Safe bearing capacity evaluation of the bridge site along Syafrubesı-Rasu wagadhi road, Central Nepal

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ABSTRACT

The research work was carried out at the proposed bridge site along Syafrubesı-Rasu wagadhi Road, Central Nepal. The study was based on field SPT/DCPT and laboratory tests. N values were determined from SPT/DCPT test. The index and mechanical properties of granular soils were computed in laboratory. Then, ultimate and allowable bearing capacities with safety factor 3 for maximum tolerable limit of 40 mm settlement was estimated by empirical equations provided by Teng (1988) and Terzaghi and Peck (1978). It is recommended that safe bearing capacity for 6 m size open square foundation at left and right abutments are suitable within 4.5 to 6 m depth.

INTRODUCTION

The part of this paper is prepared to evaluate the safe bearing capacity of the soil strata for foundation of the bridge along the proposed Syafrubesı-Rasu wagadhi Road in Central Nepal. It lies at an altitude of 1395 m above mean sea level, at the central development region of Nepal (Fig.1). The study was carried out with the detailed exploration of drilling borehole and necessary field and laboratory tests. The engineering properties of granular soils from disturbed samples are correlated with the N values obtained from standard penetration test/dynamic cone penetration test (SPT/DCPT) after necessary corrections to evaluate the bearing pressure for an open 6 m size square foundation. A Borehole is drilled at Chainage K0+159 on the left and right abutments with depths of 25–30 m. The proposed bridge at this site lies at steep V-shaped valley formed by the Bhotekoshi River. There is no influence of groundwater condition. The ground water level is existed at 9.60 m and 14.36 m below the ground at left and right abutments. In addition, the bed rock in this area is encountered at 33.75 m below the surface. This delineates that there is no influence of rock on foundation.

OBJECTIVES AND METHODOLOGIES

The objective of the present study is to determine the bearing capacity of the strata and recommend the depth of foundation.

Boreholes were drilled by water flush rotary drilling method with double tube core barrels. Core barrels of Nx (2 1/8”-core size) and Bx (1 5/8”-core size) with respective casings are used. Nx core barrel is used till it could proceed the drilling smoothly. Then the borehole size is reduced to Bx size. All the holes are completed to the size of Bx barrel. Barrels of 1.5 m length are used for convenient SPT/DCPT hammering at every 1.5 m depth. The recovered cores are stored in the standard core boxes of 0.3 m width.
and one metre length with 4 to 5 partition suitable for Nx and Bx cores, respectively. The recovered soft and sandy material and collected sludge are placed in the plastic sample bags before storing in the core boxes.

Disturbed samples of granular soil were collected from split spoon sampler obtained during Standard Penetration Tests. The collected samples were tested for index and strength properties namely, grading analysis and hydrometer tests, moist density, dry density and specific gravity, natural moisture content, maximum and minimum dry densities, natural angle of repose, and direct shear tests. Then, soils were classified according to the Unified Soil Classification System (USCS) based on Japanese Geotechnical Standard (JGS). Moist density, dry density and specific gravity tests of soils were determined on the basis of Japanese Industrial Standard (JIS) System. Direct shear tests on soils were tested on the basis of British Standard (B.S.).

The determined engineering properties of granular soils from disturbed samples are correlated with the N values obtained from SPT/DCPT after necessary corrections. The allowable bearing pressure for an open square foundation is equal to the ultimate bearing capacity divided by a factor of safety of 3. The following empirical equations are used for the open foundation on granular soils (Teng 1988).

\[ q_{ult} = 2N^2BR_w + 6(100+N^2)DR'_w \ldots \ldots (1) \]

Where, \( q_{ult} \) is net ultimate bearing pressure, psf; \( N \), adjusted standard penetration resistance value; \( B \), width of footing, ft; \( D \), depth of footing, ft, and \( R_w \) and \( R'_w \), correction factor for position of water table.

When water level is below the bottom of footing, \( R'_w \) is 1 and when the water level is bottom of footing, \( R_w \) is 0.5 (Fig. 2).

The allowable bearing pressure for open foundation based on tolerable settlement of 25 mm is given by the following empirical (Terzaghi and Peck 1978) relationship.

\[ q_a = 720 \,(N–3) \,((B+1)/2B)^2 \,((B+D)/B)R'_w \ldots (2) \]

Where, \( q_a \) is net allowable bearing pressure in psf for maximum settlement of 25 mm. The allowable bearing pressure for tolerable settlement of 40 mm is considered. Therefore,

\[ q_a =1.6 \, \times \, 720 \,(N–3) \,((B+1)/2B)^2 \,((B+D)/B)R'_w \ldots (3) \]

The bearing capacity of a footing is largely affected by the characteristics of the volume of soil within a depth equal to about 1 to 1.5 times the width of the
footing. The bearing capacity of granular soil depends upon the unit weight and the angle of internal friction of the soil, both of which vary primarily with the relative density of the soil. The relative density of granular soils in-situ is generally determined by SPT or DCPT tests. The dynamic cone resistance is correlated with the SPT N (Nc) values as given as 1.50 N for depth up to 3.0 m, 1.75 N for depth from 3.0–6.0 m, and 2.0 N for depth greater than 6.0 m. The relative density of granular soils in-situ is determined either by standard penetration test or by dynamic cone penetration test. Because of the extreme difficulty in obtaining undisturbed samples from coarse granular soils, the engineering properties of such soils are determined from disturbed samples and correlated with the N values obtained from SPT/CPT after necessary corrections. The correlation between SPT with the relative density, angle of internal friction and unit weight of granular soil are shown in Table 1.

For SPT tests made at shallow depth, the number of blows is usually too low. At a greater depth, the same soil with same relative density would give higher penetration resistance and the weight of overburden soil on SPT may be approximated by the following formula.

\[ N = N' \left( \frac{50}{P+10} \right) \]............(4)

Where, \( N' \) and P are actually recorded SPT value and effective overburden pressure, not exceeding 40 psi respectively.

### RESULTS

#### Geotechnical properties at chainage k0+159

The river bank is comprised of alluvial soil, colluvial soil and mixture of them (Fig. 3). The right abutment mostly comprises of colluvium, whereas the left abutment comprises of colluvium underlain by alluvial terraces of the Bhotekoshi River. Two boreholes at BHN K0+159 LA and BHN K0+159 RA were drilled at pier location of left and right abutments respectively. Fig. 4 (a and b) shows the borehole log data in order to represent the soil strata as well as N-values.

The borehole BHN K0+159 LA was drilled to a depth of 30 m where as the borehole BHN K0+159 RA drilled to a depth of 35.75 m. Both boreholes comprised of boulders and/or cobbles with matrices of gravel, gravelly sand, and sand. As the depth increases, the percentage of boulders also increases on both boreholes. At left abutment, beyond the depth of 9 m, the boulders dominate the finer materials. At the right abutment, the stratum of colluvial and alluvial

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Table 1 Relationship between Dr, SPT and the Angle of Internal Friction of Granular Soils (Teng, 1988).

<table>
<thead>
<tr>
<th>Compact V. Loose</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
<th>V. Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dr, %</td>
<td>0–15</td>
<td>15–35</td>
<td>35–65</td>
<td>65–85</td>
</tr>
<tr>
<td>N/150 blows</td>
<td>0</td>
<td>4</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>( \phi^0 )</td>
<td>28</td>
<td>30</td>
<td>36</td>
<td>41</td>
</tr>
</tbody>
</table>

\[ 1_\gamma_t, \text{ kNm}^{-3} \] \( < 580, 550–725, 635–750, 640–810, > 750 \)

\[ 2_\gamma_b, \text{ kNm}^{-3} \] \( < 350, 320–375, 350–405, 380–490, > 435 \)
mixed soil were found to contain finer material more than very coarse grained soil. The alluvial soil soil predominately consists of fine grained soil with some very coarse grained material. Since, the borehole of right abutment is located at the hillslope, trenches have been driven to depth of 4.5 m from the surface to collect samples. Table 2 summaries the geotechnical properties of the site.

### Table 2 Geotechnical Properties of BHN K0+159 LA and BHN K0+159 RA

<table>
<thead>
<tr>
<th>Borehole Depth (m)</th>
<th>Sample No.</th>
<th>Description</th>
<th>w (%)</th>
<th>Gs</th>
<th>ρd (t/m³)</th>
<th>θ (°)</th>
<th>c, N/cm²</th>
<th>φ°</th>
</tr>
</thead>
<tbody>
<tr>
<td>BHN K0+159 LA</td>
<td>0.0–1.5</td>
<td>S0 Gap graded sandy gravel</td>
<td>0.879</td>
<td>2.67</td>
<td>1.257</td>
<td>1.018</td>
<td>0.924</td>
<td>36.3</td>
</tr>
<tr>
<td></td>
<td>3.0–4.5</td>
<td>S0 Gap graded sandy gravel</td>
<td>3.816</td>
<td>2.676</td>
<td>1.423</td>
<td>1.253</td>
<td>1.174</td>
<td>37.3</td>
</tr>
<tr>
<td></td>
<td>1.5–1.95</td>
<td>S1 Well graded sand (SW)</td>
<td>13.983</td>
<td>2.686</td>
<td>1.6</td>
<td>1.389</td>
<td>1.292</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td>10.3–10.65</td>
<td>S2 Gap graded gravelly sand</td>
<td>19.857</td>
<td>2.665</td>
<td>1.469</td>
<td>1.287</td>
<td>1.204</td>
<td>33.3</td>
</tr>
<tr>
<td>BHN K0+159 RA</td>
<td>0.0–1.5</td>
<td>S0 Gap graded sandy gravel</td>
<td>3.365</td>
<td>2.67</td>
<td>1.257</td>
<td>1.018</td>
<td>0.924</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3.0–4.5</td>
<td>S0 Gap graded sandy gravel</td>
<td>1.135</td>
<td>2.665</td>
<td>1.354</td>
<td>1.24</td>
<td>1.185</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>6.0–6.45</td>
<td>S1 Gap graded sand with some fines</td>
<td>25.202</td>
<td>2.658</td>
<td>1.43</td>
<td>1.1</td>
<td>0.965</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td>10.5–10.95</td>
<td>S2 Uniformly graded sand (SP)</td>
<td>29.748</td>
<td>2.682</td>
<td>1.433</td>
<td>1.179</td>
<td>1.08</td>
<td>36.3</td>
</tr>
<tr>
<td></td>
<td>16.5–16.95</td>
<td>S3 Gap graded sand</td>
<td>19.118</td>
<td>2.697</td>
<td>1.481</td>
<td>1.283</td>
<td>1.201</td>
<td>33.7</td>
</tr>
<tr>
<td></td>
<td>30.6–33.35</td>
<td>S4 Gap graded sand</td>
<td>21.738</td>
<td>2.655</td>
<td>1.445</td>
<td>1.165</td>
<td>1.066</td>
<td>35.3</td>
</tr>
</tbody>
</table>

Bearing capacity analysis of chainage k0+159

Bearing capacity analyses for strata up to 12 m for left abutment and 13.5 m depth for right abutment for 6 m size of open square foundation are calculated for ultimate strength with consideration of 40 mm allowable settlement. Hence, computed safe bearing capacities are shown detail in Tables 3 and 4. The
Fig. 4 Borehole log of (a) left abutment and (b) right abutment redrawn after Anon (2007)
ground levels were encountered at depths of 9.60 m and 14.3 m respectively. Since, the topography of right abutment is very steep, only 50% of the safe bearing capacity given in Table 4 should be considered for foundation.

### CONCLUSIONS

Since the topography of the area is very steep, the bearing stratum recommends that only 50% of the safe bearing capacity should be considered for foundation given above in the table. In addition, there are no influences of groundwater and deep seated rock formation. The study shows that all the strata below 4.5 m has safe bearing capacity of more than 100 t/m², the bearing stratum at the depth of 4.5 to 6 m seems to be enough for a load up to 50 t/m² at left abutment. In addition, the bearing capacity of the strata at 10.5 m has lower bearing capacity of than the upper strata at right abutment. Therefore, the open 6 m size square foundation is recommended at the depth of 4.5 to 6 m.

### REFERENCES

