Chapter 9

DESIGN OF GABION FACED REINFORCED EARTH RETAINING WALLS

9.1 GENERAL

Design of any structure begins with the selection of trial dimensions. It is then analysed to understand the effect of loads on the behaviour of the structure which yields forces and deformations as results. This information is then compared with criteria for failure conditions to arrive at a safe design.

The gabion faced gravity walls which are more suitable for small heights, follow the conventional design criteria adopted for dry rubble or random rubble masonry walls as per BS 8002 : 1994. This chapter considers the design and analysis of gabion faced reinforced soil walls which are expected to be more economical for larger heights. The modern trend in the design of any structure is the use of the limit state method which always results in a safe and economical design. So this work concentrates on the design of gabion faced reinforced soil walls based on the limit state method.

The conventional practice of design of these walls involves selecting the dimensions of the wall, giving a high factor of safety against different modes of failure which makes the design uneconomical. Moreover, the conventional design involves a trial and error process, which ultimately results in repeated calculations to arrive at a safe design. Here an attempt is made to develop design charts which makes the design process handy and speedy.

The design of gabion faced reinforced soil walls, described herein, is based on the limit state method as per BS 8006: 1995, a detailed description of which is given in Chapter 3. A typical case of gabion faced reinforced earth wall is studied and the conventional design procedure for the same is illustrated initially. This is then followed by the development of design charts using dimensional analysis of the limiting conditions. These charts are expected to
yield a safe, economical and speedy design procedure which overcomes the difficulties mentioned above.

9.2 DESIGN PROCEDURE

The following steps are resorted for the design of gabion faced reinforced earth wall.

- Fix the trial dimensions of the wall.
- Determine the forces acting on the wall.
- Check whether the maximum bearing pressure at the base of the wall is within the allowable limit (External stability check).
- Check whether sliding resistance exceeds the active horizontal force by a suitable safety factor (External stability check).
- Check for strength and pullout resistance of the reinforcement layers (Internal stability check).

These steps are repeated iteratively until a suitable design that meets all criteria is achieved.

One metre length of the wall is considered for the analysis. For selecting the overall geometry of the wall, the initial length of reinforcement is taken as 0.7H where H is the height of the wall. The embedment depth (Dm) may be fixed as the maximum value obtained from Eqns. 3.5 and 3.6. The width of the gabion facing depends on the dimensions of the gabions; usually available as multiples of 0.5m. For reinforced soil type walls, the facing width is usually kept to a minimum value of 0.5m. The initial sizing of a typical gabion faced reinforced earth wall is shown in Fig. 9.1.
The analysis detailed below accounts for the effects of gabion facing wherever necessary. Surcharge considered is uniformly distributed load only. The effects of vertical strip loading or horizontal shear loading over the reinforced fill are not considered in the analysis.

![Initial dimensions of the wall](image)

**Fig. 9.1 Initial dimensions of the wall**

### 9.2.1 Forces acting on the wall

The main forces acting on gabion walls are the vertical forces from the weight of the gabions, weight of reinforced soil block, the lateral earth pressure acting on the back face of this soil block and the surcharge over the backfill. These forces are used herein to illustrate the main design principles. If other forces are encountered, such as vehicular loads or seismic loads, they must also be included in the analysis. The soil properties for the reinforced soil block, retained fill and foundation, gabion facing, together with the superimposed loads considered in the stability calculations are shown in Fig. 9.2. In the figure it is assumed that the intensity of surcharge over the backfill and the retained soil is the same. Also the foundation soil and the retained earth are of the same type. The forces acting may be calculated as:

- Weight of gabion facing, \( W_k \)  
  \[ W_k = b H \gamma_k \]  
  \[ (9.1) \]

- Weight of reinforced soil block, \( W_s \)  
  \[ W_s = L H \gamma_{sl} \]  
  \[ (9.2) \]

- Weight due to surcharge, \( W_q \)  
  \[ W_q = q L \]  
  \[ (9.3) \]

183
Lateral force due to retaining soil, $P_s = K_{a2} \gamma_{s2} H^2 / 2$ ................. (9.4)
Lateral force due to surcharge, $P_q = K_{a2} q H$ ....................... (9.5)

Fig. 9.2 Material properties and principal forces

Fig. 9.3 Pressure distribution along base of wall

The coefficient of active earth pressure, $K_{a2}$ may be calculated as:

$$K_{a2} = \frac{1 - \sin \phi_{a2}}{1 + \sin \phi_{a2}}$$

................... (9.6)
9.2.2 External stability analysis

9.2.2.1 Bearing and tilt failure

The factored bearing pressure acting at the base of the wall based on Meyerhof distribution may be modified for gabion faced walls by taking the equivalent base length as \( b + L \) in Eqn. 3.7. Hence,

\[
q_r = \frac{R_v}{b + L - 2e}
\] .......................... (9.7)

As per Table 3.1, assuming combination A to be critical for bearing and tilt failure,

\[
R_v = 1.5 (W_g + W_s + W_q)
\] .......................... (9.8)

\[
e = \frac{\{b + L\} / 2} - (M / R_v)
\] .......................... (9.9)

\[
M = \text{net factored moments acting about the toe of the system}
\]

\[
M_r = \text{Total factored resisting moments about toe}
\]

\[
M_r = 1.5 (W_g b/2 + W_s (b + L/2) + W_q (b + L/2))
\] .......................... (9.11)

\[
M_o = \text{Total factored overturning moments about toe}
\]

\[
M_o = 1.5 (P_s H/3 + P_q H/2)
\] .......................... (9.12)

Applying the partial material factor to \( q_{ult} \) from Table 3.2, the Eqn. 3.8 gets modified as follows, which becomes the critical condition for bearing and tilt failure:

\[
q_r \leq q_{ult} / 1.35 + \gamma_s D_m
\] .......................... (9.13)

9.2.2.2 Sliding along the base

In the case of gabion faced reinforced soil walls, at the base of the reinforced soil block, the gabion wire mesh comes in contact with the foundation soil. As per Table 3.1, combination B is considered as the worst combination for sliding along the base. Applying appropriate partial material factors from Table 3.2, Eqn. 3.9 for long term stability, where there is reinforcement – to – soil contact at the base of the structure, gets modified as:
where, 1.3 is the partial safety factor against sliding between soil and reinforcement obtained from Table 3.2, $a$ is the interaction coefficient relating soil - reinforcement interfacial friction angle with $\tan \phi_{s2}$ which is usually taken as 2/3 (Gulhati and Dutta, 2005), $\phi_{s2}$ and $c_{s2}$ are the internal friction angle and cohesion of the retained fill respectively (assuming that the foundation soil and the retained soil are the same and the retained soil is weaker than the backfill), $\alpha$ is the adhesion coefficient relating soil cohesion to soil - reinforcement bond usually taken as unity for stiff clays and 2/3 for soft clays, and, 1.6 and 1 are the partial material factors applied to cohesion and angle of internal friction respectively taken from Table 3.2. The horizontal factored disturbing force, $R_h$, is calculated as:

\[
1.3 R_h \leq R_c \tan \phi_{s2} + \frac{a c_{s2} L}{1.6} \quad \text{...............} (9.14)
\]

\[
R_h = 1.5 (P_s + P_d) \quad \text{...............} (9.15)
\]

\[
R_v = W_s + W_s \quad \text{...............} (9.16)
\]

$W_q$ need not be considered for sliding check as it has a partial factor of 0.0 as per Table 3.1 for load combination B. The corresponding partial load factor is unity for the stabilising forces and 1.5 for the disturbing forces.

**9.2.3 Internal stability analysis**

**9.2.3.1 Rupture of reinforcement**

Load combination A (Table 3.1) is considered critical in this case. Eqn. 3.10 gets modified as follows, neglecting the effects of vertical strip loads and horizontal shear loads on the reinforced soil block.

\[
T_j = T_{pj} - T_{cj} \quad \text{...............} (9.17)
\]

Applying partial factors from Tables 3.1 and 3.2, Eqn. 3.11 and Eqn. 3.12 for calculating $T_{pj}$ and $T_{cj}$, become:
\[ T_{n} = \frac{1.5 K_{e1} (\gamma_{s1} h_{j} + q) S_{n}}{K_{s2} (\gamma_{s2} h_{j} + 3q) \left( \frac{h_{j}}{L} \right)^{2}} \left( 1 - \frac{3(\gamma_{s1} h_{j} + q)}{K_{e}} \right) \] ........................ (9.18)

\[ T_{q} = 2 S_{n} \frac{c}{1.6} \sqrt{K_{e}} \] ........................ (9.19)

where, 1.5 is the partial factor applied to loads in combination A (Table 3.1) and 1.6 is the partial material factor applied to cohesion as per Table 3.2. Condition for stability against mesh rupture from Eqn. 3.13 gets modified as:

\[ \frac{T_{D}}{1.1} \geq T_{j} \] ........................ (9.20)

in which, \( T_{j} \) is the maximum value obtained from Eqn. 9.17. The gabion faced retaining structures are mainly built along highways and beside railway lines and hence the partial factor for economic ramifications of failure is taken as 1.1 here.

In the case of gabion mesh as seen in Chapter 3, the basic material is stainless steel which is heavily galvanised or galvanised plus PVC coated having thickness of approximately 4mm. From Section 3.4.3, it may be seen that, for galvanised steel mesh, \( f_{m} \) may be taken as 1.5 as it is subjected to axial loads only. For galvanised mesh with PVC coating, the partial material factors for gabion mesh reinforcement may be chosen as shown in Table 9.1. Substituting the partial factor values from Table 9.1 in Eqn. 3.2, for PVC coated gabion mesh, \( f_{m} \) may be obtained as 1.5. Hence, it may be inferred that, whatever be the type, the gabion mesh has a partial material factor of 1.5. Hence, in accordance with Eqn. 3.1, design strength of gabion mesh,

\[ T_{D} = T_{ult} / 1.5 \] ........................ (9.21)
Table 9.1 Determination of $f_m$ for PVC coated gabion mesh

<table>
<thead>
<tr>
<th>Partial factor</th>
<th>Value chosen</th>
<th>Reason</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m111}$</td>
<td>1.0</td>
<td>Assuming that the manufacturing of mesh is done according to standards</td>
</tr>
<tr>
<td>$f_{m112}$</td>
<td>1.0</td>
<td>Assuming that the mesh size is greater than the minimum specified dimension used for determining the base strength</td>
</tr>
<tr>
<td>$f_{m121}$</td>
<td>1.0</td>
<td>Assuming that large quantities of data for determination of base strength over a long period of time are available</td>
</tr>
<tr>
<td>$f_{m122}$</td>
<td>1.0</td>
<td>Assuming that the extrapolation of test data can be done over one log cycle of time</td>
</tr>
<tr>
<td>$f_{m21}$</td>
<td>1.5</td>
<td>Assuming a value greater than unity since, for gabion mesh, minimum steel thickness is less than 4 mm</td>
</tr>
<tr>
<td>$f_{m22}$</td>
<td>1.0</td>
<td>Since protective layer of PVC coating is provided</td>
</tr>
</tbody>
</table>

9.2.3.2 Loss of adherence of reinforcement

The perimeter $P_j$ (Eqn. 3.14) of the $j$th layer of reinforcing elements is modified as follows:

$$
P_j \geq \frac{T_j}{(2/3) \tan \phi_{sl} L_{ij} y_{sl} h_j} + \frac{\alpha c L_{ij}}{1.3 \times 1.1 + 1.6 \times 1.3 \times 1.1} \quad \text{............... (9.22)}$$

where, 1.3 is the partial factor for reinforcement pullout resistance taken from Table 3.2. As per Section 9.2.2.2, $\alpha = 1$ for stiff clays and $2/3$ for soft clays, and $2/3$ is the soil – reinforcement interaction factor ($\omega$). For gabion faced walls, since the reinforcement is in the form of sheet, perimeter, $P_j = 2 \text{ m}$, considering unit length of the wall for analysis. It is also to be noted that load combination B is critical for reinforcement pullout considerations (Table 3.1) and hence the effect of surcharge may be neglected and partial load factor may be taken as unity for self weights.
Thus Eqn. 9.22 becomes:

\[
2L_{eq} \geq \frac{T_j}{(2/3) \tan \phi_{st} \gamma_{st} h_j + \alpha c} \times 1.3 \times 1.1 + 1.6 \times 1.3 \times 1.1 \quad (9.23)
\]

in which, \( T_j \) may be calculated from Eqn. 9.17. But Eqn. 9.18 for obtaining \( T_{pi} \) gets modified as:

\[
T_{pi} = \frac{K_{st} \gamma_{st} h_j S_{ij}}{1.5 K_{st2} \gamma_{st2} h_j \left( \frac{h_j}{L} \right)^2} \quad (9.24)
\]

The modification is due to the load combination B where \( f_{sk} = 1.0 \) for the self weights, \( f_{sl} = 1.5 \) for earth pressure behind the structure and \( f_{q} = 0 \) (Table 3.1).

**9.2.4 Serviceability limit state**

For gabion mesh, the reinforcement is metallic and hence creep is negligible and consequently, the strain \( \varepsilon_j \) in the jth layer of reinforcements may be estimated from Eqn. 3.15. \( T_{nj} \) is the average tensile load along the length of the jth layer of reinforcements which may be calculated as \( T_j \) from Eqn. 9.17. But Eqn. 9.18 for calculating \( T_{pi} \) gets modified as:

\[
T_{pi} = \frac{K_{st} \gamma_{st} h_j S_{ij}}{K_{st2} \gamma_{st2} h_j \left( \frac{h_j}{L} \right)^2} \quad (9.25)
\]

The modification is because of the reason that the load combination to be considered for serviceability limit state is the Combination C in which \( f_{sk} = 1 \) and \( f_{q} = 0 \) as seen from Table 3.1.
9.3 DESIGN EXAMPLE

The design and analysis of gabion faced reinforced earth retaining walls explained in Section 9.2 is illustrated with standard numerical values in this section.

9.3.1 Data available from site

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of wall, $H$</td>
<td>5 m</td>
</tr>
<tr>
<td>Unit weight of material in the gabion box, $\gamma_k$</td>
<td>20 kN/m³</td>
</tr>
<tr>
<td>Unit weight of retained soil, $\gamma_{s2}$</td>
<td>20 kN/m³</td>
</tr>
<tr>
<td>Cohesion of retained soil, $c_{s2}$</td>
<td>0 kPa</td>
</tr>
<tr>
<td>Internal friction angle of retained soil, $\phi_{s2}$</td>
<td>35°</td>
</tr>
<tr>
<td>Assuming that the retained soil itself is used for backfilling, Unit weight of backfill material, $\gamma_{s1}$</td>
<td>20 kN/m³</td>
</tr>
<tr>
<td>Cohesion of backfill material, $c_{s1}$</td>
<td>0 kPa</td>
</tr>
<tr>
<td>Internal friction angle of backfill, $\phi_{s1}$</td>
<td>35°</td>
</tr>
<tr>
<td>Ultimate bearing capacity of foundation soil, $q_{ult}$</td>
<td>650 kN/m²</td>
</tr>
<tr>
<td>Surcharge, $q$</td>
<td>10 kN/m²</td>
</tr>
<tr>
<td>Ultimate tensile strength of mesh confined in soil, $T_{ult}$</td>
<td>51 kN/m</td>
</tr>
<tr>
<td>Allowable strain in mesh, $\varepsilon_{ult}$</td>
<td>10%</td>
</tr>
<tr>
<td>Stiffness of the mesh, $EA$</td>
<td>8000 kN</td>
</tr>
</tbody>
</table>

9.3.2 Initial dimensions

From the available data listed above, the initial dimensions of the structure to be designed are fixed as per Section 9.2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of reinforcement ($L \approx 0.7H$)</td>
<td>4 m</td>
</tr>
<tr>
<td>Depth of embedment, $D_m$</td>
<td>Max of: $(H/20$ or $1.35 \times 10^{-3} \times q_{ult})$</td>
</tr>
<tr>
<td>Width of gabion facing, $b$</td>
<td>0.5 m</td>
</tr>
</tbody>
</table>

9.3.3 Forces acting on the wall

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of gabion facing, $W_g$</td>
<td>50 kN/m (from Eqn. 9.1)</td>
</tr>
<tr>
<td>Weight of reinforced soil block, $W_s$</td>
<td>400 kN/m (from Eqn. 9.2)</td>
</tr>
<tr>
<td>Weight due to surcharge, $W_q$</td>
<td>40 kN/m (from Eqn. 9.3)</td>
</tr>
</tbody>
</table>
Lateral force due to retaining soil, \( P_s \) = 67.75 kN/m (from Eqn. 9.4)
Lateral force due to surcharge, \( P_q \) = 13.55 kN/m (from Eqn. 9.5)

9.3.4 External stability analysis

9.3.4.1 Bearing and tilt failure

Resultant of all factored vertical loads, \( R_v \) = 735 kN/m

Total factored resisting moments about toe, \( M_r \) = 1668.75 kNm/m

Total factored overturning moments about toe, \( M_o \) = 220.2 kNm/m

Net factored moments about toe, \( M \) = 1448.57 kNm/m

Eccentricity, \( e \) = 0.28 m

Factored bearing pressure at the base of the wall, \( q_r \) = 186.47 kPa

On calculating the term, \( q_{ult} / (1.35 + \gamma D_m) \), the value is obtained as 491.5kPa which is greater than 186.47 kPa, the factored bearing pressure acting at the base of the wall. Hence it can be concluded that the designed structure is safe against tilt and bearing failures.

9.3.4.2 Sliding along the base

Horizontal factored disturbing force, \( R_h \) = 121.9 kN/m

Resultant of all factored vertical loads, \( R_v \) = 450 kN/m

Factored sliding force (1.3 \( R_h \)) = 158.5 kN/m

Factored resisting force (\( R_v \) tan \( \phi_s \) (taking \( c_{s2} \) as zero)) = 210 kN/m

Since the factored sliding force is less than the factored resisting force, the structure may be considered safe against sliding failure about its base.
9.3.5 Internal stability analysis

Design strength of the gabion wire mesh, $T_D = 34$ kN/m (from Eqn. 9.21). For internal stability analysis, it is assumed that the spacing between reinforcements ($S_{ij}$) is 0.5m. Assuming that the failure wedge passes through a plane inclined at an angle $(45^\circ + \phi_{sl}/2)$ with the horizontal,

Length of reinforcement in the active zone, $L_{ai} = (H - h_i) \tan (45^\circ - \phi_{sl}/2)$

.......................... (9.26)

Minimum length of reinforcement required for internal stability, $L_{min} = L_{ai} + L_{cj}$

.......................... (9.27)

Table 9.2 shows the internal stability calculations against mesh breakage and it can be seen that the tension developed in the reinforcements is less than the factored design strength. Table 9.3 shows the internal stability calculations against mesh pullout and it can be seen that the maximum length of reinforcement required as per internal stability calculations is 2.6m which is very much less than the provided length of 4m. Hence the designed structure may be considered safe against internal failure modes.

Table 9.2 Internal stability calculations for mesh breakage

<table>
<thead>
<tr>
<th>$h_i$ (m)</th>
<th>$S_{ij}$ (m)</th>
<th>$T_{pj}$ (kN/m)</th>
<th>$T_{cj}$ (kN/m)</th>
<th>$T_f$ (kN/m)</th>
<th>Check (Eqn. 9.20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>0.50</td>
<td>4.08</td>
<td>0</td>
<td>4.08</td>
<td>Safe</td>
</tr>
<tr>
<td>1.00</td>
<td>0.50</td>
<td>6.16</td>
<td>0</td>
<td>6.16</td>
<td>Safe</td>
</tr>
<tr>
<td>1.50</td>
<td>0.50</td>
<td>8.29</td>
<td>0</td>
<td>8.29</td>
<td>Safe</td>
</tr>
<tr>
<td>2.00</td>
<td>0.50</td>
<td>10.49</td>
<td>0</td>
<td>10.49</td>
<td>Safe</td>
</tr>
<tr>
<td>2.50</td>
<td>0.50</td>
<td>12.80</td>
<td>0</td>
<td>12.80</td>
<td>Safe</td>
</tr>
<tr>
<td>3.00</td>
<td>0.50</td>
<td>15.22</td>
<td>0</td>
<td>15.22</td>
<td>Safe</td>
</tr>
<tr>
<td>3.50</td>
<td>0.50</td>
<td>17.80</td>
<td>0</td>
<td>17.80</td>
<td>Safe</td>
</tr>
<tr>
<td>4.00</td>
<td>0.50</td>
<td>20.56</td>
<td>0</td>
<td>20.56</td>
<td>Safe</td>
</tr>
<tr>
<td>4.50</td>
<td>0.50</td>
<td>23.56</td>
<td>0</td>
<td>23.56</td>
<td>Safe</td>
</tr>
<tr>
<td>5.00</td>
<td>0.25</td>
<td>13.42</td>
<td>0</td>
<td>13.42</td>
<td>Safe</td>
</tr>
</tbody>
</table>
### Table 9.3 Internal stability calculations for mesh pullout

<table>
<thead>
<tr>
<th>$h_j$</th>
<th>$S_{vj}$</th>
<th>$T_{pj}$ (Eqn. 9.24)</th>
<th>$T_{ej}$ (Eqn. 9.19)</th>
<th>$T_e$ (Eqn. 9.17)</th>
<th>$L_{ej}$ (Eqn. 9.26)</th>
<th>$L_{ej}$ (Eqn. 9.23)</th>
<th>$L_{min}$ (Eqn. 9.27)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(m)</td>
<td>(m)</td>
<td>(kN/m)</td>
<td>(kN/m)</td>
<td>(kN/m)</td>
<td>(m)</td>
<td>(m)</td>
<td>(m)</td>
</tr>
<tr>
<td>0.50</td>
<td>0.50</td>
<td>1.36</td>
<td>0</td>
<td>1.36</td>
<td>2.34</td>
<td>0.21</td>
<td>2.55</td>
</tr>
<tr>
<td>1.00</td>
<td>0.50</td>
<td>2.73</td>
<td>0</td>
<td>2.73</td>
<td>2.08</td>
<td>0.21</td>
<td>2.29</td>
</tr>
<tr>
<td>1.50</td>
<td>0.50</td>
<td>4.14</td>
<td>0</td>
<td>4.14</td>
<td>1.82</td>
<td>0.21</td>
<td>2.03</td>
</tr>
<tr>
<td>2.00</td>
<td>0.50</td>
<td>5.61</td>
<td>0</td>
<td>5.61</td>
<td>1.56</td>
<td>0.21</td>
<td>1.78</td>
</tr>
<tr>
<td>2.50</td>
<td>0.50</td>
<td>7.15</td>
<td>0</td>
<td>7.15</td>
<td>1.30</td>
<td>0.22</td>
<td>1.52</td>
</tr>
<tr>
<td>3.00</td>
<td>0.50</td>
<td>8.80</td>
<td>0</td>
<td>8.80</td>
<td>1.04</td>
<td>0.22</td>
<td>1.27</td>
</tr>
<tr>
<td>3.50</td>
<td>0.50</td>
<td>10.58</td>
<td>0</td>
<td>10.58</td>
<td>0.78</td>
<td>0.23</td>
<td>1.01</td>
</tr>
<tr>
<td>4.00</td>
<td>0.50</td>
<td>12.54</td>
<td>0</td>
<td>12.54</td>
<td>0.52</td>
<td>0.24</td>
<td>0.76</td>
</tr>
<tr>
<td>4.50</td>
<td>0.50</td>
<td>14.72</td>
<td>0</td>
<td>14.72</td>
<td>0.26</td>
<td>0.25</td>
<td>0.51</td>
</tr>
<tr>
<td>5.00</td>
<td>0.25</td>
<td>8.59</td>
<td>0</td>
<td>8.59</td>
<td>0.00</td>
<td>0.24</td>
<td>0.13</td>
</tr>
</tbody>
</table>

### 9.3.6 Serviceability limit considerations

The tension and strain in each layer of reinforcement is calculated using Eqn. 9.25 and Eqn. 3.15 respectively and tabulated in Table 9.4. The strain values developed in the reinforcements are below the permissible limit of 10%. Hence the deformations developing within the system are acceptable.

### Table 9.4 Strain in the reinforcements

<table>
<thead>
<tr>
<th>$h_j$ (m)</th>
<th>$T_e$ (Eqn. 9.25) (kN/m)</th>
<th>$e_j$ (Eqn. 3.15) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>1.36</td>
<td>0.068</td>
</tr>
<tr>
<td>1.00</td>
<td>2.73</td>
<td>0.136</td>
</tr>
<tr>
<td>1.50</td>
<td>4.12</td>
<td>0.206</td>
</tr>
<tr>
<td>2.00</td>
<td>5.55</td>
<td>0.277</td>
</tr>
<tr>
<td>2.50</td>
<td>7.02</td>
<td>0.351</td>
</tr>
<tr>
<td>3.00</td>
<td>8.56</td>
<td>0.428</td>
</tr>
<tr>
<td>3.50</td>
<td>10.19</td>
<td>0.509</td>
</tr>
<tr>
<td>4.00</td>
<td>11.92</td>
<td>0.596</td>
</tr>
<tr>
<td>4.50</td>
<td>13.77</td>
<td>0.688</td>
</tr>
<tr>
<td>5.00</td>
<td>7.89</td>
<td>0.394</td>
</tr>
</tbody>
</table>
9.4 DEVELOPMENT OF DESIGN CHARTS

In the conventional design procedure, if at any stage, the design is found to be unsafe or uneconomical, the entire design procedure has to be reworked, which makes the design process lengthy and cumbersome. Design charts always provide easy and quick methods to arrive at suitable design parameters. For the development of these charts, often it becomes necessary to make suitable assumptions in the design procedure keeping in view that the fundamental principles are followed.

9.4.1 Assumptions

In order to simplify the preparation of design charts, certain assumptions were used without causing much variations in the final results.

1. The soil at site is used as the backfill material. Even though the ideal backfill material is a frictional material, the non availability of such type of soil in the near locality may necessitate the use of soil at site itself as a backfill material. Moreover, the design using soils with low frictional resistance will be on the safer side.

2. The retained soil and the foundation soil are the same, which is the condition in most of the cases.

3. The gabion fill and backfill are of the same density and hence the ratio of unit weight of the material in the gabion box \( (\gamma_g) \) to unit weight of backfill material \( (\gamma_b) \) is taken as unity. Even though in actual practice this may not be true, the actual ratio between the two will lead only to lesser values of length of reinforcement \( (L) \), which means, this assumption provides conservative results.

4. Effect of water table is not considered except in assigning corresponding in situ densities of the soil. Even though there is presence of water table at site, it is a general assumption in the design of gabion faced walls.
that due to the highly permeable nature of the gabions there is no pore pressure development in the backfill.

5. Effect of cohesion and inclination of facing are neglected, as they always add on to the stability of a structure which ultimately takes the design to a safer side.

6. Surcharge over the backfill is taken as a uniformly distributed load and the intensity of surcharge over the backfill and the retained soil are assumed to be the same.

**9.4.2 Selection of variables**

Design charts were developed for each mode of failure i.e., external and internal modes of failure as per the limit state method explained in Chapter 3. Different geometric parameters have been considered and non dimensional parameters have been used to derive the charts. In the external mode of failure, bearing and tilt as well as sliding modes of failure were considered. In the internal modes of failure, safety against mesh breakage and mesh pullout was taken into account. For the preparation of design charts, equations for limiting conditions were taken and they were converted into functions of non-dimensional parameters, which serves as variables. Graphs were plotted such that the required length and spacing of reinforcement, for a safe and economical design may be obtained directly from the charts.

**9.4.3 External stability analysis**

*9.4.3.1 Bearing and tilt failure*

The limiting condition for bearing and tilt failure was obtained from Eqn. 9.13 by neglecting the effect of embedment, which is small when compared to the height of wall. Then the limiting condition becomes:

\[ q_v = \frac{q_{u}}{1.35} \]

............... [9.28]
Substituting the value of \( q_f \) from Eqn. 9.7 and on simplification, Eqn. 9.28 comes to the form:

\[
1.0125 (W_g + W_s + W_q)^2 = q_{ult} (W_g b/2 + W_s (b + L/2) + W_q (b + L/2) - P_s H/3 - P_q H/2)
\]

.................... (9.29)

Considering the assumptions cited in Section 9.4.1 and substituting the values from Eqns. 9.1 to 9.5, Eqn. 9.29 may be converted to dimensionless form as:

\[
1.0125 \left\{ \frac{(b/H)^2 + (L/H)^2 + (q/\gamma_s H)^2 (L/H)^2 + 2 (b/H) (L/H) + 2 (q/\gamma_s H) (L/H)^2}{2 (b/H) (L/H) \left[ \frac{1}{2} (b/H) + (b/H) (L/H) + \frac{1}{2} (L/H)^2 \right] + 2 (q/\gamma_s H) + 1/2 (q/\gamma_s H) (L/H)^2 - K_s/6 - K_q/2 (q/\gamma_s H) } \right\} = 0
\]

.................... (9.30)

Putting \( q/\gamma_s H \) as surcharge factor, SF, \( q_{ult}/\gamma_s H \) as bearing capacity factor, BCF and \( b/H \) as facing width factor, FWF and then rewriting in terms of length factor, LF = L/H, Eqn. 9.30 reduces to a quadratic form as:

\[
(LF)^2 \left\{ \left( 1 + SF \right) \left[ 1.0125 (1 + SF) - BCF/2 \right] \right\} + (LF) \left\{ FWF (1 + SF) \left[ 2.025 - BCF \right] \right\} + \left\{ FWF^2 \left[ 1.0125 - BCF/2 \right] + (K_s/6) BCF (1 + 3SF) \right\} = 0
\]

.................... (9.31)

Solving the quadratic equation, the length of reinforcement which is an indication of the base width of the reinforced structure, may be obtained. This gives a safe design value against bearing and tilt failure. From Eqn. 9.31, it is clear that LF depends on the non dimensional factors \( \phi \) (in terms of \( K_s \)), SF, FWF and BCF.

\[
LF = f(\phi, SF, FWF, BCF)
\]

.................... (9.32)

Using the relation from Eqn. 9.31, graph can be plotted after obtaining different LF values by varying \( \phi \), SF and FWF for BCF = 5 and is shown in Fig. 9.4. Similarly, LF values were obtained for other values of BCF like 7.5, 10, 15, 20, 25, 30, 40 and 50.
Fig. 9.4 Design chart from bearing and tilt considerations for BCF = 5
Fig. 9.5 Design chart from bearing and tilt considerations
- Interpolation chart
Thereafter, charts (Fig. 9.5) were obtained with \((BCF)_{eq} / BCF = 5\) as X axis and \((LF)_{eq} / LF\) for \(BCF = 5\) as Y axis for different values of SF. The FWF values were averaged for the preparation of these graphs as the numerical values did not show much difference and hence these charts can be used for any value of FWF. Thus, for any BCF value other than 5, corresponding LF values may be obtained from the interpolation chart shown in Fig. 9.5. The lowest value of BCF was fixed as 5 because below this, LF values obtained are either greater than unity or discriminant of the quadratic equation (Eqn. 9.31) becomes negative which yields imaginary roots.

9.4.3.2 Sliding failure

In the case of sliding failure, the limiting condition was obtained from Eqn. 9.14 as:

\[
1.3 \, R_h = R_v \, a \, \tan \phi + \frac{a \, c_{t2} \, L}{1.6} \quad \text{........................ (9.33)}
\]

Taking into account the assumption nos. 1 and 5 cited in Section 9.4.1, Eqn. 9.33 reduces to:

\[
1.3 \, R_h = (2/3) \, R_v \, \tan \phi \quad \text{........................ (9.34)}
\]

The soil – reinforcement interaction coefficient, a, is usually taken as 2/3 as explained in Section 9.2.2.2. On substitution from Eqns. 9.15 and 9.16, Eqn. 9.34 comes to the form:

\[
1.95 \, (P_s + P_d) = 0.667 \, (W_s + W_d) \, \tan \phi \quad \text{........................ (9.35)}
\]

Substituting the values from Eqns. 9.1 to 9.5 and assuming the unit weights to be same for the gabion fill material and backfill, Eqn. 9.35 can be converted to dimensionless form as:

\[
2.925 \, K_s \, \{ 1.2 + \, (q/\gamma_s H) \} / \tan \phi = (b/H) + (L/H) \quad \text{........................ (9.36)}
\]
Rewriting in terms of length factor, \( LF = L/H \), Eqn. 9.36 reduces to the form:

\[
LF = 1.4625 K_s (1 + 2SF) / \tan \phi - FWF \\
\text{................. (9.37)}
\]

Solving the equation, the length of reinforcement which is an indication of the base width of the reinforced structure, can be obtained. This gives a safe design value against sliding failure. From Eqn. 9.37, it is clear that \( LF \) depends on the non dimensional factors \( \phi \), SF and FWF.

\[
LF = f(\phi, SF, FWF) \\
\text{................. (9.38)}
\]

Using the relation from Eqn. 9.37, graph was plotted after obtaining different LF values by varying \( \phi \), SF and FWF and is shown in Fig. 9.6.

9.4.4 Internal stability analysis

9.4.4.1 Failure due to mesh breakage

The limiting condition for failure due to mesh breakage was obtained from Eqn. 9.20 as:

\[
T_i = \frac{T_j}{1.1} \\
\text{................. (9.39)}
\]

Substituting for \( T_j \) from Eqns. 9.17 to 9.19 and taking \( L \approx 0.5H \) (in order to simplify the final equation, \( L \) is given an average value recommended by the parametric studies), Eqn. 9.39 comes to the form:

\[
0.75 \left( \gamma_s h_i + q \right) H^2 T_D - K_s \left( \gamma_s h_i + 3q \right) h_i^2 T_D = 1.2375 K_s \left( \gamma_s h_i + q \right)^2 H^2 S
\]

\text{................. (9.40)}

Dividing through out by \( \gamma_s H^2 \), Eqn. 9.40 can be converted to dimensionless form as:

\[
0.75 \left( \frac{h_i}{H} \right) + \left( \frac{q}{\gamma_s H} \right) \left( \frac{T_D}{\gamma_s H^2} \right) - K_s \left( \frac{h_i}{H} \right) \left( \frac{3q}{\gamma_s H} \right) \left( \frac{h_i}{H} \right)^2 \left( \frac{T_D}{\gamma_s H^2} \right)
\]

\[
= 1.2375 K_s \left( \frac{h_i}{H} \right) \left( \frac{q}{\gamma_s H} \right) \left( \frac{T_D}{\gamma_s H^2} \right) \left( \frac{S}{H} \right)
\]

\text{................. (9.41)}
Fig. 9.6 Design chart from sliding considerations
Rewriting in terms of the required vertical spacing factor, \( VSF = S_u/H \), Eqn. 9.41 reduces to the form:

\[
VSF = \frac{RLF \left[ 0.75(DF + SF) - K_d (DF + 3SF)(DF)^2 \right]}{1.2375K_u (DF + SF)^2} 
\]

where, \( RLF = \) reinforcement load factor = \( T_n / \gamma_s H^2 \) and \( DF = \) depth factor = \( h_i / H \). Solving Eqn. 9.42, the spacing of reinforcement may be obtained which gives a safe design value against mesh breakage. From Eqn. 9.42, it is clear that \( VSF \) depends on the non dimensional factors \( \phi, SF, RLF \) and \( DF \).

\[
VSF = f (\phi, SF, RLF, DF) 
\]

Using the relation from Eqn. 9.42, graphs were plotted after obtaining different \( VSF \) values by varying \( \phi, SF \) for \( RLF = 0.1 \) at different depth factors, \( DF = 0.25, 0.50, 0.75 \) and 1.00 and is shown in Fig. 9.7. \( DF \), the depth factor indicates the level (measured from top) at which the spacing is to be determined. For convenience in the presentation of charts, the entire wall depth is divided into four portions along the depth of the wall. \( DF = 0.25 \) represents the first portion where \( 0 < h_i / H \leq H/4 \). Similarly, \( SF = 0.5, 0.75 \) and 1.00 represents the second, third and fourth portions (where \( H/4 < h_i / H \leq H/2 \), \( H/2 < h_i / H \leq 3H/4 \) and \( 3H/4 < h_i / H \leq H \) respectively). For the values of \( RLF \neq 0.1 \), \( VSF \) values may be taken initially from Fig. 9.7 and then modified with values from Fig. 9.8 which represents an interpolation chart.

9.4.4.2 Failure due to mesh pullout

In the case of failure by loss of adherence of the mesh with the surrounding soil, the limiting condition for failure was obtained from Eqn. 9.23 as:

\[
2L_{\gamma j} = \frac{T_j}{(2/3) \tan \phi_{\alpha}} \gamma_{\alpha} H_j \alpha c + \frac{1.3 \times 1.1}{1.6 \times 1.3 \times 1.1_w} 
\]

202
Fig. 9.7 Design chart from mesh breakage considerations
Fig. 9.8 Interpolation chart from mesh breakage considerations

The assumptions taken into account at this point are assumption nos. 1 and 4 cited in Section 9.4.1. \( T_i \) may be substituted from Eqns. 9.17 and 9.24 where, \( L \) may be approximated as 0.5 \( H \) for simplicity in calculations (as explained in Section 9.4.4.1). Thus, Eqn. 9.44 reduces to the following form as:

\[
0.75 \frac{L_{ei}}{\gamma_s h_i H^2} \tan \phi - L_{ri} K_a \gamma_s h_i^3 \tan \phi = 0.53625 K_a \gamma_s h_i H^2 S_{ri} \\
....................... \text{(9.45)}
\]

Dividing through out by \( \gamma_s h_i H^4 \), Eqn. 9.45 may be converted to dimensionless form as:

\[
0.75 \left( \frac{L_{ri}}{H} \right) \tan \phi - K_a \left( \frac{h_i}{H} \right)^2 \left( \frac{L_{ei}}{H} \right) \tan \phi = 0.53625 K_a \left( \frac{S_{ri}}{H} \right) \\
....................... \text{(9.46)}
\]
Rewriting in terms of the required embedment length factor, \( L_{ej}/H \), Eqn. 9.46 reduces to the form:

\[
\frac{L_{ej}}{H} = \frac{1}{\tan \phi} \left\{ \frac{0.53625 K_a S_{vj}}{H} \left( \frac{h_j}{H} \right)^2 \right\} \tan \left( \frac{45^\circ - \phi}{2} \right) \tag{9.47}
\]

The length of reinforcement required in the active zone is shown by Eqn. 9.26. It can be rewritten in dimensionless form as:

\[
\frac{L_{ej}}{H} = (1 - h_j/H) \tan (45^\circ - \phi/2) \tag{9.48}
\]

The length factor, \( LF = L/H \) may be expressed from Eqn. 9.27 as:

\[
LF = L_{ej} / H + L_{adj} / H \tag{9.49}
\]

Thus the equation for \( LF \) may be obtained by summing up Eqn. 9.47 and Eqn. 9.48, which is rewritten as:

\[
LF = \frac{1}{\tan \phi} \left\{ \frac{0.53625 K_a VSF}{0.75 - K_a Df^2} \right\} + \{(1 - DF) \tan (45 - \phi/2) \right\} \tag{9.50}
\]

Solving Eqn. 9.50, the safe length of reinforcement needed to provide internal stability to the structure may be obtained. From Eqn. 9.50, it is clear that \( LF \) depends on the non dimensional factors \( \phi \), VSF and DF.

\[
LF = f(\phi, VSF, DF) \tag{9.51}
\]

Using the relation from Eqn. 9.50, graph was plotted after obtaining different \( LF \) values by varying \( \phi \), VSF and DF and is shown in Fig. 9.9.
Fig. 9.9 Design chart from mesh pullout considerations
9.4.5 Serviceability limit state

The condition for serviceability limit state is shown by Eqn. 3.15. The equation may be substituted for $T_{aw}$ from Eqns. 9.17 and 9.25 and may be rewritten as (taking $L = 0.5H$ as explained in Section 9.4.4.1):

$$
\varepsilon_i E_A (0.75 H^2 - K_a h_j^2) = 0.375 K_a \gamma_s h_j S_v H^3 \\
\text{.................. (9.52)}
$$

Dividing Eqn. 9.52 through out by $\gamma_s H^5$, it may be converted to non-dimensional form as:

$$
\varepsilon_i (E_A / \gamma_s H^3) [0.75 - K_a (h_j/H)^2] = 0.375 K_a (h_j/H) (S_v/H) \\
\text{.................. (9.53)}
$$

Eqn. 9.53 may be rewritten in terms of the strain in the $j$th layer of reinforcement, $\varepsilon_j$, as:

$$
\varepsilon_j = \frac{0.375 K_a}{RSF} \left( \frac{DF \times VSF}{0.75 - K_a DF^2} \right) \\
\text{.................. (9.54)}
$$

where, $RSF = \text{reinforcement stiffness factor} = E_A / \gamma_s H^3$ and it is seen that $\varepsilon_j$ depends on the non-dimensional factors $\phi$, $VSF$, $RSF$ and $DF$.

$$
\varepsilon_j = f(\phi, VSF, RSF, DF) \\
\text{.................. (9.55)}
$$

Using the relation from Eqn. 9.54, graph was plotted to get different $\varepsilon_j$ values by varying $\phi$ for $VSF = 0.1$ and $RSF = 0.5$ as shown in Fig. 9.10. Interpolation chart (Fig. 9.11) was also prepared to get the strain values corresponding to $VSF$ values ranging from 0.05 to 0.5 and $RSF$ values ranging from 0.5 to 5.
Fig. 9.10 Design chart from serviceability limit state considerations for RSF = 0.5 and VSF = 0.1.

Fig. 9.11 Interpolation chart from serviceability limit state considerations.
9.5 DESIGN CHARTS BASED ON DEFORMATION CRITERIA

Parametric studies were conducted to understand the behaviour of gabion faced reinforced earth walls by varying some of the salient geometric and material properties and were described in Chapters 7 and 8 respectively. From the results of the parametric studies, some design charts were developed which could be supplemented with the design charts developed in this chapter. The design charts developed from the parametric studies may be included under the serviceability limit state as they were developed based on the deformation criteria. The length of reinforcement, the facing width and the type of filling material inside gabions can be checked based on limiting deformation from these charts. Fig. 9.12 shows the design chart from which the geometric parameters $L/H$ and $b/H$ can be selected for a particular deformation value and for a particular vertical spacing factor ($h/H$).

![Design chart for fixing geometric parameters (H = 6m)](image)

**Fig. 9.12** Design chart for fixing geometric parameters ($H = 6m$)

The method of using the design chart is as follows. For a known value of allowable displacement, moving vertically upward, for a particular $h/H$ ratio, the length of reinforcement can be fixed by moving left and then moving
rightwards from the same point, the required width of gabion facing can be chosen. When there are more than one value of $h / H$ for a given allowable displacement, the maximum value should be taken and the corresponding $L / H$ and $b / H$ should be chosen for the design.

The results from the material parametric studies may also be used as design charts for selecting a suitable soil type and rock type as the backfill and gabion fill respectively for a particular deformation after checking the strains developed in the reinforcement and the average safety factor. The results are presented in design chart form from Figs. 9.13 to 9.16 for various types of backfills.

In order to use the design charts for material parameters, depending on the type of backfill, one of the four design charts may be selected. After selecting the design chart, for the permissible deformation, the suitable type of backfill based on the geotechnical properties may be fixed, moving rightwards. Then moving downwards, the reinforcement strain and the average safety factor may be fixed. If they are within the permissible limits, the type of gabion fill may be chosen, moving downwards. If the reinforcement strain and average safety factor values exceed the permissible limits, a better gabion fill may be chosen moving further rightwards, along the curve for the same backfill.

Again, these values are for a wall height of 6m. For any other wall height, the corresponding values may be interpolated from Fig. 9.17 where the response parameter ratio indicates the ratio of the response parameter for the required height to the corresponding response parameter for $H = 6m$. The response parameter may be the parameters like $L / H$, $b / H$, $u / H$, $e_{th}$ or ASF.
Fig. 9.13 Design chart for fixing backfill – gabion fill combination
(Gravel and sand backfill, H = 6m)
Fig. 9.14 Design chart for fixing backfill – gabion fill combination
(Silty sand backfill, H = 6m)
Fig. 9.15 Design chart for fixing backfill - gabion fill combination (Clayey sand backfill, H = 6m)
Fig. 9.16 Design chart for fixing backfill – gabion fill combination (Clay with low plasticity backfill, H = 6m)
Fig. 9.17 Interpolation chart for selection of geometric and material parameters for any wall height

Fig. 9.18 Design chart for fixing strip loading position

If a strip load like pipeline, edge of bridge seating, crane, compactor etc., has to be provided over the wall, its position may be fixed using the design
The chart may be used as follows. In places where deformation has to be limited for practical reasons, for the limiting deformation and for the desired strip width, the strip position may be directly selected from the chart by moving upwards from the corresponding deformation value in the X axis. When the curve for desired strip width is intercepted, the position may be chosen by moving leftwards. But the designer should keep in mind the critical planes where the positioning of strip loading should always be avoided. In cases, where two options of strip positions are available, the position away from the reinforced area \((a_r/L > 1)\) is safer and hence should be adopted. But if there are any space restrictions, the designer may go in for the position within the reinforced area.

The charts presented are just quick guidelines for material and geometry selection for general conditions. The final decision should depend upon the discretion and logic of the design engineer at site.

### 9.6 DESIGN STEPS USING DESIGN CHARTS

For an economical and speedy design of a gabion faced reinforced earth wall, the following simple procedure may be adopted.

1. Calculate the different dimensionless factors required for the analysis like \(SF = q/\gamma H\), \(FWF = b/H\), \(BCF = q_{ult}/\gamma H\), \(RLF = T_{ij}/\gamma H^2\) and \(RSF = EA/\gamma H^3\).

2. If \(BCF = 5\), corresponding to \(\phi\), \(SF\) and \(FWF\), get length factor \(LF\) from Fig. 9.4. In the case of other values of \(BCF\), the length factor may be obtained from the interpolation chart shown in Fig. 9.5. This gives a value for length factor providing safety against bearing and tilt considerations.

3. The safe value for length factor against sliding failure may be obtained from Fig. 9.6 corresponding to \(\phi\), \(SF\) and \(FWF\).

4. The spacing of reinforcement at salient portions, \(0 - H/4\), \(H/4 - H/2\), \(H/2 - 3H/4\) and \(3H/4 - H\) represented by \(DF = 0.25, 0.5, 0.75\) and \(1.00\) respectively may be obtained from Fig. 9.7 as VSF values considering the
mesh rupture criteria for RLF = 0.1. For other values of RLF, the VSF values obtained from Fig. 9.7 may be modified using the interpolation chart shown in Fig. 9.8.

5. Corresponding to the DF and VSF values obtained from Step 4, the length factor value providing adequate safety against mesh pullout failure may be obtained from Fig. 9.9.

6. The length factor required for an economic and safe design may be taken as the maximum value obtained from steps 2, 3 and 5.

7. To ensure the safety of the structure against serviceability limit state considerations, the strain developing in the mesh may be obtained from Fig. 9.10 for RSF = 0.5 and VSF = 0.1. For all the other values of RSF and VSF, the strain values may be obtained from the semilog plot in Fig. 9.11. The strain values thus obtained should not exceed the permissible maximum elongation of the gabion mesh which is 10%.

8. Based on deformation criteria, the length factor may be checked from Figs. 9.12 and 9.17. Figs. 9.13 to 9.17 may be used to suggest a suitable gabion fill - backfill combination. Position of strip loading may be fixed using Fig. 9.18.

In all the cases, in the steps described above, intermediate values may be obtained by interpolation. Thus, assuming the appropriate initial dimensions of the gabion faced walls, length and spacing of reinforcement required for an economical design of gabion faced reinforced earth walls may be arrived at easily by following the eight simple steps mentioned above.

9.7 DESIGN EXAMPLE USING DESIGN CHARTS

The design of the wall illustrated in Section 9.3 may be simplified using the above mentioned procedure as follows. The corresponding design charts (mentioned in Section 9.6) required for the design are reproduced for the purpose of illustration.
Step 1: The dimensionless factors required for the problem may be calculated as: SF = 0.1, FWF = 0.1, BCF = 6.5, RLF = 0.068 ≈ 0.1 (taking $T_0 = T_{ult}/1.5$) and RSF = 3.2.

Step 2: Bearing considerations: For $\phi = 35^\circ$, SF = 0, FWF = 0.1, LF = 0.29 (from Fig. 9.19 (a)). For $\phi = 35^\circ$, SF = 0.5, FWF = 0.1, LF = 0.48 (from Fig. 9.19 (a)). By interpolation, for $\phi = 35^\circ$, SF = 0.1, FWF = 0.1, LF = 0.336. This is for BCF = 5. But, BCF$_{req}$ = 6.5. The data for this has to be obtained from Fig. 9.19 (b).

The value of BCF$_{req}$ / (BCF = 5) is 1.3. Corresponding to this, for SF = 0.0, the value of LF$_{req}$ / (LF for BCF = 5) is 0.9 and for SF = 0.5, it is 0.85. The corresponding value for SF = 0.1 may be interpolated as 0.89. Therefore, LF for BCF = 6.5 is 0.336 x 0.89 = 0.3.

Step 3: Sliding considerations: For $\phi = 35^\circ$, FWF = 0.1, SF = 0, LF = 0.46 and the corresponding value of LF for SF = 0.5 is 1 (Fig. 9.20). By interpolation, for SF = 0.1, LF = 0.57.

Step 4: Mesh rupture considerations: For this problem, RLF = 0.1 and hence to determine the spacing of reinforcement, Fig. 9.21 may be made use of. Spacing is determined at the four salient depths 0.25H, 0.50H, 0.75H and 1.00H measured from the top.

Table 9.5 Spacing values at salient depths

<table>
<thead>
<tr>
<th>DF</th>
<th>SF = 0</th>
<th>SF = 1</th>
<th>SF = 0.1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Fig. 9.7)</td>
<td>(Fig. 9.7)</td>
<td>(By interpolation)</td>
</tr>
<tr>
<td>0.25</td>
<td>0.87</td>
<td>0.28</td>
<td>0.75</td>
</tr>
<tr>
<td>0.50</td>
<td>0.4</td>
<td>0.18</td>
<td>0.36</td>
</tr>
<tr>
<td>0.75</td>
<td>0.24</td>
<td>0.12</td>
<td>0.22</td>
</tr>
<tr>
<td>1.00</td>
<td>0.14</td>
<td>0.06</td>
<td>0.12</td>
</tr>
</tbody>
</table>
Fig. 9.19 (a) Illustration of design example: Step 2

Fig. 9.19 (b) Illustration of design example: Step 2

Fig. 9.20 Illustration of design example: Step 3
Fig. 9.21 Illustration of design example: Step 4
From Table 9.5, it is seen that vertical spacing factor varies with depth and the design engineer can choose appropriate spacing from this table varying with depth. However, for comparison purpose with the design example in Section 9.3, uniform spacing of reinforcement throughout the depth is chosen here also. For this, minimum value of VSF (0.12) obtained from Table 9.5 is adopted. Therefore, minimum spacing of reinforcement required = 0.12H = 0.6m. Gabion boxes are available in standard dimensions as multiples of 0.5m (Chapter 3). Accordingly, uniform spacing of 0.5m may be adopted throughout the depth i.e., VSF = 0.1.

Step 5: The length of reinforcement corresponding to pullout considerations may be obtained from Fig. 9.22. For $\phi = 35^\circ$ and VSF = 0.1, corresponding to DF = 0.25, 0.5, 0.75 and 1, the LF values are 0.42, 0.3, 0.16 and 0.04 respectively. The maximum value of LF = 0.42 may be adopted throughout for comparison with the design example in Section 9.3. Practically, the lengths may be reduced towards the bottom as per the values obtained here, but after considering the external stability conditions.

Step 6: The length factor may be fixed as the maximum value obtained from steps 2, 3 and 5. Therefore LF = 0.57 and the length of reinforcement required for a safe and economical design is 2.85. A uniform length of 3m may be adopted throughout.

Step 7: Serviceability limit considerations: For standard values, RSF = 0.5 and VSF = 0.1, as well as for $\phi = 35^\circ$, the maximum strain in the mesh may be obtained from Fig. 9.23 (a) as 4.25% for DF = 1. For RSF = 3.2, strain ratio = 0.15 (Fig. 9.23 (b)). Therefore, strain developed = 0.48%. Now the strain value has to be modified for VSF value. Corresponding to VSF = 0.1, strain ratio = 1 (Fig. 9.23 (b)). Hence, maximum strain developed in the mesh = 0.48%, which is very much less than 10% and hence the design may be considered as safe.
Fig. 9.22 Illustration of design example: Step 5

Fig. 9.23 (a) Illustration of design example: Step 7

Strain ratio = req. strain / std. strain (log)

Fig. 9.23 (b) Illustration of design example: Step 7
Step 8: Assume that the maximum deformation value may be limited to 0.005H. For VSF = 0.1 (curve for VSF = 0.08 is chosen as it is the closest value to 0.1), FWF = 0.3 and LF = 1 (Fig. 9.24 (a)). For H = 5m, RPR = 0.86 from Fig. 9.24 (c). Therefore, FWF for H = 5m is 0.86 x 0.3 = 0.258 and LF = 0.86 x 1 = 0.86. These values are greater than the provided values, hence the design may be considered as safe against limiting deformation.

The backfill used for design has a cohesion value = 0 kPa. So the material will be either sand, gravel or silty sand. $\gamma = 20\text{kN/m}^3$, $\phi = 35^\circ$. Backfill material from the design charts with similar properties is SM3. In Fig. 9.24 (b), for $u_{\text{max}}/H = 0.005$, moving rightwards and meeting the curve for SM3 and then moving downwards straight, maximum strain developed in the reinforcement is 0.3% and average safety factor is 8.25. RPR for $u_{\text{max}}/H$ and $e_{\text{rt}}$ For H = 5m, $e = 0.3\% \times 0.86 = 0.258\%$. RPR for ASF = 1.2. Therefore, for H = 5m, ASF = 1.2 x 8.25 = 9.9. Regarding the material of gabion fill, it is seen that sedimentary rocks produce a deformation of 0.005H. But it is better to avoid the use of sedimentary rocks, which may disintegrate in course of time and go in for any locally available type metamorphic or igneous type rocks.

![Fig. 9.24 (a) Illustration of design example: Step 8](image-url)
On comparing with the detailed design in Section 9.3, it can be seen that this modified procedure using design charts can be adopted very safely for
design purposes. It is expected that use of the design charts will ease the design engineers' workload and also expedite the design process.

9.8 SUMMARY

Conventional design method of gabion faced reinforced earth walls become tiresome when design using assumed data, go to the unsafe side and repeated calculations become essential. Limit state method is the method of design followed in practice nowadays which incorporates the limit state of collapse and serviceability limit state. Design charts were prepared for determining the dimensions of gabion faced reinforced earth walls by conducting non dimensional analysis of the limiting conditions of failure. Additional design charts were incorporated based on the results of parametric studies conducted in the previous chapters. These design charts may be used as hands – on – tool for the design engineers at site.