PREFACE

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(Nand Kishore)
Executive Director, Geo.Tech., RDSO
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1.0 Introduction

The concept of earth reinforcement is not new; the basic principles are demonstrated abundantly in nature by animals and birds and through the action of tree roots. The fundamentals of the technique are described in the Bible, covering the reinforcement of clay or bricks with reeds or straw for the construction of dwellings. Constructions using these techniques are known to have existed in the 5th and 4th millenniums B.C. Since early civilization, man has attempted to use soil with other materials to enable it for being used for his necessities. Typical early use include use of branches of tree etc. to support tracks over marshy areas and to build hutsments. Structures are also built by insects and birds using mud and leaves. These are all familiar sights even today. This kind of principle is also used in building parts of Great Wall of China and the Babylonian ziggruaunts i.e. temples. In the 19th century Passel used tree branches to reinforce back fills in order to reduce the earth pressure and thereby economise the retaining walls. Textile material was perhaps first used in road construction in South Carolina in the early 1930s. The first use of woven synthetic fabrics for erosion control was made in 1958 by Barrett.

The reinforcement improves the earth by increasing the bearing capacity of the soil and reduces the settlement. It also reduce the liquefaction behaviour of the soil. The construction of reinforced earth structure has become widespread in Geotechnical engineering practice in the last two decades owing to their ease of construction and economy compared to those of conventional methods. Reinforcement of soil, is practised to improve the mechanical properties of the soil being reinforced by the inclusion of structural element such as granular piles, lime/cement mixed soil, metallic bars or strips, synthetic sheet, grids, cells etc.

2.0 Scope

This report covers the earth reinforcement techniques by way of providing suitable material. It contain the design methodology of reinforcement of earth retaining walls, steep slopes.

3.0 Reinforced Earth

The concept of combining two materials of different strengths characteristics to form a composite material of greater strength is quite familiar in civil engineering practices and is in use for ages. The reinforced concrete constructions are examples for such composite materials. It combines the high tensile strength of steel with the high compressive, but relatively low tensile strength of concrete. Likewise, soils which have little if any tensile strength can also be strengthened by the inclusion of materials with high tensile strength. This mobilisation of tensile strength is obtained by surface interaction between the soil and the reinforcement through friction and adhesion. The reinforced soil is obtained by placing extensible or inextensible materials such as metallic strips or polymeric reinforcement within the soil to obtain the requisite properties.
Soil reinforcement through metallic strips, grids or meshes and polymeric strips sheets is now a well developed and widely accepted technique of earth improvement. Anchoring and soil nailing is also adopted to improve the soil properties.

The use of reinforced earth technique is primarily due to its versatility, cost effectiveness and ease of construction. The reinforced earth technique is particularly useful in urban locations where availability of land is minimum and construction is required to take place with minimum disturbance traffic. The various applications of reinforced earth is shown in Fig 1.

Fig 1 : Range of application of reinforced earth
4.0 Fundamental Principles

Limit state principle is adopted for design of reinforced earth in this document. Two limit states are considered in the design:

i) ultimate limit state

ii) Serviceability limit state

Ultimate limit state is associated with collapse or other similar forms of structure failure. Margins of safety against attaining limit state of collapse is provided by use of partial material factor and partial load factor. Disturbing forces are increased by multiplying by prescribed load factor to produce design loads. Restoring forces are reduced by dividing by prescribed by factor to produce design strength.

Serviceability limit states are attained if the magnitudes of deformation occurring within the design life exceed prescribed limits.

4.1 Partial factors

Limit state design for reinforcement earth employs three principal partial factors. The factors are as given below:

(i) Load factor:
   Dead load factor \( f_t \)
   Live load factor \( f_q \)

(ii) Material factor \( f_m \)

(iii) Factor to take account economic ramification \( f_r \)

The numerical value above factor is of unity or greater. The ranges of these values depend upon the type of structure, mode of loading and its design life.

In ultimate limit state of collapse, potential failure mechanisms will vary from one application to another.

4.2 Design Strength of reinforcing Material

Design strength of reinforcing material should be equal to or greater than the design load. In case of external stability the design load may be resisted by forces generated in the soil. Resisting forces will be a function of several variables including pore water pressure and soil shear strength;

In case of internal stability design load may be resisted by forces generated in the soil and reinforcement.

A design strength may be determined by dividing the unfactored reinforcement base strength by a prescribed value of the partial material factor \( f_m \). If the economic consequence of failure is high the derived design strength may be further reduced by partial factor \( f_n \). The partial factors are further discussed in detail in Para 6 of this report.
4.3 Fundamental mechanisms

Soil has an inherently low tensile strength but a high compressive strength. An objective of incorporating soil reinforcement is to absorb tensile loads or shear stresses within the structure. In absence of the reinforcement, structure may fail in shear or by excess of the deformation.

When an axial load is applied to the reinforced soil, it generates an axial compressive strain and lateral tensile strain. This is illustrated by model in Fig 2. If the reinforcement has an axial tensile stiffness greater than that of the soil, then lateral movements of the soil will only occur if soil can move relative to the reinforcement. Movement of the soil, relative to the reinforcement, will generate shear stresses at the soil/reinforcement interface, these shear stresses are redistributed back into the soil in the form of internal confining stress. Due to this, the strain within the reinforced soil mass is less than the strain in unreinforced soil for the same amount of stresses, this is indicated in Fig 2 where $\delta_{hr} < \delta_{h}$ and $\delta_{vr} < \delta_{v}$, provided the surface of the reinforcement is sufficiently rough to prevent the relatively movement and the axial tensile stiffness of reinforcement is more than that of soil.

![Fig 2: Effect of reinforcement on a soil element](image)

4.4 Soil reinforcing mechanisms in walls and slopes

Fig 3 shows a steep slope in a dry cohesion less soil with a face inclined at $\beta_s$ to the horizontal, where $\beta_s$ is greater than the internal angle of shearing resistance. Without the benefit of soil reinforcement the slope would collapse, however by the incorporation of suitable soil reinforcement the slope may be rendered stable. Investigation of the basic reinforcing mechanism reveals that the soil in the slope comprises two distinct zones, viz. active-zone and the resistant zone as shown in Fig 3. Without reinforcement the active zone is unstable and tends to move outwards and downwards with respect to the resistant zone. If soil reinforcement is installed across the active and resistant zones it can serve to stabilize the active zone. Fig 3 shows a single layer of reinforcement with a length $L_{aj}$.
embedded in the active zone and length $L_{ej}$ embedded in the resistant zone. A practical reinforcement layout would contain multiple layers of reinforcement, however the single layer shown in Fig 3 is adequate to illustrate the basic mechanisms involved. The embedded length of $L_{ej}$ should be sufficient enough to mobilise enough bond strength between the soil and reinforcement to resist the disturbing force caused by active zone. The tensile strength of the reinforcement is not constant but it decreases towards the free end of $L_{ej}$ and it is zero at the end.

Flexible reinforcement is incorporated in fill during construction. Consequently, the layers of reinforcement are horizontal. Flexible reinforcement is also inserted into cut sections during construction (in the form of soil nails) at inclinations close to the horizontal. This inclination is convenient since it coincides with the general inclination of the tensile strength developed in the soil in the active zone.

![Mechanism in Walls & Slopes](image)

**Fig 3:** Mechanism in Walls & Slopes

### 4.5. Soil reinforcement interaction

For soil reinforcement interaction to be effective reinforcement is required absorbs strains which would be otherwise cause failure. In this context an ultimate state of collapse in terms of interaction with the soil and reinforcement this state can be bought by rapture of reinforcement or failure of bond between soil and reinforcement. In serviceability limit state is occurred when deformation occur beyond serviceable limit or strain within the reinforcement exceed prescribed limit.

If the soil is cohesion less the bond resistance will be friction and will depend upon surface roughness and soil. If soil is cohesive the bond stress will be adhesive.

In case of grid reinforcement the bond stress will be governed by the shear strength of the soil and roughness of the reinforcement.
Having absorbed load it is necessary for the reinforcement to sustain this load during the design life without rupture or without suffering time dependent deformation which might give rise to serviceability limit. To maximise the tensile load capacity the flexible reinforcement are install horizontally to coincide with the principle tensile strain. The axial forces absorbed by the reinforcement are statically determinate.

4.6 Soil Properties required to be taken into consideration

The soil property required to be taken into consideration are effective shear strength parameter c’ and $\phi'$ which are obtained by taking into consideration pore pressure within the soil. However shear strength of fill or soil incorporating multiple layer of reinforcement.

In wall and slope load imposed on the soil reinforcement will increase if positive pore water pressure are allowed to develop. The development of adverse pore pressure in reinforcement fill wall can be prevented by providing appropriate drainage. In case the development of pore water pressure is unavoidable in water front construction increased reinforcement is required to be considered.

In addition to physical interaction of soil and reinforcement electrochemical interaction is also required to be considered for design life to assess the durability.

4.7 Reinforcement Geometry

Soil reinforcement can take a variety of forms, some of which are shown in Fig 4. Grids meshes and strips can be metallic or polymeric whilst sheet reinforcement takes the form of polymeric geotextiles. Anchored earth fill employs multiple layers of flexible steel bars or polymeric materials, which are shaped, at the end remote from the face of the wall, to form an anchor. When used as soil nails, steel bars have a simple circular cross section.

Sheet reinforcement, and polymeric grids are generally installed full width, such that each metre length of face is associated with a 1 m width of reinforcement, and so, in a multiplayer system, the total stabilizing force developed by the reinforcement is a function of the number of layers of reinforcement and their vertical spacings. Strip reinforcement, including wide strips of metallic or polymeric grid, are not placed full width.

Consequently, the total stabilizing force developed by such reinforcement will be a function of the number of reinforcement elements and both their horizontal and vertical spacing.

The total length of each reinforcing element will influence the overall geometry of the reinforced mass and this in turn will influence external stability. For example, in the case of a reinforced fill wall, the length of the reinforcing elements at the base of the wall determine the width of the base of the wall and therefore affect the performance of the reinforced mass with respect to forward sliding on the base, bearing, tilting, settlement and overall stability.
4.8 Reinforcement Bond

The length of the reinforcement depends upon the bond characteristic which in turn decide the load carrying capacity and the length of the reinforcement should be sufficient to prevent pull out the resisting zone. In assessing the bond performance for design purpose the shear stress develop between soil and reinforcement are assumed to be uniformly distributed over the entire length even though actually it is not uniform. The shear stresses is taken to be the product of vertical stress multiply by coefficient $\mu$ or $\tan \delta'$ where $\delta'$ is the angle of bond stress. For narrow rough strip reinforcement embedded in cohesion less fill dilatancy in the fill causes vertically effected stresses to rise locally give rise to enhanced pull-out resistance.

4.9 Factors affecting tensile behaviour of flexible reinforcement

The strength of the reinforcing will reduce with passage of time in case of metallic reinforcement which is in practice galvanized steel will be affected by the corrosion. Causes reduction in tensile rupture load therefore the design strength to be considered is the strength of the reinforcement available at the end of design life. The affect of the corrosion in metallic reinforcement taken into account by allowing prescribed sacrificial thickness which vary according to the design life.
5.0 Components of Reinforced Earth Structure:

Reinforced earth structure consists of three main components shown in Fig 5, namely

i) Reinforcing element
ii) Soil back fill
iii) Facing element

5.1 Description of Reinforcing element:

A variety of materials can be used as reinforcing materials. Those that have been used successfully include steel, concrete, glass fibre, wood, rubber, aluminium and thermoplastics. Reinforcement may take the form of strips, grids, anchors & sheet material, chains planks, rope, vegetation and combinations of these or other material forms.

Types of reinforcing materials:

i) Strips:
These are flexible linear element normally having their breadth, ‘b’ greater than their thickness, ‘t’. Dimensions vary with application and structure, but are usually within the range t = 3-5 mm, b = 5-100 mm. The most common strips are metals. The form of stainless, galvanized or coated steel strips being either plain or having several protrusions such as ribs or gloves to increase the friction between the reinforcement and the fill. Strips can also be formed from aluminium, copper, polymers and glass fibre reinforced plastic (GRP). Reed and bamboo reinforcements are normally categorized as strips, as are chains.
ii) Planks:
Similar to strips except that their form of construction makes them stiff. Planks can be formed from timber, reinforced concrete or prestressed concrete. The dimensions of concrete planks vary; however, reinforcements with a thickness, 't' = 100 mm and breadth, 'b' = 200–300 mm have been used. They have to be handled with care as they can be susceptible to cracking.

iii) Grids and Geogrids:
Reinforcing elements formed from transverse and longitudinal members, in which the transverse members run parallel to the face or free edge of the structure and behave as abutments or anchors as shown in Fig 7. The main purpose is to retain the transverse members in position. Since the transverse members act as an abutment or anchor they need to be stiff relative to their length. The longitudinal members may be flexible having a high modulus of elasticity not susceptible to creep. The pitch of the longitudinal members, \( p_L \) is determined by their load-carrying capacity and the stiffness of the transverse element. The pitch of the transverse elements, \( p_T \) depends upon the internal stability of the structure under consideration. A surplus of longitudinal and transverse elements is of no consequence provided the soil or fill can interlock with the grid. Mono and Bi Oriented grid as shown in Fig 6.

![Fig 6: Geogrid](image)

(a) Mono Oriented geogrid  
(b) Bi-Oriented geogrid

![Fig 7](image)
Grids can be formed from steel in the form of plain or galvanized weldmesh, or from expanded metal. Grids formed from polymers are known as “Geogrids” and are normally in the form of an expanded proprietary plastic product.

iv) Sheet reinforcement:  
May be formed from metal such as galvanized steel sheet, fabric (textile) or expanded metal not meeting the criteria for a grid.

v) Nailing:  
Earth may be protected by geosynthetics with earth nailing.

vi) Anchors:  
Flexible linear elements having one or more pronounced protrusions or distortions which act as abutments or anchors in the fill or soil. They may be formed from steel, rope, plastic (textile) or combinations of materials such as webbing and tyres, steel and tyres, or steel and concrete as shown in Fig. 8.

![Fig 8](image)

vii) Composite reinforcement:  
Reinforcement can be in the form of combinations of materials and material forms such as sheets and strips, grid and strips and anchors, depending on the requirements.

In reinforcement with polymers, polymeric joints are required. Polymeric reinforcement joints are subdivided into prefabricated joints and joints made during execution of the works. A number of different jointing systems are in use. Joints in geotextiles should normally be sewn where load transference is needed. For polymeric meshes or grid a bodkin may be employed. A Bodkin joint is an effective method of joining some polymeric grid reinforcement as shown in Table 1 and also in Fig 9.

<table>
<thead>
<tr>
<th>Material</th>
<th>Joint type</th>
<th>Load carrying efficiency %</th>
<th>Displacement mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polymeric geogrid</td>
<td>Bodkin (see Fig. 16)</td>
<td>&gt; 95</td>
<td>3 to 15</td>
</tr>
<tr>
<td></td>
<td>Lacing</td>
<td>Dependent upon method</td>
<td></td>
</tr>
</tbody>
</table>
Care should be taken to ensure that:

- Bodkins have sufficient cross-sectional area and strength to avoid excessive deformation;
- Bodkins are not so large as to distort the parent material causing stress concentrations;
- Joints are pre-tensioned prior to loading, to reduce joint displacement as the components lock together.

5.2 Description of Soil Backfill

The fill material for reinforced earth structures shall be preferably cohesionless and it should have an angle of on interface friction between the compacted fill and the reinforcing element of not less than 30, measured in accordance with IS 13326 Part (I). The soil should be predominantly coarse grained; not more than 10 percent of the particles shall pass 75 micron sieve. The soil should have properties such that the salts in the soil should not react chemically or electrically with the reinforcing element in an adverse manner.

A wide variety of fill types can be used with the grids including crushed rock, gravel, industrial slag, pulverised fuel ash and clay, but fill particles greater than 125 mm should be avoided.

5.3 Description of Facing element

Facings may be ‘hard’ or ‘soft’ and are selected to retain fill material, prevent local slumping and erosion of steeply sloping faces, and to suit environmental requirements.

The facing shall comprise of one of following:

(i) reinforced concrete slabs
(ii) plain cement concrete form fill hollow block (precast)
(iii) masonry construction, rubble facia
(iv) other proprietary and patented proven system
Common facing used with structure are shown in Fig.10

![Fig 10: Common facing used with structure](image1)

5.3.1 Hard facings
Facing may consist of concrete, steel sheet, steel grids or meshes, timber, proprietary materials or combination of these. They should conform to the appropriate material standard and should be sized by normal design procedures using the appropriate standard.

Interlocking concrete blocks, grout filled bags or Gabions can provide a substantial facing. These facing shown in Fig.11

![Fig 11: Hard Facing](image2)
5.3.2 Soft facings

Generally, external temporary formwork is erected to support the face during the construction of steep slopes ($>45^\circ$). It can take the form of a lightweight system of scaffold tubes and boards or consist of some form of ‘climbing’ shutter. The grids are turned up the face of the framework and returned into the embankment directly below the next reinforcement layer. The two grids are connected using a high density polyethylene bodkin. The soft facing shown in Fig 12.

![Fig 12: Soft Facing](image)

Turf and topsoil can be placed on the fill side of the grid reinforcement as it is turned up the face of the slope to create a natural and aesthetic appearance.

Where the vertical spacing of the main reinforcement is greater than 500mm, biaxial grid reinforcement is used as intermediate secondary reinforcement to provide local stability at the face of the slope.

5.4 Fasteners between the facing and reinforcing elements

Fasteners are used to make a connection between the reinforcement and the facing and take the form of dowels, rods, hexagon headed screws and nuts and bolts and may consist one of the following materials:

- Plain steel
- Coated steel
- Galvanized steel
- Stainless steel
- Polymers

The choice of material used to form the fastener should be compatible with the design life of the structure.

5.5 Drainage

If the embankment becomes waterlogged and pore water pressures increase, the magnitude of the tensile forces induced into the grid reinforcement also increases. Pore
water pressures can be controlled by providing drainage layers at the back of the reinforced zone in combination with an underdrain as shown in Fig. 13.

Fig 13: Facing with drainage
6.0 Design Principles

6.1 General
By its nature reinforced soil is a combination of structural and geotechnical engineering. The evolution of limit state design in structural engineering has led to the definition of a number of partial load factors which are applied to loads in design combinations and material factors which are applied to the structural components. In geotechnical engineering the application of partial factors to the various geotechnical parameters has not been found practical in general design and overall factors of safety are still used.

For the purposes of reinforced soil design, a limit state is deemed to be reached when one of the following occurs:-

a) collapse or major damage;
b) deformations in excess of acceptable limits;
c) other forms of distress or minor damage, which would render the structure unsightly, require unforeseen maintenance or shorten the expected life of the structure.

The condition defined in (a) is the ultimate limit state, (b) and (c) are serviceability limit states.

6.2 Service life
The service life of reinforced soil structures should be considered in design. In most applications the selected design life of the reinforcing elements is equal to the service life of the structure. In certain cases, mostly foundations to embankments, the entire structure can have a long term service life but it may only be necessary for the reinforced portion to function for a shorter time while the surrounding ground gains strength.

Table 2 gives examples for the categorization of the service life of reinforced soil for a variety of applications.

<table>
<thead>
<tr>
<th>Category</th>
<th>Typical service life (years)</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary works</td>
<td>1 to 2</td>
<td>Contractors site structures</td>
</tr>
<tr>
<td>Short term</td>
<td>5 to 10</td>
<td>Contractors site structures Basal reinforcement</td>
</tr>
<tr>
<td>Industrial</td>
<td>10 to 50</td>
<td>Structures at mines</td>
</tr>
<tr>
<td>Long term</td>
<td>60</td>
<td>Marine structures and highway embankments</td>
</tr>
<tr>
<td>Long term</td>
<td>70</td>
<td>Retaining walls</td>
</tr>
<tr>
<td>Long term</td>
<td>120</td>
<td>Highway retaining walls and highway structures and bridge abutments to DoT requirements.</td>
</tr>
</tbody>
</table>
6.3 Factors of safety

6.3.1 The partial factor are required to be applied at appropriate stage in the design to obtain overall factor of safety to the reinforced structure. The partial factor required to be considered in design are as follows:

6.3.2 Economic ramification factor (fn):
The assigned value depend upon the risks which particular structure is subjected to. The value given in BS 8006:1995, Table 3.

6.3.3 Partial factor:
6.3.3.1 Partial material factors for metallic reinforcements:

The design tensile strength of metallic reinforcement should be (As per BS8006:1995 page 34):

\[ T_D = T_U / f_m \]

For plain or galvanized steel reinforcements subject to axial tensile loads, the value of \( f_m \) should be 1.5.

6.3.3.2 Partial material factors for polymeric reinforcements:

The material factor for polymeric reinforcing is generally supplied by the manufacture as given in Fig 14, which is required to be verified if necessary by independent Agency.

The design tensile strength of polymeric reinforcement should be (As per BS8006:1995 page 34):

\[ T_D = T_{CR}/f_m \text{ or } T_{CS}/f_m \]

6.3.3.3 Partial material factor for soil:
This factor is applied to soil parameters such as \( c \) and \( \phi \) to account for uncertainty (As per BS8006:1995 page 34):

\[ C_d = C_k / f_{ms} \]

6.3.3.4 Partial load factors:

There are three types of partial load factors used. These are:
a) Partial load factors prescribed for soil self weight, \( f_k \);
b) Partial load factors prescribed for external dead loads, \( f_r \);
c) Partial load factors prescribed for external live loads, \( f_q \);

Load factors are applied as follows (As per BS8006:1995 page 34):

\[ F_d = f_f \times F_k \]

Load factors prescribed for external dead loads and live loads are normally the same for each reinforced soil application. However load factors prescribed for soil self weight will differ depending on the reinforced soil application. Value of the appropriate partial load factors are shown in BS 8006:1995 in Table 16, 17, and 18.
6.3.4: Soil/reinforcement interaction factors;

In reinforced soil walls and abutments there are two main interfaces where the soil and the reinforcement interact:
- Soil sliding across the surface of the reinforcement \( (f_{s}) \);
- Pull-out of the reinforcement from the resistant zone \( (f_{p}) \).

The example of application of various partial factors in calculation of long term design strength (LDTS) for polymeric reinforcement for particular material (like Miragrid geogrids) for various condition of design life and type of soil is indicated in Fig. 14.

### PROPERTIES OF MIRAGRID XT GEOGRIDS

<table>
<thead>
<tr>
<th>Property</th>
<th>Units</th>
<th>2XT</th>
<th>3XT</th>
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<tr>
<td>Partial factor-creep rupture</td>
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<td>Creep-limited strength</td>
<td>kN/m</td>
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<td>26</td>
<td>35</td>
<td>42</td>
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<td>At 50 years design life</td>
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<td>33</td>
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<td>Soil environmental, pH ≤ 10</td>
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<td>20</td>
<td>28</td>
<td>35</td>
<td>48</td>
<td>66</td>
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<td>in clay, silt or sand</td>
<td>kN/m</td>
<td>13</td>
<td>19</td>
<td>27</td>
<td>34</td>
<td>46</td>
<td>62</td>
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<td>in sandy gravel</td>
<td>kN/m</td>
<td>11</td>
<td>16</td>
<td>22</td>
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<td>in gravel</td>
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<td>17</td>
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<td>At 100 years design life</td>
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<td>27</td>
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<td>in clay, silt or sand</td>
<td>kN/m</td>
<td>12</td>
<td>17</td>
<td>26</td>
<td>31</td>
<td>43</td>
<td>58</td>
</tr>
<tr>
<td>in sandy gravel</td>
<td>kN/m</td>
<td>11</td>
<td>16</td>
<td>22</td>
<td>30</td>
<td>42</td>
<td>57</td>
</tr>
<tr>
<td>in gravel</td>
<td>kN/m</td>
<td>10</td>
<td>14</td>
<td>21</td>
<td>28</td>
<td>39</td>
<td>53</td>
</tr>
<tr>
<td>Partial factor- construction damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, silt or sand</td>
<td></td>
<td>1.25</td>
<td>1.15</td>
<td>1.13</td>
<td>1.10</td>
<td>1.10</td>
<td>1.05</td>
</tr>
<tr>
<td>Sandy gravel</td>
<td></td>
<td>1.35</td>
<td>1.25</td>
<td>1.17</td>
<td>1.13</td>
<td>1.13</td>
<td>1.13</td>
</tr>
<tr>
<td>gravel</td>
<td></td>
<td>1.60</td>
<td>1.50</td>
<td>1.40</td>
<td>1.25</td>
<td>1.25</td>
<td>1.22</td>
</tr>
<tr>
<td>Partial factor- construction damage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcement strain</td>
<td>%</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>Strain at maximum load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal roll width</td>
<td>m</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
</tr>
<tr>
<td>Normal roll length</td>
<td>m</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Estimated roll weight</td>
<td>kg</td>
<td>40</td>
<td>50</td>
<td>55</td>
<td>60</td>
<td>75</td>
<td>85</td>
</tr>
</tbody>
</table>
6.4 Fasteners and connections

Fasteners and connections are often necessary in reinforced soil structures, particularly where reinforcing elements are connected to some form of facing. Appropriate materials factors should be applied to the strength of the connection in the same way as for the reinforcing elements.

6.5 Design Information:

In order to develop the design of reinforced soil structure the following:

Information is required to be evaluated:

(A) Site investigation
(B) Environmental consideration
(C) Load combination
(D) Design record

(A) Site investigation:

In site investigation initial field study, ground investigation and its field study report should be considered in design and in some case (construction over soft soil) investigation during construction should be monitor

i) Initial field study
The availability and characteristics of the potential local fill materials should be accessed together with details of local drainage.

ii) Ground investigation

i) Area of investigation
- ground conditions
- behaviour of the foundation strata under the imposed loads
- information of settlement (total & differential)

ii) Ground water investigation
- Ground water conditions (pH and chemical content of ground water may affect the durability of reinforcing elements, fasteners & facing).
- Fluctuations in ground water regime may affect the overall structural behaviour.

iii) Field study report

Site investigation report should contain the relevant design parameters for the appropriate structure. The fill or ground material which is proposed to be used in structure should be tested for particle size distribution, short and long term strength parameters and consolidation parameters where applicable should be included.
(iv) Investigation during construction

Where the construction over soft soil undertaken the monitoring of settlement and pore water dissipation should be monitor. The results of this inspection should be compared to the findings of the ground investigation and the design assumptions, and the design checked against any variations.

(B) Environmental Consideration:

In design the environmental consideration should be consider which is as follows:

The effects of loads and pressure should be considered in design such as impact or seismic loads, loads due to water pressure including seepage pressure, buoyancy and lateral pressure and increased allowance for reinforcement deterioration

The chemical and biological effects of the material used should be considered in design. Material commonly used in reinforced soil are metallic reinforcement, polymeric reinforcement and polymeric reinforcement joints. During design the chemical (pH values, chemical contents in soil), biological (UV effect) and heat (temperature) should be considered.

The Post construction damage should be considered in design. The adjacent structure interaction effects should be considered in design such as if reinforced earth structure is adjacent to or part of any other structure then interaction

(C) Load combinations

The most adverse loads likely to be applied in the structure should be considered in design.

(D) Design record

Important design records should be maintained to enable review of the structure in the future.
7.0  Design of reinforced earth retaining walls

The aim of design is to achieve of an acceptable probability that designed structures will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use along with adequate durability and resistance to the effects of misuse and fire etc.

Design of reinforced earth structure investigate two main criteria to develop the dimensions and layout of the reinforced earth structure. The external stability of a reinforced soil wall is easily investigated since it behaves as a rigid gravity structure and conforms to the simple laws of statics. The analysis of internal stability is essentially one of designing reinforcement against tension failure and ensuring that it has a sufficient anchorage length into the stable soil.

Design methods are based on both limit equilibrium and limit state approaches and most common methods of design are described as under:

7.1 Design of walls and abutments

Vertical walls and associated abutment structures can be constructed using horizontal grid reinforcement as shown in Fig. 15

![Different structure with grid reinforcement](image)

Fig 15 : Different structure with grid reinforcement
7.1.1 Basis for design

In the reinforced earth wall two type of stability checked:

i) **External stability**: It consider the reinforced structure as whole and check the stability for sliding, overturning, bearing/tilt and slip as shown in Fig 16

![Fig 16: External failure Mechanism](image)

ii) **Internal stability**: It cover internal mechanism (tension and pull out failure) such as shear within the structure, arrangement and behavior of the reinforcement and backfill. It checks the stability for each reinforcement layers and stability of wedges within the reinforced fill

![Fig 17: Tension and Pull out failure](image)
The flow diagram of the design procedure is shown in Fig 18

1. Initial size of structure
2. External stability check
3. Select type of reinforcement
4. Internal stability
   - Anchored earth
   - Tie back wedge method
   - Coherent gravity method
5. Calculate tensile forces to be resisted by each layer of anchors
6. Calculate tensile forces to be resisted by each layer of reinforcements
7. Calculate pull-out capacity of anchors
8. Consider local stability check rupture and adherence
9. Calculate adherence capacity of the reinforcements
10. Check long term rupture
11. Check serviceability
12. Check internal forward sliding
13. For standard load cases/geometry internal design complete
14. Coherent gravity method
15. For non-standard load cases/geometry cases reinforcement
16. Design connection

Fig 18 : Design Procedure for reinforced soil walls

Generally two design methods are identified for internal stability
- i) Tie-back wedge method
- ii) Coherent Gravity method
(i) **Tie-back wedge method:**
In this method the analysis considers the stability of individual reinforcing element, resistance to horizontal sliding of the upper element of the structure and stability of wedges within the reinforced fill. A uniform frictional or cohesive-frictional fill is assumed and horizontal soil pressure are taken to be the active condition throughout the structure.

(ii) **Coherent Gravity method:**
In this method the assumption is that the reinforced mass is divided into two fundamental zones, divided by the line of maximum tension in the reinforcement. It also assumes that the state of stress within the reinforced mass varies. At the top and at the bottom of the structure the at-rest condition and active stress state condition was assumed to exist respectively. Cohesionless fill material was used within the structure. An apparent coefficient of adherence between reinforcing element and the fill based upon pull out was assumed.

### 7.1.2 Initial Dimension

Prior to considering external stability the overall geometry of the wall or abutment should be selected. Consideration of either the external or the internal stability may require the dimensions of the structure to be increased from the initial size. The initial dimensions of the structure should not be less than the minimum specified in Table 3 unless it can be satisfactorily demonstrated by previous experience that smaller values are adequate.

The geometrical size of the structure should be based upon a concept of mechanical height, \( H \), which is defined as the vertical distance from the toe of the structure to the point where a line at arc tan 0.3 to the vertical outcrops the upper ground line above the wall. Fig 19 give details of the initial sizing of the structure and referred to in Table 3. Walls with a trapezoidal cross section should only be considered where foundation are formed by excavation into rock or when good foundation exists.

![Initial sizing of structures](image-url)
Table 3: Dimensions of wall and abutments

<table>
<thead>
<tr>
<th>Structure type</th>
<th>Minimum reinforcement length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall with normal retaining function</td>
<td>0.7H (3m minimum)</td>
</tr>
<tr>
<td>Bridge abutments</td>
<td>The greater of 0.6H + 2m or 7m</td>
</tr>
<tr>
<td>Trapezoidal walls and abutments ¹)</td>
<td>0.7H for reinforcements in top half of structure,</td>
</tr>
<tr>
<td></td>
<td>0.4H for reinforcements in bottom half of structure,</td>
</tr>
<tr>
<td></td>
<td>or 3 m minimum</td>
</tr>
<tr>
<td>Stepped walls and abutments</td>
<td>0.7H in top half of structure, for longer strips at base</td>
</tr>
<tr>
<td>Walls subject to low thrust from retained fill such as</td>
<td>0.6H or 3 m minimum</td>
</tr>
<tr>
<td>negative back slope or embedded wall</td>
<td></td>
</tr>
<tr>
<td>For trapezoidal walls the vertical spacing of the</td>
<td></td>
</tr>
<tr>
<td>reinforcements should obey the following.</td>
<td></td>
</tr>
<tr>
<td>L/H &lt; 0.55 : Sv/H ≤ 0.125</td>
<td></td>
</tr>
<tr>
<td>0.55 ≤ L/H &lt; 0.65 : Sv/H ≤ 0.167</td>
<td></td>
</tr>
<tr>
<td>0.65 ≤ L/H &lt; 0.75 : Sv/H ≤ 0.222</td>
<td></td>
</tr>
<tr>
<td>where</td>
<td></td>
</tr>
<tr>
<td>Sv is the vertical spacing of reinforcements;</td>
<td></td>
</tr>
<tr>
<td>L is the length of reinforcement at any level;</td>
<td></td>
</tr>
<tr>
<td>H is the height of structure.</td>
<td></td>
</tr>
</tbody>
</table>

Embedment

The toe of the structure should be embedded below ground surface. Embedment is recommended to avoid load failure by punching in the vicinity of the facing and to avoid the phenomenon of local soil flow similar to piping within the structure. The amount of embedment depends on various factors which included:

- pressure imposed by the structure on its foundation.
- Frost depth (usually taken as 0.45 m).
- Risk of piping if a water head build up behind the facing in river and sea wall.
- Risk of scour at the toe of river training wall and sea walls.

Structures should have an embedment depth of at least the commonly adopted frost penetration depth of 0.45 m unless they are founded on a rock or structural base such as raft, mattress or old pavements. The minimum embedment should not be less than that value as given in Table 20 of BS 8006:1995, which is applicable to a structure slenderness ratio of not less than L/H = 0.7 and for good ground conditions. On sites where the foundation is weak or soft, greater embedment depth expressed in terms of the mechanical height of the wall provides a conservative value and is generally used for initial sizing. The minimum embedment depth expressed in terms of the factored bearing pressure at the base of the wall provides a more rigorous solution.

For structures subject to water action by river or sea, anti-scour precautions, rip-rap or gabion mattresses should be provided to ensure stability. In these cases an embedment depth greater than the minimum defined as given in Table 20 of BS 8006:1995 should be considered.
7.1.3 External Stability

The external stability of a reinforced soil wall is easily investigated since it behaves essentially as a rigid body and conforms to the simple laws of statics. External stability assessment should consider the effects of dead loads, other loads (live load, dynamic load etc.) and forces acting on the structure. The failure for sliding, overturning, tilting/bearing and slip should be checked by external stability as shown in Fig 20. Definition of soil properties and principal loads considered in calculation. Short and long term stability of soil needs to be considered to allow for the construction and in-service condition as well as in changes in pore water pressure. Passive earth pressure acting on the foot of the wall/structure below ground level may be ignored while considering various forces for stabilisation.

Consider the external stability of the surcharged, vertical wall as shown in Fig. 21 which shows the externally applied forces assuming a Rankine Distribution of lateral earth pressure and a uniform distribution of ground bearing pressure over a base length of \( L - 2e \). It has become general practise to adopt the Meyerhof pressure distribution for bearing pressure.
7.1.3.1 Sliding

The stability against forward sliding of the wall or structure should be considered at the interface between the sub-soil and reinforced fill. The resistance to movement should be based upon the sub-soil properties or reinforced properties, which ever is lesser in magnitude. Also weightage should be given to sliding on or between reinforcement layers, if any used at the base of the wall/structure.

The following contact condition at the base of the structure may be considered (As per BS 8006: 1995. page 47 & 48):

(A) Long term stability
   (i) Soil – to Soil contact
       \[ f_s R_h \leq \frac{R_v \tan \phi'_p}{f_{ms}} + \frac{c' L}{f_{ms}} \]
   (ii) Soil - to - reinforcement contact
       \[ f_s R_h \leq R_i \frac{\alpha \tan \phi'p}{f_{ms}} + \frac{abc' c' L}{f_{ms}} \]

(B) Short term stability
   (i) Soil – to – soil contact
       \[ f_s R_h \leq \frac{c_u L}{f_{ms}} \]
   (ii) Soil – to – reinforcement contact
       \[ f_s R_h \leq \frac{abc' c_u L}{f_{ms}} \]
Where a layer of reinforcement coincides with the base of the wall the value of $f_s$ for soil/reinforcement sliding should be used. Where reinforcement does not coincide with the base of the wall the value of $f_s$ for soil-to-soil sliding should be used.

### 7.1.3.2 Overturning:

The coefficient of active earth pressure for backfill is as given below:

$$K_{ab} = \frac{(1 - \sin \phi_b)}{(1 + \sin \phi_b)}$$

In reinforced earth structure restoring moment about the toe is always greater than the overturning moment about the toe.

The moment of all factored restoring loads $\geq$ Moment of all factored disturbing loads about the toe of the wall. Or The restoring moment is the product of overturning moment and factor of safety against overturning.

### 7.1.3.3 Tilting /bearing failure

Generally for designing the reinforced earth structure, bearing pressure $q_r$ based upon Meyerhoff distribution is assumed as shown in Fig 22. A typical bearing pressure imposed by reinforced structure on the foundation strata is also shown in Fig 22 (As per BS 8006: 1995, page 47):

![Diagram showing pressure distribution and idealized bearing pressure](image)

(a) Pressure imposed at base (b) Idealized bearing pressure

Fig 22: Pressure distribution along base of wall

Therefore

$$q_r = \frac{R_v}{L} - 2e$$

The imposed bearing pressure $q_r$ should be compared with the ultimate bearing capacity of the foundation soil as follows:

$$q_r \leq q_{ult}/f_{ms} + \gamma D_m$$
7.1.4 Internal stability

Stability within a reinforced structure is achieved by the reinforcing elements carrying tensile forces and then transferring to the soil by friction, friction and adhesion, or friction and bearing. In addition forces can be transferred the soil through fill trapped by the elements of the grid. The fill is than able to support the associated shear and compressive forces. In the case of anchored earth such as soil nailing, stability within a structure is achieved by the anchor elements carrying tensile forces and transferring these by friction along the anchor shaft or anchor loop and bearing of the anchor to the surrounding fill.

The internal stability is concerned with the integrity of the reinforced volume as a whole and also whether the structure has the potential to fail by the rupture or loss of bond of reinforcements. The analysis considers the local stability of individual layer of reinforcing elements and stability of several wedges originating at different height at angle $45 - \phi/2$, where the tensile force required to be resisted is maximum with in the reinforced fill. The arrangement and layout of reinforcing elements should be chosen to provide stability and to suit the size, shape and detail of facing.

The simplest layout is a uniform distribution of identical reinforcing element throughout the length and height of the structure. A more economical layout may be achieved using reinforcement of different properties or by dividing the structure in to different zones with different spacing of height.

7.1.4.1 Local stability of a layer of reinforcing element:
In the local stability of reinforcing layer of element, the vertical spacing between the layer is determined in the zone. Then from the vertical spacing, the local stability is checked for rupture and adherence. If rupture and adherence stability not satisfied the condition then reduces the vertical spacing and again repeat the above process. Loads are distributed throughout the reinforced soil block in accordance with the Meyreof distribution as shown in Fig 23. The Maximum ultimate limit state tensile force $T_j$ to be resisted by jth layer of elements at a depth of $h_j$ below the top of the structure may be obtained from the summation of the appropriate forces as follows see Fig. 23 (As per BS 8006:1995 page 51):

![Fig 23: Stability – effect to be considered](image-url)
For frictional fill
\[ T_j = T_{pj} + T_{sj} + T_{fj} \]
and for cohesive friction fill
\[ T_j = T_{pj} + T_{sj} + T_{fj} - T_{cj} \]
The above forces \( T_{pj}, T_{sj}, T_{fj} \) and \( T_{cj} \) are derived as follows:

### 7.1.4.2 Vertical loading due to self weight of fill plus any surcharge and bending moment caused by external loading acting on the wall as shown in Fig 24 (As per BS 8006:1995 page 52):

\[ T_{pj} = K_{aw} \sigma_{vj} S_{vj} \]
Therefore
\[ \sigma_{vj} = R_{vj}/L_j - 2e_j \]

For cohesive frictional fill (As per BS 8006:1995 page 52):
\[ T_{pj} \geq 0.5 \gamma_{wa} S_{vj} (h_j + w_s f_{js}/\gamma_w) \]

For a uniform surcharge the expression for \( T_{pj} \) becomes (As per BS 8006:1995 page 52):
\[ T_{pj} = \frac{K_{aw} (f_{fs} \gamma_w h_j + f_q w_s) S_{vj}}{1 - \frac{K_{ab} (f_{fs} \gamma_b h_j + 3 f_q w_s) (h_j/L)^2}{3(f_{fs} \gamma_w h_j + f_q w_s)}} \]

### 7.1.4.3 Vertical strip loading \( S_L \) applied to a strip contact area of width \( b \) on top of the wall, as shown in Fig 25. For the purpose of deriving the magnitude only of the tensile force \( T_{sj} \) dispersal of the vertical load \( S_L \) from the contact area on top of the wall, may be taken at a slope of 2 vertically to 1 horizontally as shown in Fig 25 (As per BS 8006:1995 page 52):
\[ T_{sj} = K_{aw} S_{vj} (f_r S_L)/D_j \]
Where \( D_j = (h_j + b) \) if \( h_j \leq (2d - b) \)
\[ = (h_j + b)/2 + d \] if \( h_j > (2d - b) \)

The tensile force obtained from the equation above should be taken as not less than that derived from the bending moment caused by the vertical loading \( S_L \) alone acting on the wall treated as a rigid body.

Fig 25: Dispersal of vertical strip load through reinforced fill

7.1.4.4 Horizontal shear \( F_L \) applied to the strip contact area of width \( b \) on top of the wall, see Fig 26 For the purpose of deriving the magnitude only of the tensile force \( T_{fj} \) dispersal of the load \( F_L \) from the contact area of the top of the wall may be taken as shown in Fig 26 (As per BS 8006:1995 page 52)

\[ T_{fj} = 2 S_{vj} f_f F_L Q (1-h_jQ) \]

Where \( Q = \tan (45^\circ - \phi_p'/2) / (d + b/2) \)

The tensile force obtained from the equation above should be taken as not less than that derived from the bending moment caused by the horizontal loading \( F_L \) alone acting on the wall treated as a rigid body.

Fig 26: Dispersal of horizontal shear through reinforced fill
7.1.4.5 Tensile force due to cohesion in reinforced fill (As per BS 8006:1995 page 52):

\[ T_{cj} = 2S_{cj} c'/f_{ms} \sqrt{K_a} \]

7.1.5 Local stability check

The resistance of the jth reinforcing element should be checked against rupture and adherence failure whilst carrying the factored loads.

7.1.5.1 Local stability checked against rupture:

The tensile strength of the jth layer of reinforcing elements needed to satisfy local stability considerations is (As per BS 8006:1995 page 52):

\[ T_D / f_n \geq T_j \]

7.1.5.2 Local stability checked against adherence:

The perimeter \( P_j \) of the jth layer of reinforcing elements needed to satisfy local stability considerations is (As per BS 8006:1995 page 52):

\[
P_j \geq \frac{T_j}{\mu L_{cj} (f_s \gamma_w h_j + f_w w_s) + a_{bc}' c' L_{cj}}
\]

\[
+ \frac{f_p f_n}{f_{ms} f_p f_n}
\]

For convenience it may be assumed

\[ \mu = \alpha' \tan \phi' / f_{ms} \]

where

\[ \alpha' = \text{Interaction coefficient relating soil/reinforcement bond angle with } \tan \phi'. \] The value of \( \alpha' \) coefficient of interaction which is provide by the material manufacturer or can be obtained by Lab. Test. Generally the value of \( \alpha' \) for geogrids are shown in Table 4:

Table 4: Values of interaction coefficient (\( \alpha' \))

<table>
<thead>
<tr>
<th>0.9 to 1</th>
<th>for crushed rock and gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85 to 0.95</td>
<td>for sands</td>
</tr>
<tr>
<td>0.8 to 0.85</td>
<td>for pulverized fuel ash</td>
</tr>
<tr>
<td>0.6 to 0.7</td>
<td>for clay</td>
</tr>
</tbody>
</table>

7.1.6 Wedge stability

As per flow chat of design show in Fig 18, the reinforcement designed as per local stability analysis is required to be checked for wedge stability also. The reinforcement structure will assume to fail internally in the form of wedge. It is not known at which level the wedge is originate. Therefore the wedge originate from different level to be checked. Checked for stability considering all the forces acting on it. Wedges are assumed to behave as rigid bodies and may be any size and shape. Stability of any wedge is maintained when friction forces acting on the potential failure plane in connection with the tensile resistance/ bond of the group of reinforcing elements or embedded in the fill beyond the plane are able to resist the applied loads tending to cause movement, as shown in Fig 27. The following loads, factored in accordance with combinations in table 4 and forces are considered in the analysis:
A selection of potential failure planes investigated for each of the typical points a,b,c etc., shown in Fig 28. The forces acting on each wedge should be revolved into two mutually perpendicular directions. Since the forces are assumed to be in equilibrium the two equations may be solved simultaneously to yield the value of the gross tensile force $T$ to be resisted by reinforcing elements or anchors. For each of the typical points the maximum value of $T$ should be established by analysing the forces acting on a number of different wedges. The maximum value of $T$ and the corresponding value of $\beta$ are used to calculate the frictional/tensile capacity of the group of elements anchoring the wedge as shown in Fig.29.
For the case of a wall with a level top containing frictional fill and which supports uniform surcharge only the inclination of the potential failure plane may be take as \( \beta = (45 - \phi_p / 2) \).

However in the more complex general case it is not possible to give any guidance on either the angle of the potential failure plane which produces the maximum value of \( T \) or on the number of points which should be checked. These should be determined for each structure, it may be assumed that no potential failure plane will pass through the strip contact area representing a bridge bank seat. When the facing consists of a structural element formed in one piece the shear resistance offered by the rupture of the facing may be considered.

**7.1.7 Wedge stability check by comparison**

The value of the force to be resisted with active surcharge

\[
T = \frac{h \tan \beta (f_{n_{w}} \gamma_{w} h_{j} + 2 f_{q} w_{s})}{2 \tan (\phi_{w} + \beta)}
\]

The resistance provided by an individual layer of reinforcing elements should be taken to be the lesser of either:

a) the frictional resistance of that part of the layer embedded in the fill beyond the potential failure plane or, in the case of anchored earth, the pull out resistance of the part of the anchors embedded in the fill beyond the potential failure plane (which should be neglected when the distance between the potential failure plane and the start of the anchorage is less than 1m) or

b) the tensile resistance of the layer of elements.

For reinforced soil the total resistance of the layers of elements anchoring the wedge is satisfied by

\[
\sum_{j=1}^{m} \left[ \frac{T_{Dj}}{f_{n_{j}}} \right] \geq T \quad \text{or}
\]

\[
\sum_{j=1}^{m} \left[ \frac{P_{j} L_{eqj} \left\{ \mu f_{k_{s}} \gamma_{h_{j}} + \mu f_{t} w_{s} + \frac{a_{pc} \cdot C_{r}}{f_{ms}} \right\}}{f_{p} f_{n}} \right] \geq T
\]

The lesser value for each layer should be used in the summation.

The above method is explain through a design example given in **Annexure –A**.
8.0 Design of Reinforced Steep Slopes

For a uniform fill soil there is a limiting slope angle $\beta_{\text{lim}}$ to which an unreinforced slope may be safely built.

For the case of a non-cohesive and dry material, the limit angle of the slope equals the friction angle of the soil:

$$\beta_{\text{lim}} = \phi$$

A slope with a greater angle than the limiting slope angle is a steep slope; to build an embankment with a steep slope it is necessary to provide some additional forces to maintain equilibrium. The easiest method is to place horizontally some reinforcing layers in the slope so that the reinforcements can resist the horizontal forces, thus increasing the allowable shear stresses. The forces which must be applied to the soil to maintain equilibrium can be added up in a gross force that works in a horizontal direction, that is the direction of the reinforcement.

![Steep Slope Diagram](image)

**Fig. 30**: Steep Slope

8.1 Design Criteria

The design problem for a reinforced slope can be set as following:-

Once the geometry of the slope is defined, the surcharge load fixed, the geotechnical characteristics of the soil known and the design resistance ‘P’ of the grids set, we must find the number, vertical position and the length of the reinforcing layers required to provide the equilibrium for every possible failure mechanism.

8.2 Design Chart Procedure

The design charts that is known as Jewell Chart 1991 developed to determine the soil pressure coefficient and required length of reinforcement. These charts is for different values of $r_u$ (i.e 0,0,25 & .50) is placed at Annexure C. In each design chart there are three graph for each $r_u$, these three value are $K_{\text{req}}$, $(L/H)_{ovrl}$ and $(L/H)_{ds}$ respectively considered against the slope angle $\beta$ and internal friction angle $\phi$. 
The steps for the design of a reinforced slope, referring to the charts in Annexure - C are:

1) Define the geometrical configuration of the slope (soil characteristic (c’ φ’ and β), height of embankment and slope angle β) and eventually the uniformly distributed surcharge loading on the top of the slope (w_s) as shown in Fig 30.

2) Calculate the apparent height (H^), where

\[ H^ = H + \frac{w_s}{\gamma} \]

3) Set the safety factors FSgrid

Creep limited strength = \( \frac{\text{Characteristic ultimate limited strength}}{\text{partial factor for creep rupture}} \)

Long term design strength = \( \frac{\text{Creep limited strength}}{\text{partial factor constr. X partial factor environment}} \)

Or

\[ P_{des} = P_c / f_m f_d f_e LF \]

Or

In some cases the design resistance is determined as a fraction of the allowable resistance by means of safety factor for design. The safety factor range between 1.05 and 1.5.

FS_grid shall be obtained by multiplying several partial factors of safety.

\[ FS_{grid} = FS_{creep} \cdot FS_{construction} \cdot FS_{chemical} \cdot FS_{biological} \cdot FS_{junction} \]

Partial factors of safety for some geogrids as summarized in Table 5 and 6.

Table 5 : FSconstruction for different type of soil

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>FS construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt and clay</td>
<td>1.0</td>
</tr>
<tr>
<td>Fine and medium sand</td>
<td>1.0</td>
</tr>
<tr>
<td>Coarse sand and fine gravel</td>
<td>1.0</td>
</tr>
<tr>
<td>Crushed gravel</td>
<td>1.10</td>
</tr>
<tr>
<td>Ballast, sharp stones</td>
<td>1.10</td>
</tr>
<tr>
<td>Pulverized fuels ashes</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Factors of safety for some geogrids in Table 6

Table 6

<table>
<thead>
<tr>
<th>FS</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>FSjunction</td>
<td>1.00</td>
</tr>
<tr>
<td>FSchemical</td>
<td>1.00</td>
</tr>
<tr>
<td>FSbiological</td>
<td>1.00</td>
</tr>
<tr>
<td>FScreeep</td>
<td>1.30</td>
</tr>
</tbody>
</table>
The ratio of $T_{ult}$ by means of $F_s$ creep in the Long Term Design Strength of the geogrid. LTDS is a function of the creep phenomena in the geogrids which have an increasing importance in relation to the design life of the project. Table- 7 provides the values of LTDS for the some geogrids at different temperatures:

Table 7 : LTDS in kN/m for some geogrids at different temperatures

<table>
<thead>
<tr>
<th></th>
<th>10°</th>
<th>20°</th>
<th>30°</th>
<th>40°</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT201</td>
<td>19.1</td>
<td>16.4</td>
<td>13.2</td>
<td>11.9</td>
</tr>
<tr>
<td>TT301</td>
<td>27.6</td>
<td>23.5</td>
<td>18.9</td>
<td>17.0</td>
</tr>
<tr>
<td>TT401</td>
<td>34.0</td>
<td>30.6</td>
<td>24.5</td>
<td>22.0</td>
</tr>
<tr>
<td>TT601</td>
<td>42.0</td>
<td>38.1</td>
<td>30.6</td>
<td>27.5</td>
</tr>
<tr>
<td>TT 701</td>
<td>46.7</td>
<td>42.0</td>
<td>33.7</td>
<td>30.2</td>
</tr>
</tbody>
</table>

4) If pore water pressure is not given then find the maximum pore water pressure ‘$r_u$‘ is as given below :
   \[ r_u = \max\left(\frac{u(z)}{z. \gamma}\right) \]

5) Using the value of the slope angle $\beta$ and the angle of friction $\phi$ of the soil, calculate the coefficient of earth pressure $K$ and the ratios of reinforcement length to embankment height ($L/H_{ovrl}$ and ($L/H_{ds}$ using one of the set of chart as in Annexure -C.(Fig. A1 to A9) The chart to be used shall be selected on the base of the anticipated value of the pore water pressure coefficient ‘$r_u$’. Where ($L/H_{ovrl}$ is the ratio of length and height overall and ($L/H_{ds}$ is the ratio of length and height of design strength.

After finding the value of ($L/H_{ovrl}$ and ($L/H_{ds}$ the two condition is occur which is as under:

a) if ($L/H_{ovrl} > (L/H_{ds}$
   the reinforcement length shall be constant and equal to
   \[ L = (L/H_{ovrl} * H^\gamma \]

b) If ($L/H_{ovrl} < (L/H_{ds}$
   The reinforcement length can be
   (i) constant and equal to
   \[ L = (L/H_{ds} * H^\gamma \]
   ( ii) with a length varying uniformly from:
   - length at the base
   \[ L = (L/H_{ds} * H^\gamma \]
   to:
   - length at the crest
   \[ L = (L/H_{ovrl} * H^\gamma \]

6) Calculate the spacing constant ‘Q’ for each grid, assume a compacted fill layer thickness $v$ in mm (minimum vertical spacing) and is defined as below:
   \[ Q = P/K.\gamma.v \]
7) Calculate the zones for reinforcement layers spaced equally at $v$, $2v$, $3v$ as shown in Table 8.

**Table 8: Reinforcement Spacing**

<table>
<thead>
<tr>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing $S_v$ (m)</td>
<td>Depth $Z_i$ (m)</td>
<td>Thickness $S_i$ (m)</td>
</tr>
<tr>
<td>$S_{v1} = v$</td>
<td>$Q - Q/2$</td>
<td>$S_1 = H^\gamma - Q/2$</td>
</tr>
<tr>
<td>$S_{v2} = 2v$</td>
<td>$Q/2 - Q/3$</td>
<td>$S_2 = Q/2 - Q/3$</td>
</tr>
<tr>
<td>.</td>
<td>.</td>
<td>.</td>
</tr>
<tr>
<td>$S_{vn} = nv$</td>
<td>$Q/n - \frac{w_s}{\gamma}$</td>
<td>$S_n = Q/n - \frac{w_s}{\gamma}$</td>
</tr>
</tbody>
</table>

Note:

Col I: Assume the vertical spacing, the minimum value consider as $0.2m$.
Col. II: Depth of spacing range i.e. $Q$ to $Q/2$, $Q/2$ to $Q/3$, $Q/3$ to $Q/4$, ...
Col. III: Total thickness of the zone, i.e. $H^\gamma - Q/2, Q/2 - Q/3$, ...

If $H^\gamma < Q$ the minimum spacing $v$ at the base of the slope will have to be reduced or a more resistant geogrid has to be selected.

(8) Calculate the number of required reinforcement layers:

(a) The first layer is placed on the foundation at the base of the slope, the other required layers are calculated starting from the base. Referring to Table 8, the steps of the procedure are:

(i) Divide the thickness of every zone (see Table-9) by the spacing of the reinforcement layers in that zone to calculate the number of grids in a zone.

$$N_i = \frac{S_i}{S_v} \text{ whole number}$$

(ii) Calculate the remaining thickness of the zone

$$R_i = S_i - S_v N_i$$

(iii) Add $R_i$ to the thickness of the next zone:

$$S_{Si+1} = S_{i+1} + R_i$$

with

$$R_0 = 0$$

(iv) Repeat the calculation for all the zones.

**Table 9: Calculation of the required layers**

<table>
<thead>
<tr>
<th>IV</th>
<th>V</th>
<th>VI</th>
<th>VII</th>
</tr>
</thead>
<tbody>
<tr>
<td>Si /$S_v$</td>
<td>$N_i$</td>
<td>$R_i$</td>
<td>$S_{Si+1}$</td>
</tr>
<tr>
<td>$R_0 = 0.0$</td>
<td>$S_{S1} = S_1 + R_0$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_1 / S_v$</td>
<td>$N_1 = \text{INT} \left( \frac{S_1}{S_v} \right)$</td>
<td>$R_1 = S_1 - S_{v1} N_1$</td>
<td>$S_{S1} = S_2 + R_1$</td>
</tr>
<tr>
<td>$S_2 / S_v$</td>
<td>$N_2 = \text{INT} \left( \frac{S_2}{S_v} \right)$</td>
<td>$R_2 = S_2 - S_{v2} N_2$</td>
<td>$S_{S2} = S_3 + R_2$</td>
</tr>
<tr>
<td>.</td>
<td>.</td>
<td>.</td>
<td>.</td>
</tr>
<tr>
<td>$S_{Sn} / S_v$</td>
<td>$N_n = \text{INT} \left( \frac{S_n}{S_v} \right)$</td>
<td>$R_n = S_n - S_{vn} N_n$</td>
<td>.</td>
</tr>
</tbody>
</table>

$$N_{tot} = 1 + N_1 + N_2 + N_3 + \ldots + N_n$$

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Note:
Col. IV : Finding the number of layer is the zone i.e divide the Col. III by Col. I
Col. V : Rounded down the number of layer i.e rounded down Col IV
Col. VI : Finding the remaining thickness i.e Reminder = Col III – Col V x Col I
Col. VII : Finding the thickness of next zone i.e current thickness (Col III) + previous reminder (Col VI).

(v) If the top layer of reinforcement is more than 0.6 m below the slope crest it would be prudent to add an additional layer near to the crest.

(9) Calculate the gross horizontal force required for equilibrium:
\[ HT = \frac{1}{2} K \cdot \gamma \cdot H^2 \]

(10) Verify that the average required tensile force for every layer is less than the safe design strength of the grid:
\[ \frac{HT}{N_{tot}} \leq P \]
If this condition is not verified, increase the number of geogrid layers or repeat the procedure changing the minimum spacing.

(11) When using the wrap-around techniques, calculate the wrapping length \( L_r \) for every layer:
\[ \bar{\zeta}_i = z_i + \frac{w_s}{\gamma} \]
\[ L_{ri} = \frac{F_{S_{\text{wrap}}} \cdot K \cdot (\bar{\zeta}_i + S_{vi}/2) \cdot S_{vi}}{\bar{\zeta}_i \cdot f_{ds} \cdot \tan \phi'} \]
\[ L_r = \max (L_{ri}) \text{ optinally} \]
The factor of safety \( F_{S_{\text{wrap}}} \) can usually be assumed to be in the range 1.20 to 1.40.

(11) Draw the final layout of the reinforced slope.

The above method is explain through a design example given in Annexure –B.
8.3 Application of Reinforced Earth Structure in Railways:

In Jammu-Udhampur rail link (BG) project of Northern Railways, having a length of the project is 53.2 Km, was constructed on unstable formation and undulatory hill terrain. At km. 54/043 near bridge no. 162, about 80m length is passing across the slope of hill. At the foothill, river Tawi and canal is flowing. The height of bank is about 35 m. Due to land constraint it was not possible to construct the bank with 2.5 : 1 slope.

At this location, 35 m high reinforced earth embankment with slope of 1.5 : 1 over 4 to 7 m high retaining wall at base was adopted. The picture is shown in Fig 31.

(a) Aerial view of topography of Jammu-Udhampur site

(b) View of laying of fill material over geogrid
(c) View of Anchoring of grid to facia

(d) View of Laying of geogrid
(e) View of reinforced earth embankment

Fig. : 31
Fig. 32. Laying procedure of geogrids at a typical site

(a) View of laying geogrid in civil engg. project

(b) View of fill material over geogrid in civil engg. project
9.0 Test Properties For Design

As such a typical list of important properties of geosynthetics required for reinforcement function may thus include :-

1. Basic Physical Properties  
- Constituent material and method of manufacture  
- Mass per unit area  
- Thickness  
- Roll width, roll length

2. Mechanical Properties  
- Tensile strength  
- Tensile modulus  
- Seam strength  
- Interface friction  
- Fatigue resistance  
- Creep resistance

3. Hydraulic Properties  
- Compressibility  
- Opening size  
- Permittivity  
- Transmissivity

4. Constructability/survivability Properties  
- Strength and stiffness  
- Tear resistance  
- Puncture resistance  
- Penetration resistance  
- Burst resistance  
- Cutting resistance  
- Inflammability  
- Absorption

5. Durability (Longevity)  
- Abrasion resistance  
- Ultra-violet stability  
- Temperature stability  
- Chemical stability  
- Biological stability  
- Wetting & drying stability

All these may not be important for every application. For design purpose ultimate tensile strength, partial factor of safety for environment and creep reduction factor are required.
10.0 Conclusion

The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance. Design with geosynthetics vary by the design theory, guidelines and assumptions used. Unfortunately there is no standard set of design guidelines for all designers to use. A single product or system cannot produce universal solution. The difference associated with the various design methods produce differences in the design of these structure in terms of embedded length, required strength and amount of reinforcement required to provide acceptable factors-of-safety against internal and external potential modes of failure of the system.

Reinforcement of the earth has been mostly to reduce the total and differential settlement and has improve the serviceability limit of structures founded on the soft or compressible earth. The concept has facilitated construction of the high to very high retaining walls and slopes. The reinforcement of slopes can be achieved in a variety of ways such as nailing, root piles, dowells, anchors or even as a combination of reinforcement and drainage.

Construction of embankment with geosynthetics reduces land acquisition in case of soft and hilly terrain and metropolitans cities areas which reduces the cost of land and large amount of earth work. Due to portable size of geosynthetics which reduces the transportation cost. It increases the reliability and life of structure and reduces the maintainability of the structure. In long term it is a cost benefit material which lead to the economy and support Indian industry.

The concept of reinforcing earth has also attracted the attention of the academic world, for although the concept is easily grasped the theoretical aspects involved are numerous. As a result, much research and development work has been undertaken in universities and laboratories and earth reinforcing is now recognized as a separate subject in its own right in geotechnical field.
11.0 References

(2) Engineering with Geosynthetics – G.V.S.Venkatappa Rao, G.V.S. Suryanarayan Raju.
(3) Mechanics of Reinforced soil – Andrzej Sawicki.
(5) Earth Reinforcement and Soil Structures – Colin JFP Jones.
### Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_D$</td>
<td>Design tensile strength of the reinforcement</td>
</tr>
<tr>
<td>$T_U$</td>
<td>Ultimate tensile strength of the reinforcement (its base strength)</td>
</tr>
<tr>
<td>$T_{CR}$</td>
<td>Extrapolated creep rupture strength at the end of design life.</td>
</tr>
<tr>
<td>$T_{CS}$</td>
<td>Extrapolated tensile load based on creep strain at the end of design life.</td>
</tr>
<tr>
<td>$f_m$</td>
<td>Partial material factor for the reinforcement</td>
</tr>
<tr>
<td>$C_d$</td>
<td>Soil parameter design value</td>
</tr>
<tr>
<td>$C_k$</td>
<td>Characteristic value of the soil parameter which is the worst credible value</td>
</tr>
<tr>
<td>$f_{ms}$</td>
<td>Partial factor for the soil parameter or Partial materials factor applied to $\tan \phi_p$, $c'$ and $c_u$. or Partial material factor applied to qult</td>
</tr>
<tr>
<td>$F_d$</td>
<td>Design load</td>
</tr>
<tr>
<td>$F_k$</td>
<td>Unfactored characteristic disturbing load which is the worst credible value</td>
</tr>
<tr>
<td>$f_r$</td>
<td>Partial load factor for external concentrated dead loads or partial load factor applied to surcharge dead loads</td>
</tr>
<tr>
<td>$f_q$</td>
<td>Partial load factor applied to external live load.</td>
</tr>
<tr>
<td>$f_p$</td>
<td>Partial load factor for pull out resistance</td>
</tr>
<tr>
<td>$f_s$</td>
<td>Partial load factor for sliding resistance or partial factor against base sliding</td>
</tr>
<tr>
<td>$f_n$</td>
<td>Partial factor applied to economic ramifications of failure</td>
</tr>
<tr>
<td>$R_h$</td>
<td>Horizontal factored disturbing force</td>
</tr>
<tr>
<td>$R_v$</td>
<td>Vertical factored resultant force</td>
</tr>
<tr>
<td>$\phi'_p$</td>
<td>Peak angle of shearing resistance under effective stress condition</td>
</tr>
<tr>
<td>$c'$</td>
<td>Cohesion of the soil under effective stress condition</td>
</tr>
<tr>
<td>$d$</td>
<td>Maximum vertical deflection of unsupported reinforcement.</td>
</tr>
<tr>
<td>$h$</td>
<td>Average height of full above reinforcement.</td>
</tr>
<tr>
<td>$c_u$</td>
<td>Undrained shear strength of the soil</td>
</tr>
<tr>
<td>$L$</td>
<td>Effective base width for sliding. Or reinforcement length at the base of the wall.</td>
</tr>
<tr>
<td>$\alpha'$</td>
<td>Interaction coefficient relating soil/reinforcement bond angle with $\tan \phi'_p$ (from lab. Test or supplied by manufacturer)</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Angle of inclination of backfill thrust on reinforced soil block.</td>
</tr>
<tr>
<td>$\beta_s$</td>
<td>Inclination of slope.</td>
</tr>
<tr>
<td>$a_{bc'}$</td>
<td>Adhesion coefficient relating soil cohesion to soil/reinforcement bond, (from lab. Test or supplied by manufacturer)</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>Angle of internal friction for the back fill</td>
</tr>
<tr>
<td>$q_r$</td>
<td>Bearing pressure acting on the base of the wall</td>
</tr>
<tr>
<td>$e$</td>
<td>Eccentricity of resultant load $R_v$ about the centre of the base of width $L$. The value for different type of structure is calculated by laws of statics.</td>
</tr>
<tr>
<td>$B$</td>
<td>Width of element of reinforcement</td>
</tr>
<tr>
<td>$q_{alt}$</td>
<td>Length of Meyerhof pressure distribution</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Foundation soil density or unit weight of soil</td>
</tr>
<tr>
<td>$D_m$</td>
<td>Wall embedment depth</td>
</tr>
</tbody>
</table>
| $T$    | Total tensile force to be resisted by the layers of reinforcement which
anchor a wedge of reinforced soil, per metre ‘run’.

- \( T_j \): Maximum tensile force at a depth \( h_j \) below the top of the structure.
- \( T_{pj} \): Tensile force per meters due to weight of backfill.
- \( T_{sj} \): Tensile force due to vertical stiff loads.
- \( T_{gij} \): Tensile force due to horizontal shear applied a strip contact area.
- \( T_{cj} \): Tensile force due to cohesion in reinforce fill.
- \( K_{aw} \): Coefficient of earth pressure within the reinforced volume.
- \( K_a \): Coefficient of active earth pressure.
- \( \sigma_{vj} \): Factored vertical stress acting on the jth level of reinforcement according to the Meyerhof distribution (as shown in Fig. 22).
- \( S_{vj} \): Vertical spacing of reinforcements at the jth level in the wall.
- \( R_{vj} \): Resultant factored vertical load acting on the jth layer of reinforcements.
- \( L_j \): Length of the reinforcement at the jth level in the wall.
- \( e_j \): Eccentricity of resultant vertical load at the jth level in the wall.
- \( \gamma_{wa} \): Unit weight of water.
- \( h_j \): Depth of the elements below the top of the structure.
- \( f_{fs} \): Partial load factor applied to surcharge dead load or applied to soil self weight.
- \( w_s \): Surcharge due to dead load.
- \( \gamma_w \): Unit weight of the soil.
- \( P_j \): Total horizontal width of the top and bottom faces of the reinforcing element at the jth layer per meter ‘run’.
- \( \mu \): Coefficient of friction between the fill and reinforcing elements.
- \( L_{ej} \): Length of the reinforcement in the resistant zone outside failure wedge, at the jth layer of reinforcements, see Fig 3.
- \( T_{Dj} \): Design strength of the reinforcement at jth level in wall.
- \( L_{ej} \): Length of the reinforcement in the resistant zone outside potential failure wedge, of reinforcements.
- \( S_L \): Vertical loading, applied to a strip contact area of width ‘b’ on top of a structure, per metre ‘run’.
- \( F_L \): Horizontal shear.
- \( P_{des} \): Long term design strength.
- \( P_c \): Long term design strength from creep testing or creep limited strength.
- \( f_d \): Partial factor for construction or damage or site installation.
- \( f_e \): Partial factor for environment effect (i.e. chemical, biological and junction effect).
- \( LF \): Load factor as per design method generally it is taken as 1.
- \( K \): Force coefficient or coefficient of earth pressure.
- \( P \): Safe design strength.
- \( v \): Vertical spacing by assumption.
- \( Q \): Spacing constant between the grid.
- \( u(z) \): Pore water pressure at the depth \( z \) under the crest of the slope.
- \( H^\wedge \): Apparent height.
- \( N_{tot} \): Total number of geogrids.
- \( HT \): Horizontal Tensile force.
- \( r_u \): Maximum pore water pressure.
**ANNEXURE - A**

**Design example using the Tie-back wedge method**

Design a suitable layout for the 6 m high vertical soil wall shown in Fig 30 using Tensar 80RE and 55RE reinforcement, Creep limited strength for the two grid types for a design life of 120 years at a temperature of 10°C is 38.5 kN/m. The thickness of a single compacted layer of wall fill can be taken as 200 mm. The allowable range of ground bearing pressure under the wall is 0 to 200kN/m² for 80RE and 28.5 kN/m for 55RE. The coefficient of interaction (α) between grids and fill is 0.85.

\[ W_s = 10 \text{kN/m}^2 \]

![Wall fill (sand/Gravel)](\gamma_w = 19 \text{kN/m}^3 \quad \phi'_w = 35^\circ \quad c'_w = 0)

Back fill
\[ \gamma_b = 18 \text{kN/m}^3 \quad \phi'_b = 30^\circ \quad c'_b = 0 \]

![Fig 1: Layout of design](H = 6 \text{m})

Data given according to the problem:
Consider the initial dimension value from Table 3
Reinforcement length at base of the wall (L) = 0.7H
\[ = 0.7 \times 6 = 4.2 \text{m} \]
Unit weight of reinforced fill (\( \gamma_w \)) = 19 kN/m³
Unit weight of unreinforced fill (\( \gamma_b \)) = 18 kN/m³
Height of the reinforced soil block (H) = 6m.
Angle of internal friction for the wall fill (\( \phi'_w \)) = 35°
Angle of internal friction for the back fill (\( \phi'_b \)) = 30°
Cohesion of the wall fill (\( c'_w \)) = 0
Cohesion of the back fill (\( c'_b \)) = 0
Uniformly distributed surcharge (\( w_s \)) = 10 kN/m²
Coefficient of interaction between grid and fill (\( \alpha \)) = 0.85 (from Table 4)
Creep limit strength for 55 RE grid (\( T_{CR} \)) = 28.5 kN/m (from manufacturer)
Creep limit strength for 80 RE grid (\( T_{CR} \)) = 38.5 kN/m (from manufacturer)
Allowable range of bearing pressure under wall = 0 to 200 kN/m² (from initial soil sampling)

Consider the value from BS 8005:1995, Table 16
Partial factor against base sliding (\( f_s \)) = 1.3
Partial material factor to be applied to \( \tan \phi_p \) (\( f_{ms} \)) = 1.0
Partial material factor to be applied to $c'$ ($f_{ms}$) = 1.0
Partial material factor to be applied to $c_u$ ($f_{ms}$) = 1.0

Consider the value from BS 8005:1995, Table 17
Partial load factor for External load ($f_q$) = 1.5
Partial load factor for wall fill ($f_{fs}$) = 1.5

Consider the value from BS 8005:1995, Table 18
Partial load factor for load due to creep & shrinkage ($f_f$) = 1.2

Consider the value from BS 8005:1995, Table 3
Partial factor (ramification of failure) ($f_n$) = 1.1

Coefficient of active earth pressure for wall fill
$$K_{aw} = \frac{(1 - \sin \phi_w)}{(1 + \sin \phi_w)} = \frac{(1 - \sin 35°)}{(1 + \sin 35°)} = 0.27$$

Coefficient of active earth pressure for back fill
$$K_{ab} = \frac{(1 - \sin \phi_b)}{(1 + \sin \phi_b)} = \frac{(1 - \sin 30°)}{(1 + \sin 30°)} = 0.33$$

**External stability**

**i) SLIDING**

For a long term stability where there is soil-to-reinforcement contact at the base of the structure is as given below (as per BS 8006:1995 page 47)

$$f_s R_h \leq R_v \alpha \tan \phi' p + \frac{abc'c'L}{f_{ms}} + \frac{abc'c'L}{f_{ms}}$$

**LHS of equation (1)**

$$f_s R_h$$

Where $R_h$ = horizontal factored disturbing force as per Fig 21

$$= f_{fs} K_{ab} \gamma_b H^2/2 + K_{ab} f_q w_{s} H$$

$$= 1.5 \times 0.33 \times 18 \times 6^2/2 + 0.33 \times 1.5 \times 10 \times 6$$

$$= 160.38 + 29.7$$

$$= 190.08 \text{ kN/m}$$

Therefore $f_s R_h = 1.3 \times 190.08$

$$= 247.03 \text{ kN/m}$$

**RHS of equation (1)**

$$R_v \alpha \tan \phi' + \frac{abc'c'L}{f_{ms}}$$

Where $R_v$ = vertical factored resultant force as per Fig 22
Therefore
\[
R_v = \tan \phi' w + \frac{a_{bc} c' L}{f_{ms}} + \frac{0.85 \times 0 \times 4.2}{1.0} + \frac{1.6}{1.6}
\]
\[
= 781.2 \times 0.85 \times \tan 35 + 0.85 \times 0 \times 4.2
\]
\[
= 464.81 \text{kN/m}
\]

Therefore as per equation (1)
\[
247.03 < 464.81 \quad \text{i.e. LHS} < \text{RHS}
\]

This satisfy the sliding condition of the external stability. Hence the structure is safe with the required condition.

**OVERTURNING**

In reinforced earth structure restoring moment about the toe is greater than the overturning moment about the toe.

overturning moment about the toe as per Fig. 22
\[
= K_{ab} f_{fs} \gamma w H^3/6 + K_{ab} f_q w_s H^2/2
\]
\[
= 0.33 \times 1.5 \times 18 \times 6^3/6 + 0.33 \times 1.5 \times 10 \times 6^2/2
\]
\[
= 320.76 + 89.1
\]
\[
= 409.86 \text{kN}
\]

Restoring moment about the toe as per Fig. 22
\[
= f_{fs} \gamma w H L^2/2 + f_q w_s L^2/2
\]
\[
= 1.5 \times 19 \times 6 \times 4.2^2/2 + 1.5 \times 10 \times 4.2^2/2
\]
\[
= 1508.22 + 132.3
\]
\[
= 1640.52 \text{kN}
\]

Therefore Restoring moment is greater than the overturning moment.

Therefore it satisfy the condition of overturning. Hence the structure is safe with the required condition.

**Tilting / Bearing:-**

The bearing pressure as per Meyerhof distribution is as given below. (as per BS 8006: 1995 page 47).

\[
q_r = \frac{R_v}{L-2e}
\]
\[ R_v = (f_{\text{fs}} \gamma_w H + w_s f_q) \]
\[ = (1.5 \times 19 \times 6 + 10 \times 1.5) \]
\[ = 186 \text{ kN/m}^2 \]

Eccentricity (e) \[ = \frac{K_{ab} H^2 (f_{\text{fs}} \gamma_b H + 3 w_s f_q)}{6L (f_{\text{fs}} \gamma_w H + w_s f_q)} \]
\[ = 6 \times 4.2 (1.5 \times 19 \times 6 + 10 \times 1.5) \]
\[ = 2459.16 \]
\[ = 4687.20 \]
\[ = 0.525 \text{ m} \]

Bearing pressure per unit length \[ = \frac{R_v}{L-2e} \]
\[ = 186 / (4.2 - 2 \times 0.525) \]
\[ = 59.04 \text{ kN/m}^3 \]

Therefore bearing pressure per meter = 59.04 kN/m²

The bearing pressure is less than the allowable ground bearing pressure (i.e. Max. 200 kN/m²). This satisfies the bearing condition.
Hence the structure is safe with the required condition.

**Internal stability**

The design tensile strength of polymeric reinforcement is (as per BS 8006: 1995 page 35)
\[ T_D = \frac{T_{cr}}{f_m} \]
Where \( T_{cr} \) is the creep strength and \( f_m \) is the material factor and its value is 1.0

For 80 RE \( T_D = 38.5 / 1.0 = 38.5 \text{ kN/m} \)
For 55 RE \( T_D = 28.5 / 1.0 = 28.5 \text{ kN/m} \)

The tensile loads in the grids (as per BS 8006: 1995 page 51 & 52)
\[ T_j = K_{aw} \sigma_{vj} S_{vj} \]

Or \( T_j = T_{pj} \) In this case strip loading is not occur therefore the value of \( T_{sj} \) and \( T_{fj} \) is not considered

\[ T_j = \frac{K_{aw} (f_{\text{fs}} \gamma_w h_j + f_q w_s) S_{vj}}{1 - \frac{K_{ab} (f_{\text{fs}} \gamma_b h_j + 3 f_q w_s)(h_j / L)^2}{3(f_{\text{fs}} \gamma_w h_j + f_q w_s)}} \]

Therefore
\[ S_{vj} = \frac{T_j}{K_{aw} \sigma_{vj}} \]
Design Strength

\[ s_{vj} = \frac{K_{aw} (f_{s} \gamma_{w} h_{j} + f_{q} w_{s} )}{1- \frac{K_{ab} (f_{s} \gamma_{b} h_{j} + 3 f_{q} w_{s} ) (h_{j}/L)^{2} - 3(f_{s} \gamma_{w} h_{j} + f_{q} w_{s})}{1}} \]

Thus, at each depth \( h_{j} \) there will be a value of \( s_{vj} \) for each grid as shown Table 1.

<table>
<thead>
<tr>
<th>( h_{j} ) (m)</th>
<th>( s_{vj} ) for 80 RE (m)</th>
<th>( s_{vj} ) for 55RE (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>4.88</td>
<td>3.61</td>
</tr>
<tr>
<td>1.0</td>
<td>3.28</td>
<td>2.43</td>
</tr>
<tr>
<td>1.5</td>
<td>2.47</td>
<td>1.83</td>
</tr>
<tr>
<td>2.0</td>
<td>1.98</td>
<td>1.47</td>
</tr>
<tr>
<td>2.5</td>
<td>1.65</td>
<td>1.22</td>
</tr>
<tr>
<td>3.0</td>
<td>1.42</td>
<td>1.05</td>
</tr>
<tr>
<td>3.5</td>
<td>1.24</td>
<td>0.92</td>
</tr>
<tr>
<td>4.0</td>
<td>1.11</td>
<td>0.82</td>
</tr>
<tr>
<td>4.5</td>
<td>1.00</td>
<td>0.74</td>
</tr>
<tr>
<td>5.0</td>
<td>0.91</td>
<td>0.67</td>
</tr>
<tr>
<td>5.5</td>
<td>0.83</td>
<td>0.62</td>
</tr>
<tr>
<td>6.0</td>
<td>0.77</td>
<td>0.57</td>
</tr>
</tbody>
</table>

A layout of reinforcement can be determined from this data.

**Local stability check**

The higher resistance grid i.e 80RE is placed at the bottom of the structure as from the Table 5 the spacing is 0.7 consider and in the remaining height 55RE grid is consider for the same spacing. The layout of reinforcement with the vertical spacing is 0.7 is shown in Fig 31.
The resistance of jth reinforcing element should be checked against rupture and adherence failure.

\[ T_j = T_{pj} \]

\[ T_j = \frac{K_{aw} \left( f_{fs} \gamma_w h_j + f_{q w} \right) S_{v_j}}{1 - \frac{K_{ab} \left( f_{fs} \gamma_b h_j + 3 f_q w_s \right) \left( h_j / L \right)^2}{3(f_{fs} \gamma_w h_j + f_{q w})}} \]

i) Rupture: In local stability (as per Bs 8006:1995 Page 52)

The value of \( T_{D /f_n} \) and \( T_j \) is shown in Table 2.

<table>
<thead>
<tr>
<th>( h_j )</th>
<th>( S_{v_j} )</th>
<th>( T_j )</th>
<th>( T_{D /f_n} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.7</td>
<td>11.15</td>
<td>25.9</td>
</tr>
<tr>
<td>3.0</td>
<td>0.7</td>
<td>20.40</td>
<td>25.9</td>
</tr>
<tr>
<td>4.5</td>
<td>0.7</td>
<td>31.70</td>
<td>35.0</td>
</tr>
<tr>
<td>6.0</td>
<td>0.7</td>
<td>46.80</td>
<td>35.0</td>
</tr>
</tbody>
</table>

In Table 6 the value of \( T_j \) is greater than \( T_{D /f_n} \) at the depth greater than 5m, the rupture condition is does not satisfy. Therefore we decrease the vertical spacing between the grid, then consider \( S_{v_j} = 0.5 \). The layout of reinforcement with the vertical spacing is 0.5 is shown in Fig 3.

![Fig. 3](image-url)
Table- 3

<table>
<thead>
<tr>
<th>( h_j )</th>
<th>( S_{c,j} )</th>
<th>( T_j )</th>
<th>( T_{Dj} / f_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.5</td>
<td>7.96</td>
<td>25.9</td>
</tr>
<tr>
<td>3.0</td>
<td>0.5</td>
<td>14.60</td>
<td>25.9</td>
</tr>
<tr>
<td>4.5</td>
<td>0.5</td>
<td>22.60</td>
<td>25.9</td>
</tr>
<tr>
<td>6.0</td>
<td>0.5</td>
<td>33.47</td>
<td>35.0</td>
</tr>
</tbody>
</table>

The Table 3 satisfy the rupture condition for local stability. Hence the structure is safe.

ii) Adherence

For adherence the local stability consideration is as given below (as per BS 8006: 1995 page 52)

\[
P_j \geq \frac{T_j}{\mu L_{ej} (f_{s} \gamma_{w} h_j + f_q w_s) + \frac{a_{bc} c' L_{ej}}{f_p f_n} + \frac{f_{ms} f_p f_n}{f_{ms}}} \quad \text{............... (2)}
\]

The value of above is shown in Table 4, Where \( P_j = 2 \) \((1+1)\)

Table-4

<table>
<thead>
<tr>
<th>( h_j ) ( (m) )</th>
<th>( P_j ) ( (m) )</th>
<th>( T_j )</th>
<th>RHS Value of eq.(2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>2</td>
<td>7.96</td>
<td>0.186</td>
</tr>
<tr>
<td>3.0</td>
<td>2</td>
<td>14.60</td>
<td>0.135</td>
</tr>
<tr>
<td>4.5</td>
<td>2</td>
<td>22.60</td>
<td>0.112</td>
</tr>
<tr>
<td>6.0</td>
<td>2</td>
<td>33.47</td>
<td>0.104</td>
</tr>
</tbody>
</table>

Therefore for the above layer in Table 4. This satisfied the local stability for adherence. Hence the structure is safe in required condition.

Wedge stability Check

Now check

\[
\sum_{j=1}^{m} \left[ \frac{T_{Dj}}{f_n} \right] \geq T \quad \text{...} \quad \text{................................. ( 3)}
\]

or

\[
\sum_{j=1}^{m} \left[ \frac{P_j L_{ej}}{f_p f_n} \left\{ \mu f_{s} \gamma h_j + \mu f_{r} w_s + \frac{a_{bc} c'}{f_{ms}} \right\} \right] \geq T \quad \text{......................... (4)}
\]

The above two equation (as per BS 8006:1995 page 54) for each wedges at depth 1.5m, 3m, 4.5m and 6m as shown in Fig. 3
Where,

- \( P_j \) = Total horizontal width fo the top and bottom faces reinforcing element. (top + bottom \( 1+1 = 2 \))
- \( L_{ej} \) = Length of reinforcement in the resistant zone outside potential failure wedge.

\[ \text{In this case (i.e wall) with a level top containing fill and which supports uniform surcharge only the inclination of potential failure plane is } \beta = \tan (45^\circ - \phi' w/2) \text{ (Bs 8006:1995 page 54)} \]

- \( f_p \) = Partial factor for reinforcement pull-out resistance, 1.3 (from BS 8006:1995 Table 16)
- \( \mu = (\alpha \tan \phi' w) / f_{ms} = 0.85 \times \tan 35^\circ/1.0 = 0.595 = 0.60 \)

The value of the force to be resisted with active surcharge

\[ T = \frac{h \tan \beta (f_{fs} \gamma w h_j + 2 f_q w_s)}{2 \tan (\phi' w + \beta)} \]

Equation (4) and equation (5) check for each grid and for each wedge, both with and without surcharge as shown in Table 5.

**Table 5**

<table>
<thead>
<tr>
<th>( H ) (m)</th>
<th>( T ) (kN/m)</th>
<th>Effective number of grids</th>
<th>Total reinforcement resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With surcharge</td>
<td>Without surcharge</td>
<td>55RE</td>
</tr>
<tr>
<td>1.5</td>
<td>14.73</td>
<td>8.66</td>
<td>3</td>
</tr>
<tr>
<td>3.0</td>
<td>46.78</td>
<td>34.65</td>
<td>6</td>
</tr>
<tr>
<td>4.5</td>
<td>96.14</td>
<td>77.96</td>
<td>9</td>
</tr>
<tr>
<td>6.0</td>
<td>162.81</td>
<td>138.60</td>
<td>10</td>
</tr>
</tbody>
</table>

In above table total reinforcement is greater than \( T \) and therefore wedge check is satisfied. Hence the structure satisfy all required condition. Now, it is safe.
Design Example of reinforced Steep Slopes

We have to reinforce the slopes of a steep sided embankment. It is 6 m high very wide with a slope angle of 70°. The maximum surcharge load on the crest is 10 kN/m². The foundation is levelled and with adequate bearing capacity. The design life is 50 years. The soil for the construction is a sandy gravel with a small percentage of silt available in situ, having the following characteristics:

- $C' = 10$ kPa,
- $\phi' = 34^\circ$,
- $\gamma = 20$ kN/m³

We have to consider that the embankment may be wet for long periods because the rainfall in the region is quite high.

We want to reinforce the steep slope of the embankment with, TENAX/TT301 geogrids.

**Solution:**

Input Values:

- $H = 6.0$ m
- $w_s = 10$ kN/m²
- $\gamma = 20$ kN/m³
- $\phi' = 34^\circ$
- $ru = 0.25$

Hence, $H^\wedge = H + (w_s / \gamma) = 6.5$ m

The following values are selected:

- FSover tension or FS creep = 1.30
- FS construction = 1.00 (sandy gravel soil)
- FS biological = 1.00
- FS chemical = 1.00
- FS junction = 1.00

Then we have

$FS_{grid} = FS_{creep}.FS_{construction}.FS_{junction}.FS_{chemical}.FS_{biological}$

$= 1.30 \times 1.00 \times 1.00 \times 1.00 \times 1.00$

$= 1.30$

The allowable strength of Tenax TT301 geogrids is 23.5 kN/m

Design strength ($P$) = $T_{ult} / FS_{grid}$

$= 23.5 / 1.30$

$= 18.1$ KN/m

We assume $c' = 10$ kPa, $\phi' = 34^\circ$

In this case we do not know $u(z)$. In order to take into account the pore water pressure which may develop during periods of intense rainfall, it is better to assume $ru = 0.25$.

From Annexure - C

$K = 0.28$ \quad ($ru = 0.25, \phi = 34^\circ$, slope angle $\beta = 70^\circ$ from Annexure C Fig A4 of Jewell Chart 1991)
\[(L/H)_{ovrl} = 0.63 \quad (r_u = 0.25, \phi = 34^\circ, \text{slope angle } \beta = 70^\circ \quad \text{from Annexure C Fig A5 of Jewell Chart 1991})\]

\[(L/H)_{ds} = 0.58 \quad (r_u = 0.25, \phi = 34^\circ, \text{slope angle } \beta = 70^\circ \quad \text{from Annexure C Fig A6 of Jewell Chart -1991})\]

here \[(L/H)_{ovrl} > (L/H)\]

Therefore \[L = (L/H)_{ovrl} \times H = 0.63 \times 6.5 = 4.10 \text{ m}\]

We choose a compaction lift of 0.30 m accordingly it will be \[v = 0.30 \text{ m}, \text{then}\]

\[P \quad Q = \frac{\gamma_v}{K. \gamma_v} \]

\[Q = \frac{18.1}{(0.28 \times 20 \times 0.30)} = 10.68\]

Calculate the zones of equal spacing as shown in Table 1:

<table>
<thead>
<tr>
<th>Spacing</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S_{v1}) = 0.3</td>
<td>Q - Q/2 = 10.68 to 5.34</td>
<td>(S_1 = H - Q/2 = 6.50 - 5.34 = 1.16)</td>
</tr>
<tr>
<td>(S_{v2}) = 0.6</td>
<td>Q/2 - Q/3 = 5.34 to 3.56</td>
<td>(S_2 = Q/2 - Q/3 = 5.34 - 3.56 = 1.78)</td>
</tr>
<tr>
<td>(S_{v3}) = 0.9</td>
<td>Q/3 - (w_s / \gamma) = 3.56 to 0.50</td>
<td>(S_3 = Q/3 - \frac{w_s}{\gamma} = 3.56 - 0.50 = 3.06)</td>
</tr>
</tbody>
</table>

Hint:
- Col I: Assume the vertical spacing consider 0.3m.
- Col. II: Depth of spacing range i.e. Q to Q/2, Q/2 to Q/3, Q/3 to Q/4, ... .
- Col. III: Total thickness of the zone i.e. \(H - Q/2, Q/2 - Q/3,\) ... . If the thickness is less than the spacing between the grid then subtract \(w_s / \gamma\) against the second value.

Calculate the number and position of the required layers as shown in Table 2:

<table>
<thead>
<tr>
<th>IV</th>
<th>V</th>
<th>VI</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S_i / S_{vi})</td>
<td>(N_i)</td>
<td>(R_i)</td>
</tr>
<tr>
<td>1.16/0.30 = 3.86</td>
<td>(N_1 = 3)</td>
<td>(R_1 = 1.16 - 3 \times 0.30 = 0.26)</td>
</tr>
<tr>
<td>2.04/0.60 = 3.40</td>
<td>(N_2 = 3)</td>
<td>(R_2 = 2.04 - 3 \times 0.60 = 0.24)</td>
</tr>
<tr>
<td>3.30/0.90 = 3.67</td>
<td>(N_3 = 3)</td>
<td>(R_3 = 3.30 - 3 \times 0.90 = 0.60)</td>
</tr>
</tbody>
</table>

\(N_{tot} = 3 + 3 + 3 = 9\)

We add a 10th layer spaced 0.60 m near to the crest.

Hint:
- Col. IV: Finding the number of layer is the zone i.e divide the Col. III by Col. I
- Col. V: Rounded down the number of layer i.e rounded down Col IV
- Col. VI: Finding the remaining thickness i.e Reminder = Col III – Col V \times Col I
Col. VII : Finding the thickness of next zone i.e. current thickness (Col III) + previous reminder (Col VI).

Calculate the gross horizontal force for equilibrium:
\[ HT = \frac{1}{2} K \times \gamma \times H^2. \]
\[ = \frac{1}{2} \times 0.28 \times 20 \times 6.5^2 = 119.15 \text{ kN/m} \]

Check the average tensile force in the geogrids
\[ \left( \frac{HT}{N_{tot}} \right) = \frac{119.15}{10} = 11.91 \text{ kN/m} \]
\[ P = 18.08 \text{ kN/m} \]

The condition for safe design strength is \( \left( \frac{HT}{N_{tot}} \right) \leq P \)
Here this condition is verified.

Calculate the wrapping length. Conservatively we set for this type of soil:
for the lower layer:
\[ \tilde{z}_1 = z_1 + (w_s / \gamma) = 5.7 \times 0.5 = 6.2 \text{ m} \]
\[ L_{r1} = \frac{F_{\text{wrap}} \cdot K \cdot (\tilde{z}_1 + S_{v1}/2)}{\tilde{z}_1 \cdot f_{\text{ds}} \cdot \tan \phi} = 1.30 \times 0.28(6.2+0.30/2) \times 0.3 = 0.21 \text{ m} \]

for the 10th layer:
\[ \tilde{z}_{10} = z_{10} + (w_s / \gamma) = 0.6 \times 0.5 = 1.1 \text{ m} \]
\[ L_{10} = \frac{F_{\text{wrap}} \cdot K \cdot (\tilde{z}_{10} + S_{v10}/2)}{\tilde{z}_{10} \cdot f_{\text{ds}} \cdot \tan \phi} = 1.30 \times 0.28(1.1+0.90/2) \times 0.9 = 0.87 \text{ m} \]

We impose that the minimum wrapping length is 1.5 meter, so it is necessary to consider a wrapping length of 1.5 m of each layer.
Hence safe
If this condition is not verified increase the no. of geogrid layers with change in spacing.
The final layout is shown in Fig.1 :-

---

58
Steep Reinforced Slope Design Charts (Jewell, 1991)

Minimum Required Force $K_{req}$

Minimum Required Length
Overall Stability

Minimum Required Length
Direct Sliding

$R_u = u/(\gamma z) = 0.00$

Fig A1: for $K_{req}$ when $r_u = 0.0$
Fig A2: for $(L/H)_{ovrl}$ when $r_u = 0.0$
Fig A3: for $(L/H)_{ds}$ when $r_u = 0.0$
Steep Reinforced Slope Design Charts (Jewell, 1991)

Fig A4: for $K_{req}$ when $r_u=0.25$
Fig A5: for $(L/H)_{ovrl}$ when $r_u=0.25$
Fig A6: for $(L/H)_{ds}$ when $r_u=0.25$
Steep Reinforced Slope Design Charts (Jewell, 1991)

Fig A7: for $K_{req}$ when $r_u = 0.50$
Fig A8: for $(L/H)_{ovrl}$ when $r_u = 0.50$
Fig A9: for $(L/H)_{ds}$ when $r_u = 0.50$