GEOSYNTHETIC ENGINEERING: IN THEORY AND PRACTICE

Prof. J. N. Mandal

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Module - 9
LECTURE - 49
Geosynthetics for ground improvement
Recap of previous lecture.....

- Finite Element Modelling of Encased Stone Columns
  Radial Deformation
  Effect of encasement stiffness on radial deformation
  Relative Shear Stress Distribution
  Influence of stiffness of encasement
  Influence of length of encasement
  Influence of stiffness on length of encasements

- Analytical Study on Encased Stone Column

- Ground Improvement using Geocell
  Reinforcement Mechanism
  Design Considerations

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When compressive load is applied on the plate, plastic zones are first initiated at the corners of the plate.

A gradual increase of load extends them until they meet across both the ends.

Further increase in load causes these zones to extend inwards to the center of the plastic material, finally meeting at the center. The two plates may then approach each other (Johnson and Mellor, 1980).

An estimate of the resistance offered by the plastic layer before the plates approach may be done by drawing the slip line field.
The construction of the slip line field was first outlined by Prandtl (1921) and later studied by Skolovoski (1961) and others.

Johnson and Mellor (1983) applied the theory for compression of a block between two rough, rigid plates.

Jenner et al. (1988) suggested the slip line method to design geocells on thin foundation soil for supporting the embankments.
Compression of a block between rough, rigid parallel plates

Slip-line filled and pressure distribution for compression between rough parallel plates (Johnson and Mellor, 1983)

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To illustrate the construction of slip line field (numerical solution), a typical road section is considered:

*National highway*: three lane, two way with a 3 m divider, 24 meter width, situated over a 6 m thick soft clay layer of undrained cohesion 10 kN/m².

Geocell layer of 1 m thickness is provided at the base of the embankment.

The embankment has side slopes of 2 horizontal to 1 vertical.

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Geocell beneath an embankment over soft soil

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Plan view of geocell mattress (After Bush et al. 1990)

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A slip line field can be drawn with various increments. The accuracy of the solution can be increased by lowering the increments.

However, a 15 degree slip line field is drawn here. A similar approach was adopted by Bush et al. (1988).

The ultimate resistance diagram is developed by working from the outer edge of the embankment to the central portion.
Bearing pressure diagram (After Bush et al., 1990)

- The rigid head lies at a distance of 1.25d from the centre line of the embankment where ‘d’ is the depth of clay layer from the edge of the embankment.

- The pressure (p) at the edge of the embankment is 5.71C_u. It remains constant up to a distance of 'd'.

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Slip line field for a typical road section (After Bush et al., 1990)

- Lines emerging from X are \( \alpha \) lines.
  
  (i.e. along which \( p + 2C_u \cdot \phi \) is constant)

- Lines emerging from Y are \( \beta \) lines.
  
  (i.e. along which \( p - 2C_u \cdot \phi \) is constant)
The arc YG₃ can be divided into sectors (Six in this case, since a 15 degree slip line field is drawn). For a 10 degree slip line field, the number of sectors would be nine.

Further the chord YG₅ is drawn at an angle of 7.5 degree to the horizontal to intersect the line XG₅ at G₅.

The line G₅G₄ is drawn at an angle of 22.5 degrees to the horizontal. Similarly G₄G₂, G₂G, GG₁ and G₁G₃ are drawn at angle of 37.5, 52.5, 67.5 and 82.5 degree respectively to the horizontal.

Line G₅H₆ is drawn at an angle of 7.5 degrees to the line XG₅. Thus, the slip line field can be completed up to the rigid head.
The resistance at \( H \) can be calculated as follows:

\( G_3H_1 \) is an \( \alpha \) line along which \((p + 2C_u \cdot \Phi)\) is constant.

From \( G_3 \) to \( H_1 \) is a negative rotation of 15 degrees i.e. 0.262 radians.

At \( G_3 \), \( p = 5.71C_u \)  
Along \( G_3H_1 \), \( p = 5.71 C_u + 2 C_u \cdot \Phi \)

\[
\text{Therefore, } p_{H_1} = 5.71 C_u + 2 C_u \cdot \Phi \\
= C_u (5.71+2 \times 0.262) \\
= 6.234 C_u
\]

Therefore, \( p/C_u = 6.234 \)
\( p = \text{Normal pressure on rough face} \)
\( C_u = \text{Soil shear strength} \)
Similarly, to calculate pressure at H, moving from \( H_1 \) to H along a \( \beta \) line with rotation = +15 degrees i.e. 0.262 radians (anticlockwise).

Along a \( \beta \) line, \( p - 2 \, C_u \). \( \Phi \) = constant.

At \( H_1 \), \( p = 6.234 \, C_u \)

Therefore, \( p_H = C_u \times (6.234 + 2 \times 0.262) = 6.758 \, C_u \)

The pressure diagram can be plotted by calculating pressures at various points.
The stress distribution across the rigid head can be determined from the following equation,

\[
\frac{p'}{C_u} = \frac{2(2I + 0.5d)}{2X} + \frac{p}{C_u}
\]

\(p/ C_u\) is the value read from the stress field at the edge of the rigid head.

\(I = \sum\) (rotation of chords * horizontal chord length)
\(X = \sum\) (horizontal chord lengths)
\(d =\) Depth of the soft layer = 6m, and
\(p' =\) Average stress over the rigid head

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For the present case the rigid head is at a distance of 1.25 d i.e. 7.5 m from centerline of embankment.

Calculation of mean stresses across the rigid head

<table>
<thead>
<tr>
<th>Chord X</th>
<th>AB</th>
<th>BC</th>
<th>CD</th>
<th>AD</th>
</tr>
</thead>
<tbody>
<tr>
<td>X = 7.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rotation in degrees</td>
<td>37.5</td>
<td>22.5</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Rotation in radians (θ)</td>
<td>0.654</td>
<td>0.395</td>
<td>0.131</td>
<td></td>
</tr>
<tr>
<td>Product I (Σθ*X)</td>
<td>1.2425</td>
<td>0.968</td>
<td>0.3799</td>
<td>I = 2.5905</td>
</tr>
</tbody>
</table>

Therefore,

\[
p' = \frac{2*(2*2.5905+3.0)}{(2*7.25)} + \frac{p}{C_u} = 1.1 + \frac{p}{C_u}
\]
The value of $p/C_u$ may be obtained by extending the normal pressure diagram up to the edge of the rigid head.

Normal pressure diagram (Resistance diagram)

From pressure diagram, $p/C_u = 11$

Therefore, $p'/C_u = 1.1 + 11 = 12.1$

$p' = 12.1 \, C_u$

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Load from half of the embankment including a surcharge of 20 kN/m²

\[ \left( \frac{12 + 24}{2} \right) \times 5 \times 17 + (12 \times 20) = 1530 + 240 = 1770 \text{ kN/m} \]

Resistance from the ultimate resistance diagram (area of the diagram)

\[ = 5.71 C_u \times 6 + \left( \frac{5.71 + 11}{2} \right) C_u \times 8.5 + 7.5 \times 12.1 C_u \]

\[ = (34.26 + 71.02 + 90.75) C_u = 196.03 C_u \]

Therefore, \( C_u \) required to prevent plastic deformation

\[ = \frac{1770}{196.03} = 9.03 \text{ kN/m²} \]

Factor of safety \( = \frac{10 \text{ (available } C_u \text{)}}{9.03 \text{ (required } C_u \text{)}} = 1.1 \)

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The horizontal load in the geogrid mattress within plastic zone,

\[ T_g = \frac{C_u}{\sin \delta} \]

\( \delta = \text{Angle of internal friction between foundation soil and geogrid mattress} = 25^\circ \)

For F.S = 1.5, \( C_u = 15 \text{ kN/m}^2 \)

\[ T_g = \frac{10}{\sin 25} = 23.66 \text{ kN/m} \]

\[ T_g = \frac{15}{\sin 25} = 35.49 \text{ kN/m} \]
Let the design strength of geogrid is 25 kN/m.

Strength for a geocell of 1.00 m,

\[ T = T_{\text{design}} + \frac{T_{\text{design}}}{\sqrt{2}} = 25 + \frac{25}{\sqrt{2}} = 42.68 \text{ kN/m} \]

Strength for a geocell of 0.50 m,

\[ T = 2 \left[ T_{\text{design}} + \frac{T_{\text{design}}}{\sqrt{2}} \right] = 2 \times 42.68 = 85.36 \text{ kN/m} \]

- A nonwoven geotextile can be placed at the bottom of geocell mattress.
- The size of filling material is 40 mm-75 mm. The geocell is overfilled by 100 mm-150 mm before compaction.
Example:

Foundation soil:

Black Cotton, Classification = CH
C = 45 kPa, Φ = 9°

Backfill material:

Type = Fly Ash, C = Negligible, Φ = 20°

Loading Considerations:

The following loads are considered for the design of geocell mattress with surcharge load of 110 kN/m².

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Ground Improvement Using Geocell System

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Solution:

Analysis of geocell reinforced earth wall is carried out by slip line method.

R.E. Wall base width = 10.00 m.

Width of the geocell mattress = 10 – 1 = 9.00 m. (Leaving offset of 1.00 m from facing side)

From stress field diagram,

\[ \frac{P'}{C_u} = \frac{2(2I + 0.5d)}{2X} + \frac{P}{C_u} \]

\[ I = \Sigma (\text{Horizontal Chord Lengths} \times \text{Rotation}) \]
\[ X = \Sigma (\text{Horizontal Chord Lengths}) \]
\[ d = \text{Depth of soft soil layer} \]
\[ P' = \text{Average stress over the rigid head} \]
Calculation of pressure across rigid head slip line

<table>
<thead>
<tr>
<th>Chord</th>
<th>AB</th>
<th>BC</th>
<th>$X = 2.56$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord Length $(X)$</td>
<td>0.7</td>
<td>1.86</td>
<td></td>
</tr>
<tr>
<td>Rotation (Degrees)</td>
<td>22.5</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Rotation (Radians)</td>
<td>0.395</td>
<td>0.131</td>
<td></td>
</tr>
<tr>
<td>Product, $I = (\Sigma \theta \times X)$</td>
<td>0.277</td>
<td>0.244</td>
<td>$I = 0.521$</td>
</tr>
</tbody>
</table>

$$\frac{2(2I + 0.5d)}{2X} = 0.8$$
Bearing Capacity from pressure diagram,

\[
\frac{P'}{C_u} = 0.8 + 9.9 = 10.7
\]

\[
= (2 \times 5.71) C_u + \frac{(5.71+9.9)}{2} \times 4.5 C_u + (2.5 \times 10.7) C_u
\]

\[
= 73.29 C_u
\]
### Detailed calculations:

<table>
<thead>
<tr>
<th>Height of R.E. Wall (m)</th>
<th>Avg. Height (m)</th>
<th>Load (kN/m)</th>
<th>Bearing Capacity (kN/Sq. m)</th>
<th>$C_u$ required (kN/sq. m)</th>
<th>$C_u$ Available (kN/sq. m)</th>
<th>F.O.S</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-12</td>
<td>11</td>
<td>3068</td>
<td>$73.29C_u$</td>
<td>41.85</td>
<td>45</td>
<td>1.07</td>
</tr>
<tr>
<td>8-10</td>
<td>9</td>
<td>2728</td>
<td>$73.29C_u$</td>
<td>37.21</td>
<td>45</td>
<td>1.21</td>
</tr>
<tr>
<td>6-8</td>
<td>7</td>
<td>2388</td>
<td>$73.29C_u$</td>
<td>32.58</td>
<td>45</td>
<td>1.38</td>
</tr>
<tr>
<td>4-6</td>
<td>5</td>
<td>2048</td>
<td>$73.29C_u$</td>
<td>27.94</td>
<td>45</td>
<td>1.61</td>
</tr>
<tr>
<td>2-4</td>
<td>3</td>
<td>1708</td>
<td>$73.29C_u$</td>
<td>23.30</td>
<td>45</td>
<td>1.93</td>
</tr>
</tbody>
</table>
Calculation of Geocell mattress:

Jenner et al. (1988) developed the slip line field to determine the bearing pressure diagram

Horizontal stresses within the geocell, \( \sigma_h = \sigma_n - 2X \).

\( \tau = C_u = \) Shear strength at the interface

\( \sigma_n = \) At the highest part of R.E. wall

\( \phi = 20^\circ \) (Friction angle) for geocell fill material (Fly Ash)

\[
X = \frac{2 \sigma_n \sin^2 \phi \pm \sqrt{4 \sigma_n^2 \sin^4 \phi - 4 \left( \sin^2 \phi - 1 \right) \left( \sigma_n^2 \sin^2 \phi - \tau^2 \right)}}{2 \left[ \sin^2 \phi - 1 \right]}
\]

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<table>
<thead>
<tr>
<th>Height of R.E. wall (m)</th>
<th>$\sigma_n$ At the bottom of the R.E. wall (kN/sq. m)</th>
<th>$X$ (kN/sq. m)</th>
<th>$\sigma_h = \sigma_n - 2X$ (kN/sq. m)</th>
<th>Geocell mattress provided with min strength (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-12</td>
<td>306.8</td>
<td>70.51</td>
<td>165.78</td>
<td>165.78</td>
</tr>
<tr>
<td>8-10</td>
<td>272.8</td>
<td>60.75</td>
<td>151.3</td>
<td>151.3</td>
</tr>
<tr>
<td>6-8</td>
<td>238.8</td>
<td>50.64</td>
<td>137.52</td>
<td>137.52</td>
</tr>
<tr>
<td>4-6</td>
<td>204.8</td>
<td>39.92</td>
<td>124.96</td>
<td>124.96</td>
</tr>
<tr>
<td>2-4</td>
<td>170.8</td>
<td>28.10</td>
<td>114.6</td>
<td>114.6</td>
</tr>
</tbody>
</table>
The horizontal load, \( T_g \), in the geocell mattress within the plastic zone,

\[
T_g = \frac{C_u}{\sin \delta}
\]

\[
T_g = \frac{45}{\sin 14} = 186.01 \text{ kN/m}
\]

\( \delta = \text{Angle of internal friction between foundation soil and geogrid mattress} = 14^\circ \)

**Factor of safety required = 1.5**

Therefore, \( C_u = 1.5 \times 45 = 67.5 \text{ kN/m}^2 \)

\[
T_g = \frac{67.5}{\sin 14} = 279.1 \text{ kN/m}
\]
The horizontal load in the geocell mattress = 279.1 kN/m

Let, design strength of the geogrid be 175.0 kN/m

\[ T = T_{\text{design}} + \frac{T_{\text{design}}}{\sqrt{2}} \]

Strength for a geocell of 1.00 m

\[ T = 175 + \frac{175}{\sqrt{2}} = 298.74 \text{ kN/m} > 279.1 \]  \hspace{1cm} \text{(Safe)}

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The calculated and provided thickness of the geocell mattress are given in the following table.

<table>
<thead>
<tr>
<th>Height of wall (m)</th>
<th>Maximum horizontal force (kN/m)</th>
<th>Height of the geocell mattress provided (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-12</td>
<td>165.78</td>
<td>Calculated: 0.551, Provided: 0.60</td>
</tr>
<tr>
<td>8-10</td>
<td>151.30</td>
<td>Calculated: 0.512, Provided: 0.55</td>
</tr>
<tr>
<td>6-8</td>
<td>137.52</td>
<td>Calculated: 0.461, Provided: 0.50</td>
</tr>
<tr>
<td>4-6</td>
<td>124.96</td>
<td>Calculated: 0.425, Provided: 0.45</td>
</tr>
<tr>
<td>2-4</td>
<td>114.60</td>
<td>Calculated: 0.381, Provided: 0.40</td>
</tr>
</tbody>
</table>
Stabilization of Soft Swelling Soil Using Geocell

Finite element analysis has been carried out using commercial available software 'Plaxis 2D professional' for both unreinforced and reinforced case of geocell stabilization technique.

Geocell stabilization system for soft swelling soil

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The model basically consists of 50 mm thick EPS geofoam at the base, overlaid by 30 mm thick concrete layer and 1000 mm thick geocell mattress at the top. The geocell is filled with fly ash.

### Properties of Materials Used in the Geocell Stabilization System

<table>
<thead>
<tr>
<th>Properties of Materials Used in the Geocell Stabilization System</th>
<th>*M-C Mohr-Coulomb</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Material</strong></td>
<td><strong>Cohesion C (kPa)</strong></td>
</tr>
<tr>
<td>Black Cotton Soil</td>
<td>7</td>
</tr>
<tr>
<td>Sand</td>
<td>0.01</td>
</tr>
<tr>
<td>Fly ash</td>
<td>40</td>
</tr>
<tr>
<td>Geofoam</td>
<td>15</td>
</tr>
<tr>
<td>Concrete</td>
<td>100</td>
</tr>
<tr>
<td>Geogrid</td>
<td>-</td>
</tr>
</tbody>
</table>

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The obtained vertical displacement without geocell system is 133.4 mm which is quite more than that obtained with geocell system, 31.35 mm. This shows the effectiveness of geocell system.

<table>
<thead>
<tr>
<th>Ground Improvement Systems</th>
<th>Vertical Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without geocell</td>
<td>133.4</td>
</tr>
<tr>
<td>With geocell</td>
<td>31.35</td>
</tr>
</tbody>
</table>

➢ The mobilization of shear stresses is more in unreinforced case than in the reinforced case.

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Vertical displacement for (a) unreinforced case, and (b) reinforced case

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Cross section of vertical displacement for (a) unreinforced case and (b) reinforced case

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Relative shear stresses (a) unreinforced case and (b) reinforced case

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Nimbalkar and Mandal (1999) carried out centrifuge modeling for embankment on soft soil for various field conditions to investigate lightweight fill application of expanded polystyrene geofoam (EPS) as substitute for conventional fill material with economy and greater serviceability.

Artificial neural network analysis was also carried out to investigate the geofoam material application which can be successfully applied for slope stability analysis of the embankment on soft soil.
Geofoam reinforced slope

Compressive stress Vs. strength of EPS geofoam

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Finite Element Analysis:

Finite element analysis was carried out using commercial available software 'Plaxis 2D V8' for both unreinforced and reinforced case.

Modeling of unreinforced embankment

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(a) Vertical displacement of unreinforced embankment

(b) Horizontal displacement of unreinforced embankment

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Modeling of geofoam embankment

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Geosynthetics Engineering: In Theory and Practice

Vertical displacement of geofoam embankment

Horizontal displacement of geofoam embankment

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Comparison of factor of safety and displacements without and with geofoam

<table>
<thead>
<tr>
<th>Embankment</th>
<th>Factor of Safety</th>
<th>Vertical displacement (m)</th>
<th>Horizontal displacement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without Geofoam</td>
<td>0.782</td>
<td>0.534</td>
<td>0.892</td>
</tr>
<tr>
<td>With Geofoam</td>
<td>1.965</td>
<td>0.155</td>
<td>0.236</td>
</tr>
</tbody>
</table>
The conventional materials for embankment construction on soft soils have been replaced by EPS geofoam. The density of EPS geofoam considered is 20 kg/m³.

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### Properties of materials used in the geofoam embankment system

<table>
<thead>
<tr>
<th>Properties</th>
<th>Cohesion (c) (kPa)</th>
<th>Angle of internal friction $\phi$ (°)</th>
<th>Young’s modulus (E) (kPa)</th>
<th>Poisons ratio ($\mu$)</th>
<th>Axial stiffness (EA) (kN/m)</th>
<th>Material model</th>
<th>Drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft soil</td>
<td>7</td>
<td>12</td>
<td>1500</td>
<td>0.32</td>
<td>-</td>
<td>Mohr-Coulomb</td>
<td>Undrained (A)</td>
</tr>
<tr>
<td>Geofoam</td>
<td>15</td>
<td>0</td>
<td>4500</td>
<td>0.1</td>
<td>-</td>
<td>Mohr-Coulomb</td>
<td>Non porous</td>
</tr>
<tr>
<td>Concrete</td>
<td>100</td>
<td>-</td>
<td>$23 \times 10^9$</td>
<td>0.24</td>
<td>-</td>
<td>Linear elastic</td>
<td>Non porous</td>
</tr>
<tr>
<td>Geogrid</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1000</td>
<td>Elastic</td>
<td>-</td>
</tr>
</tbody>
</table>

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Finite element analysis was carried out using commercial available software ‘Plaxis 2D professional version’ for both unreinforced and reinforced case.

The obtained horizontal and vertical displacements in reinforced case are lesser in magnitude than that of unreinforced case.

The mobilization of shear stresses is more in unreinforced embankment than in the embankment reinforced with EPS geofoam.

There is a considerable increment in the value of factor of safety for embankment reinforced with geofoam.
Horizontal displacement patterns in unreinforced embankment

Horizontal displacement patterns in embankment reinforced with geofoam

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Relative shear stress patterns in unreinforced embankment
Reinforced with geofoam

Relative shear stress patterns in embankment
Reinforced with geofoam

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Comparison of factor of safety and displacements without and with geofoam

<table>
<thead>
<tr>
<th>Embankment</th>
<th>Factor of Safety</th>
<th>Horizontal displacement (m)</th>
<th>Vertical displacement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without Geofoam</td>
<td>0.9768</td>
<td>1.611</td>
<td>1.123</td>
</tr>
<tr>
<td>With Geofoam</td>
<td>2.96</td>
<td>0.02653</td>
<td>0.01182</td>
</tr>
</tbody>
</table>

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SUMMARY

- A comprehensive listing of all ground improvement techniques are illustrated on various situations.

- The current ground improvement techniques have been presented.

- The correct choices of attractive, affordable and suitable techniques are discussed.

- This will enhance the reader’s intuitive understanding of the current ground improvement methods.

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Practically natural prefabricated vertical drain, geocell and geofoam are an ideal choice for economical improvement for all type of soils in developing countries like India.

NPVD is a low cost alternative system.

NPVD is advocated because it is more appropriate choice, technically feasible, superior and more economical, low energy utilization, especially in developing countries like India.
A new mechanism of geocell and geofoam is proposed and analyzed, the results should match with the observation and further development is also necessary.

With the advent of geosynthetics, the revolution of different ground improvement systems for weak or soft soils are picking up at an unprecedented pace. The technically viable and economically feasible is first priority in any upcoming projects.

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Please let us hear from you

Any question?

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THANKS FOR LISTENING