9.3 BEARING CAPACITY contd...

9.3.2 Safe bearing pressure

There are different methods available to determine the safe bearing pressure on rocks. However, the applicability of different methods for the determination of safe bearing pressure on rock for different situations has to be understood. There is some guidelines in IS 12070-1987 for the basis for design method. Net allowable bearing pressure ($q_a$) based on rock material and RMR are also given in IS:12070 - 1987 guidelines.

Table 9.1: IS 12070-1987 guideline for choosing the basis of design method for rock foundation.

<table>
<thead>
<tr>
<th>Method basis</th>
<th>Rock quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock mass classification</td>
<td>Good rock with wide (1-3m) or very wide (&gt;3m) spacing of discontinuities.</td>
</tr>
<tr>
<td>Core strength</td>
<td>Rock mass with close discontinuities at moderately close (0.3-1m) spacing.</td>
</tr>
<tr>
<td>Pressure meter</td>
<td>Rock of low to very low strength (&lt;500kg/cm$^2$), rock mass with discontinuities at close (5-30cm) or very close (&lt;5cm) spacing, fragmented or weathered rock.</td>
</tr>
<tr>
<td>Plate load test</td>
<td>Rock of very low strength (&lt;250 kg/cm$^2$), rock mass with discontinuities at very close spacing, fragmented or weathered rock.</td>
</tr>
</tbody>
</table>
Table 9.2 Net allowable bearing pressure ($q_a$) based on rock material (IS:12070 - 1987)

<table>
<thead>
<tr>
<th>Material</th>
<th>$q_a$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive crystalline bedrock including granite, diorite, gneiss, trap,</td>
<td>10.0</td>
</tr>
<tr>
<td>hard limestone and dolomite</td>
<td></td>
</tr>
<tr>
<td>Foliated rocks such as schist or slate in sound condition</td>
<td>4.0</td>
</tr>
<tr>
<td>Bedded limestone in sound condition</td>
<td>4.0</td>
</tr>
<tr>
<td>Sedimentary rock, including hard shales and sandstones</td>
<td>2.5</td>
</tr>
<tr>
<td>Soft or broken bedrock (excluding shale) and soft limestone</td>
<td>1.0</td>
</tr>
<tr>
<td>Soft shale</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 9.3: Net allowable bearing pressure ($q_a$) as per RMR (IS: 12070 - 1987)

<table>
<thead>
<tr>
<th>Classification no.</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description of rock</td>
<td>Very good</td>
<td>Good</td>
<td>Fair</td>
<td>Poor</td>
<td>Very poor</td>
</tr>
<tr>
<td>RMR</td>
<td>100–81</td>
<td>80–61</td>
<td>60–41</td>
<td>40–21</td>
<td>20–0</td>
</tr>
<tr>
<td>$q_a$ (MPa)</td>
<td>6.0–4.5</td>
<td>4.5–2.9</td>
<td>2.9–1.5</td>
<td>1.5–0.6</td>
<td>0.6–0.4</td>
</tr>
</tbody>
</table>

**IS 12070-1987 recommendations on rock foundations,**

- The permissible settlement for calculation of safe bearing pressure from plate load test should be taken as 12mm even for large loaded areas. The low value for settlement of foundation is due to heterogeneity of rocks.
- In case of rigid structures like silos, the permissible settlement may be increased judiciously, if required.
- Where site is covered partly by rocks and partly by talus deposits or soil, care should be taken to account for heterogeneity in deformability of soil and rocks.
- It is recommended that plate load tests be conducted on talus or soil and bearing pressure be recommended considering 12 mm settlement.
9.3.3 Estimation of bearing capacity

When loaded area is same or slightly less than the spacing of the open vertical joints, ultimate bearing capacity- UCS

Tarzaghi’s expression may be adopted assuming general shear failure. If the loaded area much smaller,

\[ q_{ult} = 1.2 c' N_c + 0.5 \gamma B N_r \]

When no test data (c’ & \phi’) is available,

\[ q_{ult} = \sigma_{ci} \left( \frac{RQD}{100} \right)^2 \]

For heavily fractured rock

When rockmass is heavily fractured, and the foundation is located at some depth

\[ q_{ult} = \gamma D_f \tan^4(45+\phi'/2) \]

By considering the crushing of rock under the footing,

\[ q_{ult} = \sigma_{ci} \left[ \frac{2}{1-\sin \phi'} \right] \text{ or } q_{ult} = \sigma_{ci} (N_\phi + 1) \]

\[ N_\phi = \tan^2(45+\phi'/2), \phi' = \text{friction angle of intact rock} \]

9.3.4 Analysis of bearing capacity on rock (Goodman, 1989)

As discussed, variety of failure modes are possible for footing under load and the analysis using limit equilibrium is relatively complex. It is found that in a isotropic rock, safe bearing pressure often occurs at a settlement approximately equal to 4-6% of the footing width (Goodman, 1989). Strength of the crushed rock under footing may be analysed considering zone A and B as shown in Figure 9.4. Considering the zone B in uniaxial compression and zone triaxial compression with confinement \( P_h \), the maximum load \( q_f \). The bearing capacity of rock is dependent on the residual strength of rock in the post peak region rather than the compressive strength of intact rock.
The bearing capacity of a homogeneous, discontinuous rock mass can’t be less than the UCS of the rock mass around the footing. The bearing capacity based on Figure 9.5 is given as,

\[ q_f = q_a (N_b + 1) \]

\[ N_b = \tan^2 \left( 45 + \frac{\phi}{2} \right) \]
Module 9 : Foundation on rocks

Here, it is assumed that some load For an open jointed rock mass in which laterally across the joints. Modifying this boundary condition for an open jointed rock mass in which lateral stress transfer is nil is given by the following equation. It reduces the BC only when the S/B ratio is in the range of 1-5, upper limit increasing with friction

\[ q_f = q_u \left\{ \frac{1}{N_b} - 1 \left[ N_\phi \left( \frac{S}{B} \right)^{(N_b - 1)/N_b} - 1 \right] \right\} \]

9.4 PRESSURE BULBS - IN ELASTIC HALF PLANE (Goodman, 1989)

The stress and deformation behaviour of the near by rock mass for a rock foundation may be evaluated assuming elastic and isotropic material behaviour using

Man displacement (\(u\)) = \(\frac{C \cdot P \cdot (1 - \nu^2) \cdot a}{E}\)

where,

\(C = \) Constant dependent on the boundary condition. For perfectly roigid plate \(C = \pi/2\)
and for flexible plate, \(C = 1.7\).

\(P = \) The plate pressure, (Contact force per unit plate area)

\(a = \) Plate radius

\(E, \nu = \) Elastic modulsu and poisson's ratio respectively

How a line load is transferred to the rock in the case of a general line load acting on rocks with various geological structures is very important. If we consider a line load acting normal to the surface of a semi infinite homogeneous, elastic and isotropic medium as shown in Figure 9.6. The normal stress acting along any radius with \(\theta\) constant is principal stress, and is equal to,

\[ \sigma_r = \frac{2 \cdot P \cdot \cos \theta}{\pi \cdot r} \]

The tangential stress and the shear stress referred to these local axes are both zero. The locaus \(\sigma_r\) constant proves to be a circle tangent to the point of application of \(P\) and centred at depth
P/π.σ. Family of such circles are know as pressure bulbs. Similarly, when the load acts in shear Figure 9.7. The radial stress in this situation,

\[ \sigma_r = \frac{2 Q \sin \theta}{\pi r} \]

for inclined line load, if we split to normal and shear load, the problem reduces to normal line load and shear load. The bulb may be represented as in Figure 9.8. When discontinuities are present, the bulb narrows down due to limited stress along the discontinuities. The pressure bulbs from the model studies are shown in Figure 9.9. Figure 9.10 show the equal stress bulbs from model studies carried out by Gaziev and Erilakhman (1971).

Figure 9.6: Pressure bulb resulting from loading of an elastic half space in normal line load
Figure 9.7: Pressure bulb resulting from loading of an elastic half space in shear line load

Figure 9.8: Inclined line load
Figure 9.9: Narrowing and deepening of the bulb of pressure due to limited shear stress along discontinuities

Figure 9.10: Line of equal stress or pressure bulbs from model studies

(Gaziev and Erilakhman, 1971)