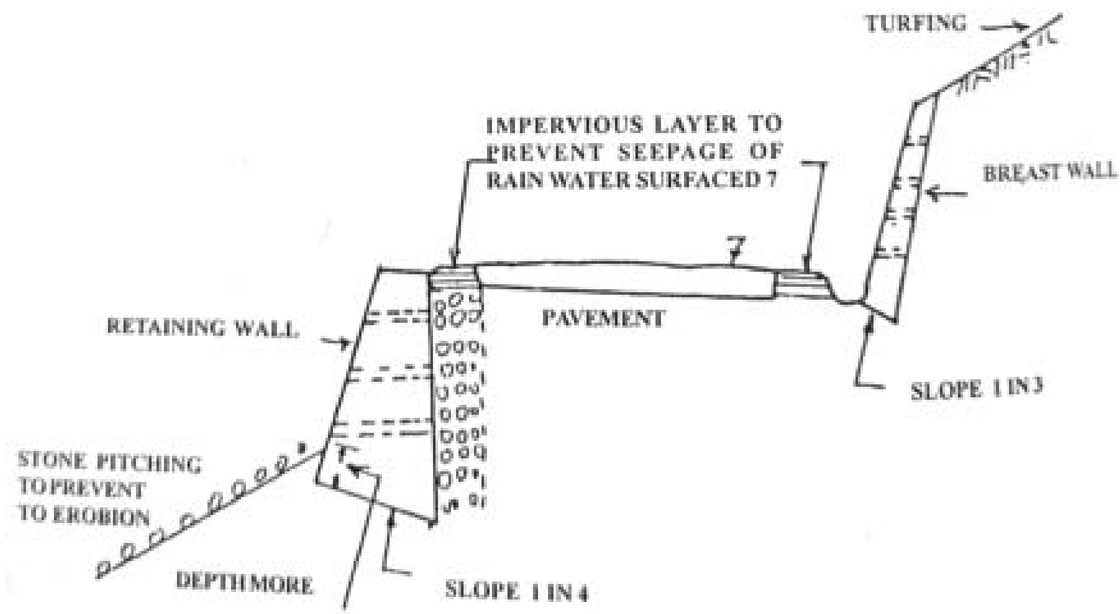


Fig 58

27.2 **Collection arrangement in backfill.** A simple system of weep holes should be adequate in freely draining (permeable) backfill. If the backfill material is not freely draining (in most cases the backfill is not freely draining) to make the simple drainage system (referred to above) effective, permeable material must be arranged in pockets or blankets to facilitate collecting the water and convey it to weep hole.

27.3 A pocket is a deposit of coarse material place around the end of each weep hole on the inside end to facilitate the drainage of a backfill which has a considerable degree of permeability. These pockets are also called pocket drains.

27.4 These pocket drains can be constructed by installing a pocket of crushed stone or gravel having a volume not less than 1/18 cum. (2 cft). Fig. 60 refers.

27.5 In order to increase the effectiveness of a drainage system where the backfill is not sufficiently permeable, the entire back of retaining wall should be covered with a blanket drain (a layer of previous backfill immediately behind the wall) about 45 cm thicken with its lower edge joining the inner ends of the weep holes, as shown in Fig. 61. The upper edge of the blanket should be covered with relatively impervious material to prevent surface water, from entering the drainage system directly. It will consist of broken stone, rubble or any other granular material.

Note: The quantity of water that percolates through a well-constructed backfill is so small that there is no danger of the drains (drainage system like pocket drains, blankets, etc.) becoming obstructed by washed out soil particles. Therefore, it is not necessary that the grain size of the materials in the drainage layers (like pockets, blankets etc.) should satisfy the requirement for filter layers.

28. FILLING OF JOINTS BY GRANULAR MATERIALS

28.1 In dry stone masonry, it is necessary to spread earth, gravel sandy soil, small stone pieces or ballast in each course layer of masonry for filling interstices, cavities, gaps between masonry stone layers. It helps the structure in following way:-

(a) The load distribution becomes uniform and on a wider surface.

(b) The frictional resistance between stones increases due to interlocking of stones and resistance against sliding gets improved.

(c) It increases the weight of wall by 10% to 20% which would increase the resistance against sliding effect of wall.

(d) Such steps would reduce the effect of seepage through the voids.

29. DRAINAGE

29.1 Adequate provision must be made to prevent water accumulating behind the wall. Adequate weep holes not less than 7.5 cm (3 inches) square should be provided to prevent weep hole getting the inlet of all weep-holes. In wet situations, a continuous loose stone drain should connect the weep-holes.

29.2 Catch water drains, to minimize the flow of water towards the area/ zones of the retaining structure be provided, where necessary. Catch water drain must be provided in such a way as to avoid accumulation of moisture in the soil mass immediately close to the breast/retaining walls.

29.3 Catch water drains shall be avoided near the top of the breast walls as catch water drains cause seepage of water in un-maintained conditions into the cut slope and destabilize it. If necessary the catch water drains may be provided far away from breast walls for above reasons. A water drain shall be provided at the toe of the breast wall to collect water from weep holes and surface run off of the slope.

30. DRAINAGE OF RETAINING/TOE WALLS

30.1 The primary function of drainage behind a retaining wall is to prevent water from accumulating in the backfill and coming in contact with the back of the wall and thereby subjecting the wall to hydrostatic pressure. However, any increase in earth pressure, which may be caused by pore pressure needs to be considered.

30.2 Excessive water pressure behind a wall is the common cause of failure. In the interest of safety, durability and economy in the design of retaining walls, it is necessary to provide effective drainage behind walls.

30.3 The sources of ground water are as follows:-

(a) Surface water due to rainfall or from other sources, which may flow to the wall from adjacent area and seep in to the backfill.

(b) Rain water falling on the ground surface behind the wall and seeping into the backfill.

(c) Under ground water from remote sources.

30.4 The first element in a drainage system for a retaining wall is adequate surface drainage to dispose of surface water. The second element is a relatively impervious layer of soil on the top of the backfill to reduce the amount of water, which seeps in to the backfill. The third element consists of a freely draining backfill, pockets, channels or blankets of highly permeable material introduction into the backfill to collect ground water which may seep into the backfill from any or all the sources mentioned above. The final element includes the conveying units such as weep holes through the walls which convey the water from the collecting drain through the wall and discharge it in front of the wall.

30.5 There are various types of collecting and conveying systems. The simplest system consists of permeable pockets in the backfill for collection and Weep holes for conveyance.

31. WEEP HOLES

31.1 The specifications for weep holes will be as under:-

(a) Weep holes shall be 8 cm wide. The height of the weep holes shall be the same as the height of the coarse in which they are formed.

(b) For abutments/wing walls and in case of culverts, the weep holes shall be 8 cm wide and 15 cm high.

(c) Weep holes shall extend through the entire width of the masonry with slope of about 1 vertical to 20 horizontal towards the drainage face.

(d) The spacing of weep holes shall not exceed 1.5 m center to center both horizontally and vertically and shall be staggered.

(e) The lowest row of weep holes will be located at about 1.5 cm above the low water level or ground level whichever is higher as applicable.

(f) The sides and bottom of the weep holes in the interior shall be made up with stone masonry having a fairly plain surface and the channel so formed slabed over with stones not less than 15 cm thick and having a bearing of not less than 15 cm on each side.

(g) Sometimes weep holes made in cement stone masonry get clogged. To avoid clogging and choking of weep holes PVC flexible pipes may be provided. The size of weep holes shall be 100 mm to 150 mm PVC (flexile) Pipes and shall be embedded at 10^0 down from the horizontal towards valley side to effectively drain the water from ground. (Para 6.5.2 IS 14458 (Para 2): 1997)

(h) The above applies to the RR masonry in cement mortar. For the dry masonry portion of any banded retaining/breast/toe wall, no weep holes be provided as gaps between the stones act as natural holes in those case. However, in this case particularly toe material immediately adjacent to the wall should be non-cohesive and free draining. Fig. 59, 60, 61 shows the arrangement for provision of weep holes and pocket/blankets.

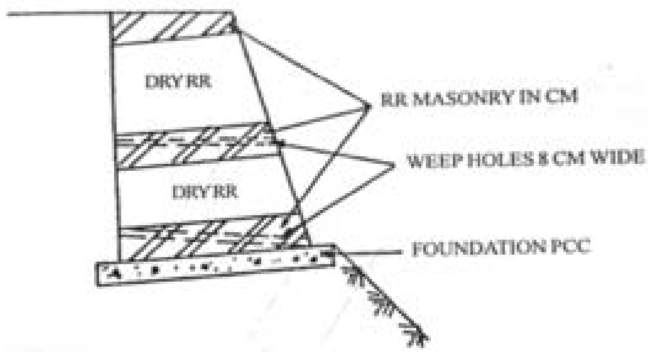


Fig. 59

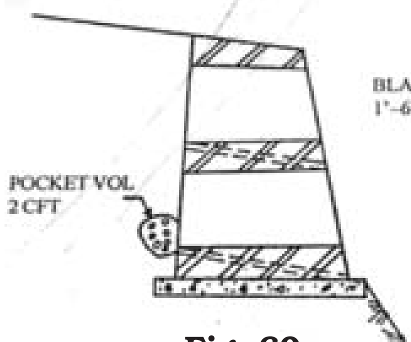


Fig. 60

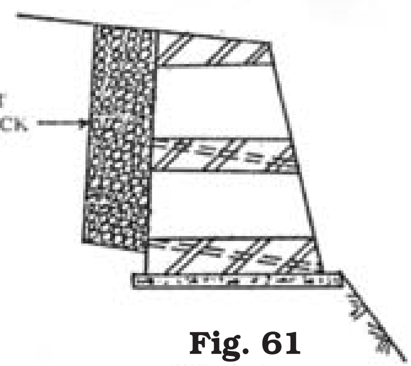


Fig. 61

32. CAUSES OF FAILURE OF RETAINING WALLS AND ITS REMEDIAL MEASURES

32.1 **Construction of successive Retaining walls**. Construction of Retaining Walls one over the other and sometimes giving some gap in continuity and closely located should be avoided.

32.2 **Alignment of Walls Section.** The location, alignment and section of wall should be carefully examined and finalized with the help of wire threads, wooden poles and pegs indicating excavation lines slope and base width should be recorded.

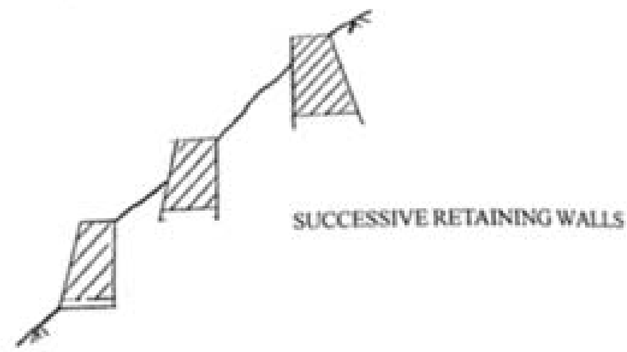


Fig. 62 Successive Retaining Walls

32.3 **Depth of foundation & Base of Retaining Walls.** Most of the retaining walls fail due to base failure by sinking, sliding out away or creep action or heaving out of soil rock below toe of wall. Depth of foundation in soil or SMB or soft erodible rock zone or badly fractured rock zone should be about ($\frac{H}{10}$ + 0.3m for H m high Retaining wall.

32.4 **Toe Walls.** Due to maximum pressure of foundation level suitable toe wall should be provided in soil or erodible soft rock. Due to erosion and heaving out of soil mass under the toe of retaining wall, masonry would disintegrate and fall apart leading to collapse of wall.

32.5 **Construction of Retaining or Breast Walls is to be taken up.** Construction of R/walls or breast Walls should be started after at least one/two rainy seasons to allow settling down of disturbances caused by cutting blasting. Construction should not be started until and unless hill side has stabilized.

32.6 **Reduction of Section in Rocky Zones.** In stable rocky zones, section can be reduced in such a way as to keep the masonry well interlocked into the hill face. It is not necessary to do excavation or rock blasting for getting full section.

32.7 **Disposal of Foundation Excavated Soil Mass.** Excavated soil mass should not be dumped and left deposited on lower side of wall base on the original hill face creating humps and undulation. There create pot holes and ditches depression and ponding effect in front of wall toe where rain water ponds and softens the base soil rock. Correct way of disposal of soil mass has been shown in Fig. 63 (b).

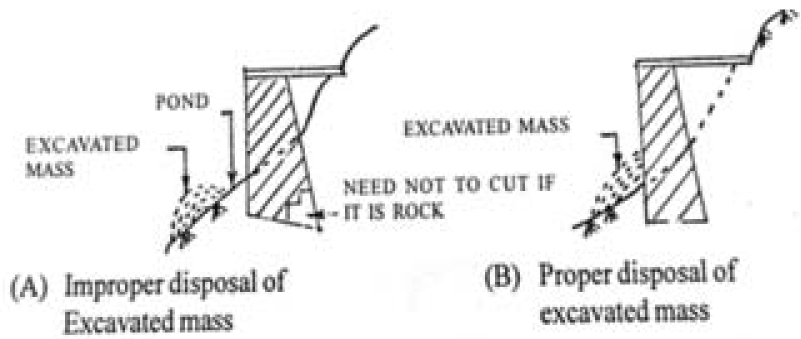


Fig. 63

32.8 **Stone Filling on stepped Surface.** Hand packed stone filling should be done after providing steps in the sloping face to give a good seat to the fill material and to avoid creation of any back pressure by slipping of stone fill masses Top bearing surface should be impervious.

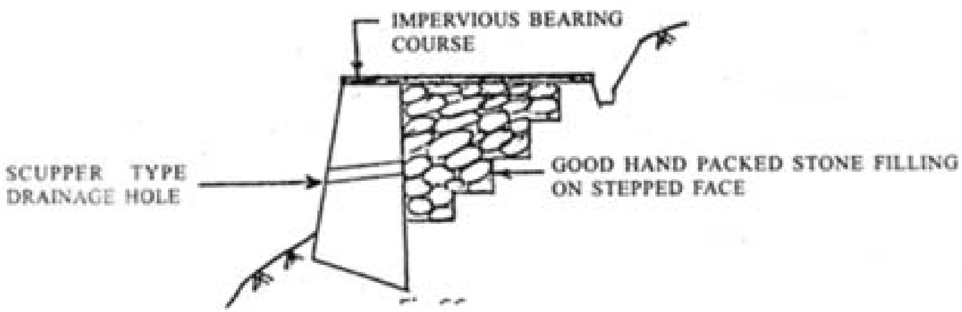


Fig. 64

33. TYPICAL FAILURE OF RETAINING WALLS AND TREATMENT

33.1 **When Some Masonry stones have split away :-**

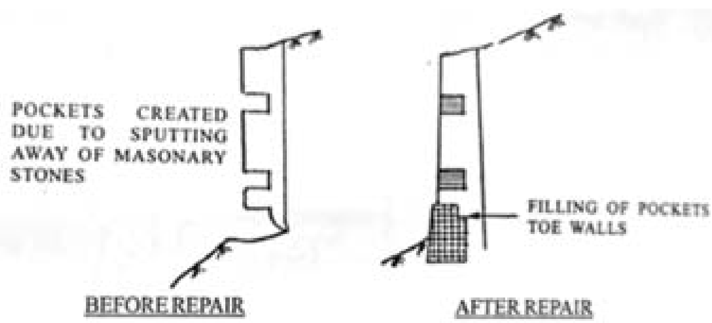


Fig. 65

(a) Empty pockets get developed in the retaining walls section due to splitting away of a few of the masonry stones.

(b) Stones should be placed and fitted tightly so that they well interlocked with present masonry stones.

(c) A proper toe wall should be constructed near toe if such failure occurs near the base.

33.2 **Failure of Base or Sinking of Base.** This occurs when pressure on foundation exceeds the bearing capacity of the soil.

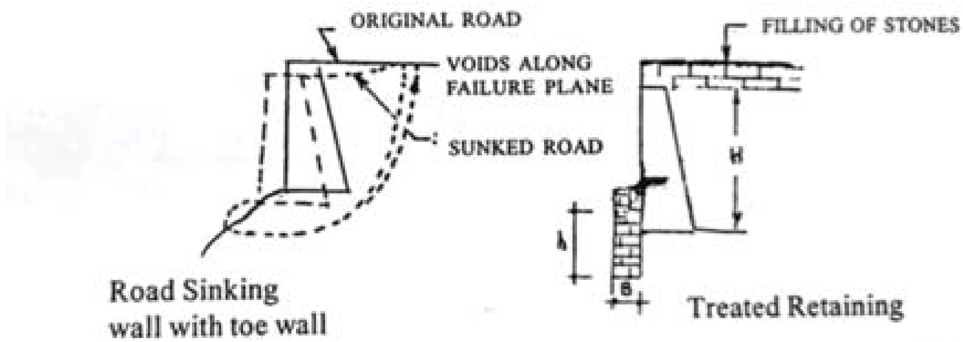


Fig. 66

(a) Additional Toe wall in front of original wall is to be provided.

(b) Depression in the surface at top is filled hand packed stone filling up to the required original road level.

Table - 12

Dimension of Toe Wall for Treatment of Damaged Walls

Height of Retaining wall 'H' in m	Height of Toe wall 'H' in m	Width of Toe wall 'B' in m
3	1.0	0.6
4	1.2	0.8
5	1.5	1.0
6	1.8	1.2
7	2.1	1.5
8	2.4	1.8
9	2.7	2.1
10	3.0	2.4

The dimension of above table could be modified suitably according to failure condition and site requirements.

33.3 **Heaving out of foundation soil mass.**

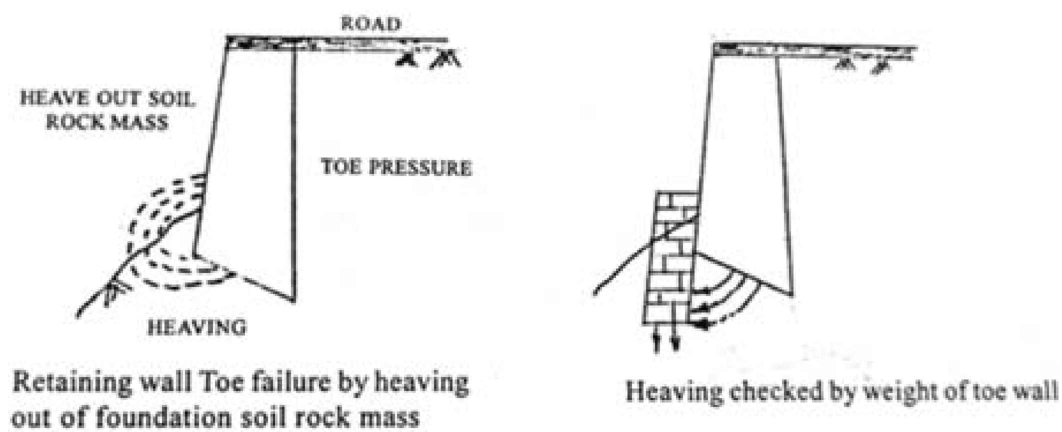


Fig. 67

33.3.1 The damage can also be treated by surface grouting in case sufficient space is not available for construction of Toe wall.

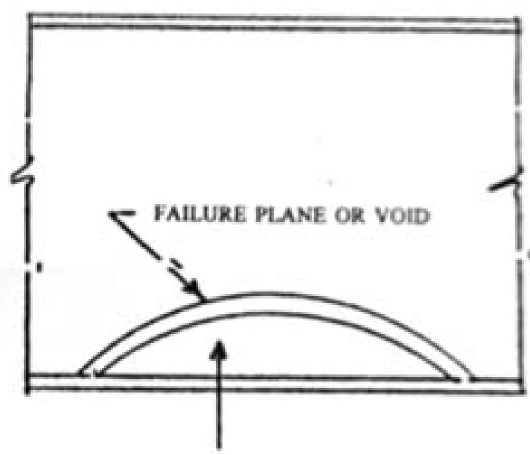


Fig. 68

(a) Grouting pits 0.3 m long x 0.3 m wide and 0.6 to 1 m deep should be dug at spacing 1.0 to 2.0 at fissures or cracks.

(b) When cracks or fissures are wider than 1.0 cm the 1 part of cement and 1 part of water mixed grout should be poured in cracks in alternate pits, till the whole grout is absorbed or soaked.

(c) The quantity of grout be restricted to 1 bag of cement per running meter length on first day and allow it to set for at least 3 days and thereafter pour same mixture in remaining pits.

(d) Fill up all the pits by compacting soil, boulders etc. properly at top level.

(e) Cement water mix should be poured in case of soil or sandy soil and fracture broken rocks.

(f) In case of fine soil beds pouring mixture of 1 part of lime 1 part of cement and 2 part of water is better alternative.=

(g) Quantity of cement water slurry of lime water (slurry) can be assessed as 1% to 5 % of soil rock quantity which is to be strengthened.

(h) The requirement of cement could vary from 1 to 5 bags per running meter length of retaining wall.

33.4 **The failure due to shearing of slopes**

(a) Slope of that failure resembles to the spheroid or parabolic shape.

(b) Wide crack may develop on the surface as failure progresses and the crack would become wider as failure increases.

(c) Damages of this type can be treated by construction or a toe wall or by grouting of cracks and fissures.

33.5 **Sinking of Large part of Hill surface or road with or without Retaining Wall**

(a) It occurs due to excessive ingress of surface water and or by soil erosion at or near foot of hill due to poor drainage, inadequate camber or shoulder slope.

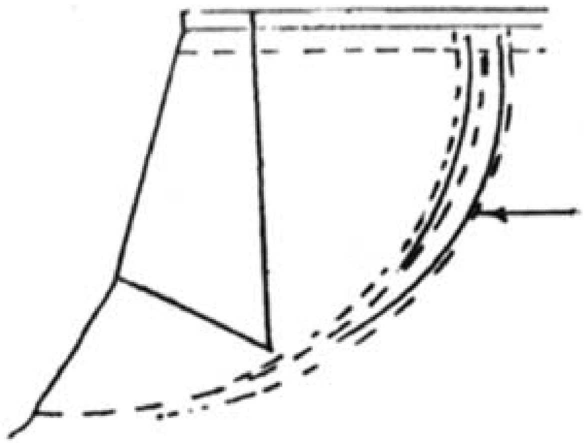


Fig. 69

(b) Probable honey combed solid Strengthened mass bulk

(c) These damages can be strengthened by surface grouting using unslacked lime or cement or by proper construction of toe wall.

(d) Turfing Or making surface impervious by pitching etc. can prevent future failure.

33.6 Bulged Retaining Walls

(a) These failure resembles to failure of a column in building or shearing failures as shown below:

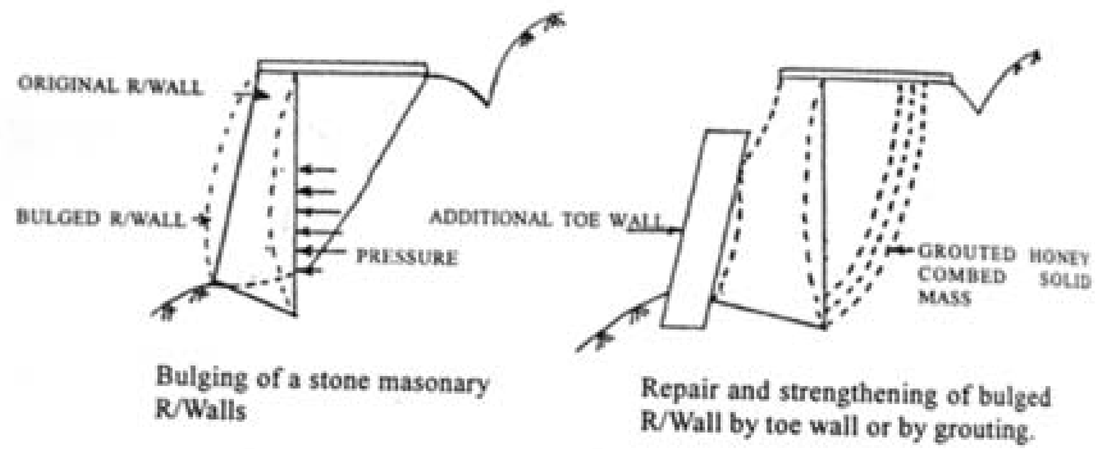


Fig. 70

(b) This type of damage can be treated by construction of suitable toe wall or by grouting or both.

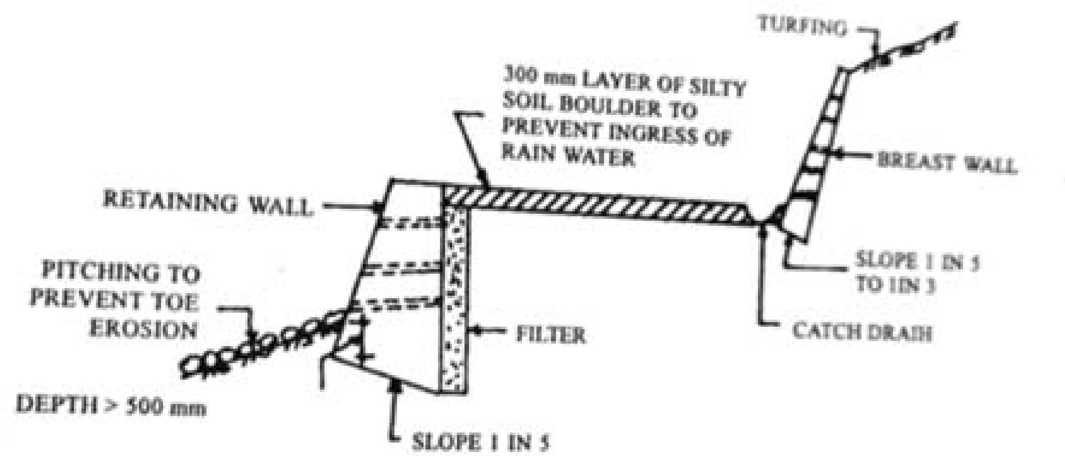
(c) When sufficient space is not available to permit construction of toe wall then surface grouting is the only dependable solution.

34. EROSION CONTROL OF TOE OF RETAINING WALLS

34.1 Water flows at a high speed from high retaining wall (having height more than 3.0 m). This may lead to toe erosion of soft rocks/soil at foundation. Following measures be adopted for prevention of toe erosion and any increase in the moisture content in and below foundation.

(a) Earth extracted from excavation of foundation lying on hill slope at time may cause obstruction to free flow of water or pounding. It must be spread properly to achieve gradual slope from face of wall downwards.

(b) Dry stone pitching from wall face downwards be done as shown in figure 4 & 5. Stones of 15 cm size may be laid on the slope for a distance of 1.00 meter below the toe of the retaining walls.



DEVELOPMENT OF ROAD WITH RETAINING WALL AND BREAST WALL

Fig. 71

35. CONCLUSION

35.1 Incorrectly designed and constructed retaining structures are a great source of weakness in hill roads. Special care must be taken that specifications and design for these structures are sound and safe. At times, complete retaining wall usually fall due to errors in construction of wall such as:

(a) Settlement of the toe, due to too shallow foundation or due to the presence of water at the toe or toe projection not being provided to reduce the pressure on foundation.

(b) Lack of proper drainage like omission of weep holes etc.

(c) Carelessly placed back-fill not properly rammed.

(d) Insufficient bonding in masonry courses.

(e) Masonry course not at right angle to the face of the wall.

(f) Inadequate base width etc.

REFERENCES

1. IRC:SP:48-1998 "Hill Road Manual".

2. "Treatise on Hill Roads" by R.S. Gahlowt (Padam Shree) and V.P. Gupta Standard Book House.

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4. IRC:6-2014 Standard Specification and Code of Practice for Road Bridges Section II Loads and Stresses

5. IRC:78-2014 Standard Specifications and Code of Practice for Road and Bridges Section: VII Foundation and Sub structure.

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7. RCC Design (Reinforced concrete structure) by Dr. B C Punmia, Laxmi Publications (P) Ltd.

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