Chapter 3

GABION FACED RETAINING WALLS

3.1 GENERAL

Gabions are rectangular baskets fabricated from a hexagonal mesh of heavily galvanized steel wire. The baskets are filled with rock and stacked atop one another to form a retaining wall. The functioning of gabion walls depend mainly on the interlocking of the individual stones and rocks within the wire mesh and their mass or weight. Gabions are porous type of structures that can sometimes be vegetated. Gabions are considered to be a “hard” structural solution that has minimal habitat and aesthetic value.

Wire mesh gabions have been used in Civil Engineering projects for many years and their ability to perform well in a variety of applications have earned them the respect of Civil Engineers throughout the world. Gabions are highly cost effective construction materials which are easy to install and maintain. With environmental issues now of more concern than in the past, gabions offer a more natural solution to previously designed concrete walls.

In spite of all these, standard literature available on this practice is limited. Hence, in this chapter, a detailed description of the construction method, which is usually adopted by the field practitioners and manufacturers, is given. Along with that, the limit state method of analysis and design for reinforced soil retaining walls (as per BS 8006: 1995) which can be suitably adopted for the design of gabion faced reinforced soil retaining walls is also described herein.

3.2 CONSTRUCTION

3.2.1 Wire mesh

Double twisted wire meshes made by mechanically twisting continuous pairs of wires (2.5 – 3.5 mm dia.) and interconnecting them with adjacent wires
to form hexagonal shapes (Figs. 3.1 and 3.2) are used to make gabion boxes of various sizes. Materials used for the mesh shall be mild steel having a tensile strength of 350 MPa – 500 MPa and a minimum elongation of 10% at breaking load performed on a gauge length of 250 mm as per BS 1052: 1980. These wires shall be provided with coating of zinc and an additional coating of PVC (Fig. 3.3).

Fig. 3.1 Hexagonal wire mesh

Fig. 3.2 Double twisted wire

Fig. 3.3 Cross section of gabion wire

In the use of gabions the following specifications should be considered as listed in BS 8002: 1994. Hexagonal woven mesh gabions should be made from wire galvanized according to BS 443: 1982. For welded mesh gabions, the panels of mesh which form the cages should be hot dip galvanized after welding according to BS 729: 1995 (this code has been recently replaced by BS EN ISO 3834 – 3: 2005). In the case of PVC coated gabion mesh, the PVC coating should conform to BS 4102: 1998. The radial thickness of the coating applied to the galvanized wire core should be a minimum of 0.25 mm. The PVC should be sufficiently bonded to the galvanized wire core to prevent a
capillary flow of water between the wire and the PVC coating leading to corrosion.

The filler material shall be naturally occurring hard stones which are weather resistant, insoluble and of minimum size 1 to 2 times the dimension of the mesh. Stones with high specific gravity are preferable since gravity behaviour of the structure is predominant. The mesh dimensions for PVC coated galvanized gabion boxes are 80 x 100 mm and 100 x 120 mm. For galvanized boxes standard mesh sizes are 60 x 80 mm, 80 x 100 mm, 100 x 120 mm. The wire mesh provides an increased strength to the walls due to the friction on the surface of the wire and the mechanical interlocking properties of the backfill (Bergado et al., 2001).

### 3.2.2 Gabions

Gabion boxes are uniformly partitioned into internal cells using diaphragm walls, (Fig. 3.4) interconnected with similar units, and filled with stones at the project site to form flexible, permeable and monolithic structures such as retaining walls, sea walls, channel linings, revetments, facing elements for reinforced soil structures and weirs for erosion control purposes.

![Fig. 3.4 A typical gabion](image)

![Fig. 3.5 Gabion with extension](image)

The gabions are manufactured in factories in sizes of 1.5 m x 1 m x 1m, 2 m x 1 m x 1 m, 4 m x 1 m x 1 m etc.. Individual empty units are connected
The edges are then laced together by single and double twist lacing wires at 100 - 150 mm spacing. The first layer of gabion is seated on levelled flat surface and continuously secured together either by lacing or by tying the edges using fasteners. The end gabion is partly filled with suitable stones to form end anchor and there after bracing wires are fixed at 0.5 m spacing to avoid bulging of front side of gabion.

![Diagram of gabion configurations](image)

**Fig. 3.6 Typical configurations of gabion faced walls**

(Courtesy: Meccaferrì)

Gabions are filled in layers of one-third, ensuring at each stage a good compaction by hand and tensioning of the gabion mesh. Overfilling by 50 - 75 mm is also made to allow for settlement of the infill. The mesh lid is then folded flat, stretched and diaphragms are tied up. The rows of gabions are filled up subsequently in a sequential manner after ensuring that each gabion is properly fixed to adjacent gabions on all sides. A geofilter is positioned at the back of the gabion between the box and backfill to prevent the entry of soil.
particles from backfill into void spaces of stones. After this, backfilling is commenced. Backfill is laid in lifts of approximately 200mm and properly compacted up to 95% of proctor density. The top of the backfill is kept in level with the top of the gabion layer. After the completing the full length of a layer, the next unit is placed on top of this and the procedure is continued up to the required level. In the case of reinforced soil type gabion faced retaining walls, gabion boxes with basal extensions are used for the construction. The construction sequence is clearly illustrated in Fig. 3.8.

![Fig. 3.7 Gravity type gabion wall - construction procedure](image)

The first structure of this kind on record is a combination of gabions and mechanically reinforced soil which was built in Sabah, Malaysia in 1979. A vertical skin of gabions was anchored to the backfill using metal strips. The 14 m high structure supports a stretch of the road from Kota Kinabalu to Sinsuran in Malaysia.

Foundation requirements, which must be established by the engineer, will vary with site conditions, height of gabion structure, etc. Generally, the
top layer of soil is stripped until a layer of the required bearing soil strength is reached. In some cases, the foundation may consist of suitable fill material compacted to a minimum of 95 percent of proctor density.

![Diagram of MSE type gabion faced wall construction sequence]

**Fig. 3.8 MSE type gabion faced wall – construction sequence**

(Courtesy: Meccaferrri)

### 3.3 ADVANTAGES

The gabion structures stand out as a simple, efficient and economical solution to various civil engineering construction problems due to the following advantages.

- Monolithicity: The various elements in a gabion faced wall are linked through continuous fastening which ensures structural continuity (Fig. 3.9). This allows regular distribution of the imposed forces and ensures that the whole weight of a structure is equal to the sum of the
Fig. 3.9 Monolithic wall

Fig. 3.10 Flexible wall

Fig. 3.11 Permeable eco-friendly wall

(Courtesy: Meccalferr)
Easy to repair any damaged boxes with minimum expense.

Cost effective and suitable in all types of soil conditions.

Work is simple and fast to execute.

No need of shuttering and curing.

Work is not affected by water shortage and on the other hand it is also not affected due to rains during monsoon.

Cost savings is of the order ranging from 30% to 50%.

Ecofriendly (Fig. 3.11).

Reduces sound pollution by absorbing sounds up to 18-28 db.

Absorbs large vibrations and hence widely used near railway tracks.

Despite the fallacy that gabion structures are temporary works the reality is far different. Dry walls (stone walls) prove that gabion works may last for hundreds of years even if the wire netting rusts over a period of time. The double twist, in case of a break in any single wire, prevents the unavelling of the mesh and the movement of stones out of the gabion. Heavy zinc coating of wires assures that eventual deterioration of the netting by rusting is very slow under normal conditions. Where corrosion is a more severe problem, it is possible to considerably extend the wire life by making use of PVC coating. With the passage of time, gabion structures provide natural balances with the environment.

3.4 ANALYSIS OF GABION FACED REINFORCED EARTH WALLS

Any analysis incorporates the field of mechanics along with the failure theories. To perform an accurate analysis, an engineer must gather information such as structural loads, geometry, support conditions, and material properties of the structure to be analysed after selecting trial dimensions of the structure. The results of the analysis typically include support reactions, stresses and displacements. This information is then compared to criteria that indicate the conditions of failure.
Analysis of gabion faced reinforced earth walls, which is in question for the present study, combines the analysis of reinforced soil walls along with the self weight characteristics of gravity type wall. In the case of reinforced soil walls, the code of practice followed is BS 8006: 1995, the details of which are summarised in this section. The BS code follows the limit state method of design, the principles of which are adopted in the reinforced soil wall design, are explained here.

3.4.1 Limit state method

The two limit states considered in the design are ultimate limit state and serviceability limit state. Ultimate limit state is associated with collapse. This state is attained for a specific mode of failure when disturbing forces equal or exceed the restoring forces. Margins of safety against attaining the limit state of collapse, are provided by the use of partial material factors and partial load factors. These partial factors assume prescribed numerical values of unity or greater. Disturbing forces are increased by multiplying by prescribed load factors to produce design loads. Restoring forces are decreased by dividing them by prescribed material factors to produce design strengths. When the design strength equals or exceeds the design load, there deems to be an adequate margin of safety against attaining the ultimate limit state of collapse. Serviceability limit state is attained if the magnitudes of deformation occurring within the design life exceed prescribed limits or if the serviceability of the structure is otherwise impaired.

3.4.2 Partial factors

Limit state method for reinforced soil employs partial safety factors namely load factors, material factors and soil – interaction factors. Prescribed ranges of these values are given in BS 8006 : 1995 (Tables 3.1 and 3.2) to take into account the type of structure, the mode of loading and the selected design life. Partial factors are applied in a consistent manner to minimise the risk of attaining a limit state.
Table 3.1: Partial load factors for load combinations (BS 8006: 1995)

<table>
<thead>
<tr>
<th>Effects</th>
<th>Combinations</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of the reinforced soil body</td>
<td>$f_{ls} = 1.5$</td>
<td>$f_{ls} = 1.0$</td>
<td>$f_{ls} = 1.0$</td>
</tr>
<tr>
<td>Mass of the backfill on top of the reinforced soil wall</td>
<td>$f_{ls} = 1.5$</td>
<td>$f_{ls} = 1.0$</td>
<td>$f_{ls} = 1.0$</td>
</tr>
<tr>
<td>Earth pressure behind the structure</td>
<td>$f_{ls} = 1.5$</td>
<td>$f_{ls} = 1.5$</td>
<td>$f_{ls} = 1.0$</td>
</tr>
<tr>
<td>Traffic load:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>On the reinforced soil block</td>
<td>$f_{q} = 1.5$</td>
<td>$f_{q} = 0.0$</td>
<td>$f_{q} = 0.0$</td>
</tr>
<tr>
<td>Behind the reinforced soil block</td>
<td>$f_{q} = 1.5$</td>
<td>$f_{q} = 1.5$</td>
<td>$f_{q} = 0.0$</td>
</tr>
</tbody>
</table>

The following descriptions of load cases identify the usual worst combination for the various criteria. All load combinations should be checked for each layer of reinforcement within each structure to ensure that the most critical condition has been found and considered.

Combination A considers the maximum values of all loads and therefore normally generates the maximum reinforcement tension and foundation bearing pressure. It may also determine the reinforcement requirement to satisfy pullout resistance although pullout resistance is usually governed by combination B.

Combination B considers the maximum overturning loads together with minimum self mass of the structure and superimposed traffic load. This combination normally dictates the reinforcement requirement for pullout resistance and is normally the worst case for sliding along the base.

Combination C considers dead loads only without partial load factors. This combination is used to determine foundation settlements as well as generating reinforcement tensions for checking the serviceability limit state.

For reinforced soil applications, the ultimate and serviceability limit states should be considered in terms of both internal and external stability. The assessment of external stability involves consideration of the stability of the reinforced soil mass. This includes assessment of potential failure modes such as bearing and tilt of the wall as well as forward sliding along the base of the wall. For each failure mode considered, prescribed load and material factors are appropriately applied to external disturbing forces and external restoring forces to ensure that the factored restoring force equals or exceeds the factored disturbing force. The internal stability of a reinforced soil mass is governed by the interaction between the soil and the reinforcement. This interaction occurs by friction or adhesion.
Table 3.2: Summary of partial factors (BS 8006 : 1995)

<table>
<thead>
<tr>
<th>Partial factors</th>
<th>Ultimate limit state</th>
<th>Serviceability limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Load factors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil unit mass e.g. Wall fill</td>
<td>$f_s$ to be taken from Table 3.1</td>
<td></td>
</tr>
<tr>
<td>External dead loads e.g. line or point loads</td>
<td>$f_l$ to be taken from Table 3.1</td>
<td></td>
</tr>
<tr>
<td>External live loads e.g. traffic loading</td>
<td>$f_t$ to be taken from Table 3.1</td>
<td></td>
</tr>
<tr>
<td><strong>Soil material factors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>To be applied to $\tan \phi$</td>
<td>$f_{ms} = 1.0$</td>
<td>$f_{ms} = 1.0$</td>
</tr>
<tr>
<td>To be applied to $c$</td>
<td>$f_{ms} = 1.6$</td>
<td>$f_{ms} = 1.0$</td>
</tr>
<tr>
<td>To be applied to $c_o$</td>
<td>$f_{ms} = 1.0$</td>
<td>$f_{ms} = 1.0$</td>
</tr>
<tr>
<td><strong>Reinforcement material factor</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>To be applied to reinforcement base strength</td>
<td>$f_m$ depends on type of reinforcement and its design life (see section 3.4.3)</td>
<td></td>
</tr>
<tr>
<td><strong>Soil / reinforcement interaction factors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sliding across surface of reinforcement</td>
<td>$f_s = 1.3$</td>
<td>$f_s = 1.0$</td>
</tr>
<tr>
<td>Pull out resistance of reinforcement</td>
<td>$f_p = 1.3$</td>
<td>$f_p = 1.0$</td>
</tr>
<tr>
<td><strong>Partial factors of safety</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation bearing capacity: to be applied to $q_{ult}$</td>
<td>$f_{ms} = 1.35$</td>
<td>NA</td>
</tr>
<tr>
<td>Sliding along base of structure or any horizontal surface where there is soil-to-soil contact</td>
<td>$f_s = 1.2$</td>
<td>NA</td>
</tr>
</tbody>
</table>

### 3.4.3 Partial material factors for reinforcements

The unfactored ultimate tensile strength of the reinforcement, $T_{ult}$, is reduced by the reinforcement material factor, $f_m$, to define the reinforcement design strength such that:

$$T_D = \frac{T_{ult}}{f_m}$$  \hspace{1cm} (3.1) 

The design strength may be governed by the ultimate limit state of collapse or serviceability limit state. For plain or galvanised steel reinforcements subjected to axial tensile loads only, $f_m = 1.5$. 

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For others,

\[ f_m = f_{m1} \times f_{m2} \]

\[ = \{ f_{m11} \times f_{m12} \} \times \{ f_{m21} \times f_{m22} \} \]

\[ = \{ f_{m11} \times f_{m12} \} \times \{ f_{m121} \times f_{m122} \} \times \{ f_{m21} \times f_{m22} \} \]

\[ \text{............... (3.2)} \]

where,

- \( f_{m1} \) = partial material factor related to the intrinsic properties of the material
- \( f_{m2} \) = partial material factor concerned with construction and environmental effects
- \( f_{m11} \) = partial material factor related to the consistency of manufacture of the reinforcement
- \( f_{m12} \) = partial material factor related to the extrapolation of test data dealing with base strength
- \( f_{m21} \) = partial material factor related to the susceptibility of the reinforcement to damage during installation in the soil. For steel metallic reinforcements, \( f_{m21} \) has a value of 1.0 when the minimum steel thickness is greater than or equal to 4 mm. For thinner reinforcements, \( f_{m21} > 1.0 \).
- \( f_{m22} \) = partial material factor related to the environment in which the reinforcement is installed. Reinforcements which utilise a protective layer of coating are more resistant to attack, as the load carrying elements are properly protected. In such cases, \( f_{m22} = 1.0 \).
- \( f_{m111} \) = partial material factor related to the reinforcement manufactured according to standards. For metallic reinforcement, \( f_{m111} = 1.0 \) for minimum specification. For polymeric reinforcement, \( f_{m111} = 1.0 \) for characteristic specification.

\[ f_{m11} = 1 + \frac{1.64 \text{ SD}}{\Gamma - 1.64 \text{ SD}} \]

\[ \text{............... (3.3)} \]

where, \( \Gamma \) is the mean reinforcement base strength and SD is the standard deviation of the reinforcement base strength.
\( f_{m12} \) = partial material factor related to the reinforcement manufactured not according to standards. For metallic reinforcement, \( f_{m12} = 1.0 \) for minimum section size. If the reinforcement base strength for metallic reinforcements is based upon sections other than minimum section size, \( f_{m12} > 1.0 \). For polymeric reinforcement \( f_{m12} = 1.0 \).

\( f_{m121} \) = partial material factor related to assessment of available data on base strength to derive a statistical envelope. \( f_{m121} = 1.0 \) if large quantities of data over a long period of time are available or else \( f_{m121} > 1.0 \).

\( f_{m122} \) = partial material factor related to the extrapolation of the above mentioned statistical envelope over the expected service life of the reinforcement. If extrapolation can be done over one log cycle of time \( f_{m122} = 1.0 \). Otherwise,

\[
\begin{align*}
{f_{m122}} & = \log_{10} \left( \frac{t_d}{t_r} \right) \\
\end{align*}
\]

(3.4)

where, \( t_d \) is the design life of reinforcement and \( t_r \) is the duration over which real time creep tests have been performed.

### 3.4.4 Design loads and design strengths

The magnitudes of disturbing loads, such as those which can be developed by lateral earth pressures, are controlled by many factors including soil strength. In calculating disturbing loads and forces, the shear strength parameters of the soil are used unfactored. The numerical value of the calculated raw disturbing load, defined in terms of total stress, is increased by multiplying by a prescribed load factor (Table 3.1) with a value of unity or greater. The end product of this factoring is the design load.

A fundamental principle of limit state design is that the design strength should be greater than or equal to the design load. In the case of external stability, the design load may be resisted by the forces generated in the soil.
which will be a function of soil shear strength. Their characteristic values are reduced by a material factor of prescribed value, to produce design strength. In the case of internal stability, the design load may be resisted by forces generated in the soil and reinforcement which is reduced by a material factor to produce design strength.

3.4.5 Design procedure

3.4.5.1 Fixing the dimensions

Prior to considering the external stability, the overall geometry of the wall should be selected. If the external and internal stability conditions are not satisfied, dimensions of the structure should be altered from the initial size. The initial length of reinforcement for medium and high wall should not be less than 0.7H (3m minimum) where H is the height of the wall. The toe of the structure should be embedded below the ground level. Embedment is recommended to avoid local failure due to punching in the vicinity of the facing and to avoid the phenomenon of local soil flow similar to piping. For vertical walls, embedment depth \( D_m \) may be fixed as the maximum value obtained among the Eqns. 3.5 and 3.6 taken from BS 8006: 1995.

\[
D_m = \frac{H}{20} \hspace{1cm} (3.5)
\]

\[
\frac{D_m}{q_{\text{all}}} = 1.35 \times 10^{-3} \text{ m}^3 / \text{kN} \hspace{1cm} (3.6)
\]

where, \( q_{\text{all}} \) is the allowable bearing capacity of foundation soil.

3.4.5.2 External stability analysis

This indicates the overall stability of the structure. External stability checks are carried out considering all the external forces. For a safe design, stability checks are made for bearing and tilt failure, forward sliding and slip circle failure as well as for the settlement of the structure.

The lateral earth pressure is usually calculated by the Coulomb equation neglecting cohesion as it adds on to the stability of the structure. Although based on granular material, it is conservative for cohesive material.
If a uniformly distributed surcharge pressure \((q)\) is present on top of the backfill surface, it may be treated as an equivalent layer of soil that creates a uniform pressure over the entire height of the wall.

The wall must be able to withstand the bearing pressure at the bottom. For this, the pressure caused by the vertical component of the resultant at the toe of the wall should not exceed the allowable bearing capacity of the soil. The pressure distribution at the base is assumed based on Meyerhof distribution.

\[
q_r = \frac{R_v}{L - 2e}
\]

where,
- \(q_r\) = factored bearing pressure acting at the base of the wall
- \(e\) = eccentricity of the resultant load \(R_v\) about the centre line of base width \((L)\)
- \(R_v\) = resultant of all factored vertical loads

As per BS 8006: 1995, the imposed bearing pressure, \(q_r\), should be compared with the ultimate bearing capacity of the foundation soil as follows:

\[
q_r \leq \frac{q_{ult}}{f_{ms} + \gamma D_m}
\]

where,
- \(q_{ult}\) = the ultimate bearing capacity of the foundation soil
- \(\gamma\) = the density of the foundation soil
- \(f_{ms}\) = partial material factor applied to \(q_{ult}\) to be taken from Table 3.2
- \(D_m\) = embedment depth

The stability against forward sliding of the structure at the interface between the reinforced fill and the subsoil should also be considered. The tendency of the active earth pressure to cause the wall to slide horizontally must be opposed by the frictional resistance at the base of the wall. The resistance to movement should be based upon the properties of either the
subsoil or the reinforced fill, whichever is the weaker, and consideration should be given to sliding on or between any reinforcement layers used at the base of the structure. As per BS 8006: 1995, for long term stability, where there is reinforcement – to – soil contact at the base of the structure,

\[ f_s R_h \leq R_h \frac{a \tan \phi}{f_{ms}} + \frac{\alpha c L}{f_{ms}} \] ........................ (3.9)

where,

\( f_s \) = the partial factor against base sliding (Table 3.2)

\( R_h \) = horizontal factored disturbing force

\( a \) = interaction coefficient relating soil / reinforcement interfacial friction angle with \( \tan \phi \)

\( \phi \) = internal friction angle of the weaker soil

\( c \) = cohesion of the weaker soil

\( \alpha \) = adhesion coefficient relating soil cohesion to soil - reinforcement bond

\( f_{ms} \) = partial material factor applied to \( \phi \) and \( c \) (Table 3.2).

To check the overall stability of the structure, all potential slip surfaces should be considered, including those passing through the structure. In the case of a failure plane passing through the structure the resistance to failure provided by the reinforcement crossing the failure plane should be considered. If residual shear surfaces are present, then appropriate soil parameters should be used. The appropriate analysis method and the factors of safety used should conform to BS 8002: 1994.

The total settlement is the combined effect of the settlement of the foundation soil under the influence of the pressures imposed by the structure and the internal compression of the reinforced backfill. The calculations of foundation settlements supporting reinforced soil structures follow classical soil mechanics theory. The actual pressures imposed on foundations by reinforced soil structures are lower and more evenly distributed than conventional concrete structures and this normally acts to reduce foundation settlements.
The amount of settlement within the reinforced volume will depend mainly upon the nature of the fill, its compaction and the vertical pressures within the fill. The pressure will be a function of the height of structure, fill type, surcharge loading and the type of facing.

3.4.5.3 Internal stability analysis

Stability within a reinforced structure is achieved by the reinforcing elements carrying tensile forces and transferring them by friction, friction and adhesion or friction and bearing. In addition, forces can be transferred through fill trapped by the elements. The fill is then able to support the shear and compressive forces. Internal stability is concerned with the integrity of the reinforced volume. The structure has the potential to fail by rupture or loss of bond of the reinforcements. The arrangement and layout of reinforcing elements should be chosen to provide stability and to suit the size, shape and detail of the facing. For simplicity, a uniform distribution of identical reinforcing elements may be used throughout the height of the wall. However, it may be economical to divide the height of the wall into a number of zones and to design appropriate reinforcing elements for each zone (BS 8006: 1995).

The potential failure mechanisms which should be considered are stability of individual elements, resistance to sliding of upper portions of the structure and stability of wedges in the reinforced fill. The factors which influence stability that should be included in the design check are the capacity to transfer shear between the reinforcing elements, the tensile capacity of the reinforcing elements and the capacity of the fill to support compression.

The tie back wedge method is commonly used to determine the internal stability. The coefficient of earth pressure should be taken as the active condition \( K_a \) for both the ultimate limit state and serviceability limit state. As per BS 8006: 1995, the maximum ultimate limit state tensile force \( T_j \) to be resisted by the \( j \)th 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.
\[ T_j = T_{pj} + T_{sj} + T_{fj} + T_{ej} \] ........................ (3.10)

where,

\( T_{pj} \) = tensile force per metre run developed in the jth layer of reinforcement due to self weight of fill plus any surcharge on the reinforced fill

\( T_{sj} \) = tensile force per metre run developed due to a vertical strip load

\( T_{fj} \) = tensile force per metre run developed due to a horizontal shear load

\( T_{ej} \) = tensile force per metre run developed in the jth layer of reinforcement due to the cohesive forces in the reinforced soil fill

The effects of vertical strip load and horizontal shear load are not considered in this work and hence they are not discussed further. The other forces are calculated (as per BS 8006: 1995) assuming Meyerhof distribution of stresses as:

\[
T_{pj} = \frac{K_{a1} \left( f_{s} \gamma_{s} h_{j} + f_{q} q \right) S_{sj}}{K_{a2} \left( f_{s} \gamma_{s} h_{j} + 3 f_{q} q \right) \left( h_{j} / L \right)^{2}} \] ........................ (3.11)

\[
T_{fj} = 2 S_{fj} \frac{c}{f_{ms}} \sqrt{K_{a1}} \] ........................ (3.12)

where,

\( K_{a1} \) = coefficient of active earth pressure of backfill soil

\( K_{a2} \) = coefficient of active earth pressure of retained soil

\( S_{sj} \) = vertical spacing of reinforcements at the jth level in the wall

\( f_{s} \) = partial factor applied to dead loads as per Table 3.1

\( f_{q} \) = partial factor applied to traffic loads as per Table 3.1

\( f_{ms} \) = partial material factor applied to cohesion as per Table 3.2

The resistance of the jth reinforcing element should be checked against rupture and adherence failure while carrying the factored loads. The tensile
strength of the jth layer of reinforcing elements needed to satisfy local stability considerations is:

\[
\frac{T_D}{f_r} \geq T_j \quad \text{.................... (3.13)}
\]

in which,

\( T_i \) = maximum value obtained from Eqn. 3.10

\( T_D \) = design strength of the reinforcement

\( f_n \) = partial factor for economic ramifications of failure

Ramification means an usually unintended consequence of an action, decision, or judgment that may complicate a situation or make the desired result more difficult to achieve. \( f_n = 1 \) for medium height walls where failure would result in moderate damage and \( f_n = 1.1 \) for high walls supporting principal roads or railway embankments. The perimeter, \( P_j \), of the jth layer of reinforcing elements needed to satisfy local stability considerations is:

\[
P_j \geq a \tan \phi_{sl} \frac{T_j}{f_p f_n} \left( \frac{f_s}{f_r} \right) \left( h_j + f_n q \right) + \frac{\alpha c L_{sj}}{f_m s f_p f_n} \quad \text{.................... (3.14)}
\]

where,

\( f_p \) = partial factor for reinforcement pullout resistance taken as 1.3 from Table 3.2

\( L_{sj} \) = embedded length of reinforcement in the resistant zone outside the failure wedge

3.4.5.4 Serviceability limit considerations

The serviceability of a structure will usually depend upon the deformations. The deformation will be the sum of the reinforcement strain during construction and loading and the subsequent creep during its service life. For metallic reinforcements, the creep is negligible and consequently, the strain \( \varepsilon_i \) in the jth layer of reinforcements may be estimated from Eqn. 3.15.
\[ e_j = \frac{T_{avj} \cdot L}{E \cdot A_j} \] ........................ (3.15)

where,

- \( T_{avj} \) = average tensile load along the length of the \( j \)th layer of reinforcements
- \( E \) = elastic modulus of the reinforcement
- \( A_j \) = cross sectional area of the \( j \)th layer of reinforcement
- \( T_{av} \) = average tensile load along the length of the \( j \)th layer of reinforcements

### 3.5 SUMMARY

Gabion faced retaining walls are truly revolutionary. The gabion units are strong, flexible and dimensionally stable and can be assembled quickly and easily. These factors contribute to installations which look better, last longer and cost less over the project life than those constructed of competitive products. Due to the endless advantages of the gabion retaining structures, they are now being preferred to the conventional RCC walls. Because of their special functional characteristics and their strength, the use of gabion structures for protective works in residential areas offers the advantage of rapid integration with the surrounding environment. Presently, they are being widely used for a variety of applications like retaining earth and water, highway protection, rock fall protection, river training works, channel lining, soil erosion protection, high security fencing, oil pipeline protection etc. The construction methods and the design procedure of gabion faced reinforced soil retaining walls have been detailed in this chapter.