Safe bearing capacity evaluation of the bridge site along Syafrubesi-Rasuwagadhi road, Central Nepal

*Suman Manandhar[1], Uttam Bol Shrestha[2], Noriyuki Yasufuku[1],

Kiyoshi Omine[1], and Taizo Kobayashi[1]

[1]*Department of Civil and Structural Engineering, Kyushu University, Fukuouka, Japan*
[2]*Department of Mines and Geology, Lainchaur, Kathmandu, Nepal*

ABSTRACT

The research work was carried out at the proposed bridge site along Syafrubesi-Rasuwagadhi 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 Syafrubesi-Rasuwagadhi 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

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*Corresponding author:
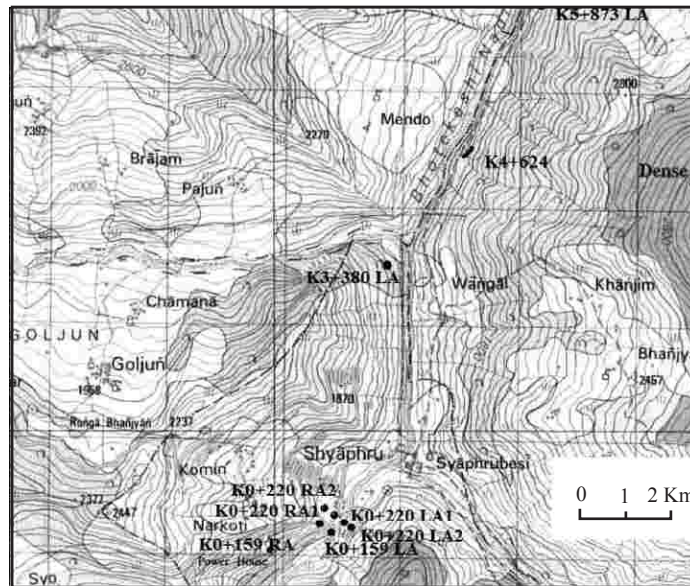E-mail address: geosuman@gmail.com

Fig. 1 Location map of the study area

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).

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

Where, qult is net ultimate bearing pressure, psf; N, adjusted standard penetration resistance value; B, width of footing, ft; D, depth of footing, ft, and Rw 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, Rw 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, qa 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,

$$q_a = 1.6 \times 720 (N-3) ((B+1)/2B)^2 ((B+D)/B)R'w$$
$$\ldots\ldots\ldots(3)$$

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

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

Compact	V. Loose	Loose	Medium	Dense	V. Dense
D_r, %	0–15	15–35	35–65	65–85	85–100
N/150 blows	0	4	10	30	50
ϕ°	28	30	36	41	
$^1\gamma_t$, kNm⁻³	< 580	550–725	635–750	640–810	> 750
$^2\gamma_b$, kNm⁻³	< 350	320–375	350–405	380–490	> 435

$^1\gamma_t$, moist unit weight, $^2\gamma_b$, submerged unit weight

following formula.

$$N = N'(50/(P+10))............(4)$$

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
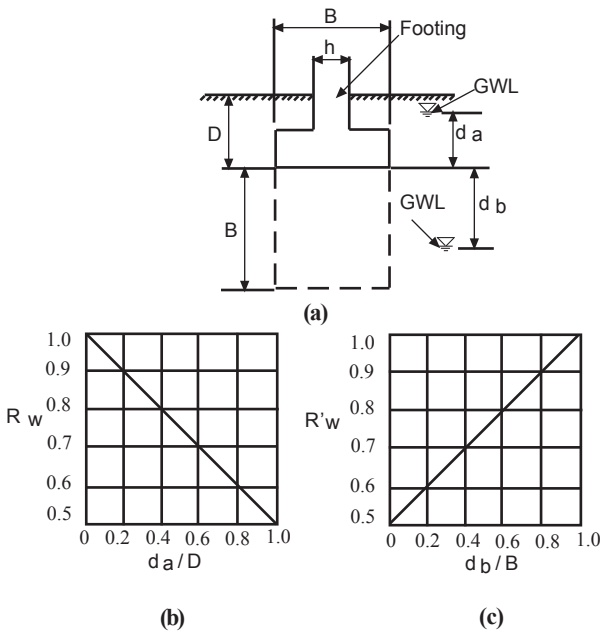

Fig.2 Correction factor of position of water level: (a) depth of water level with respect to dimension of footing; (b) water level above base of footing; (c) water level below base of footing
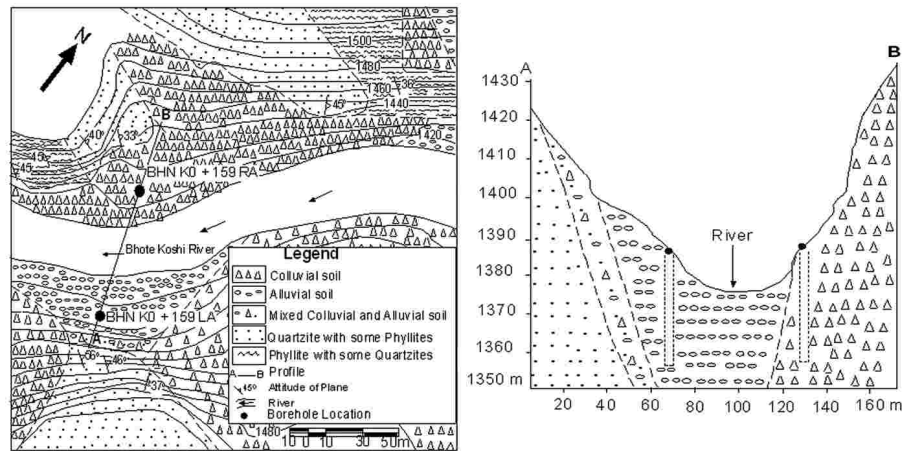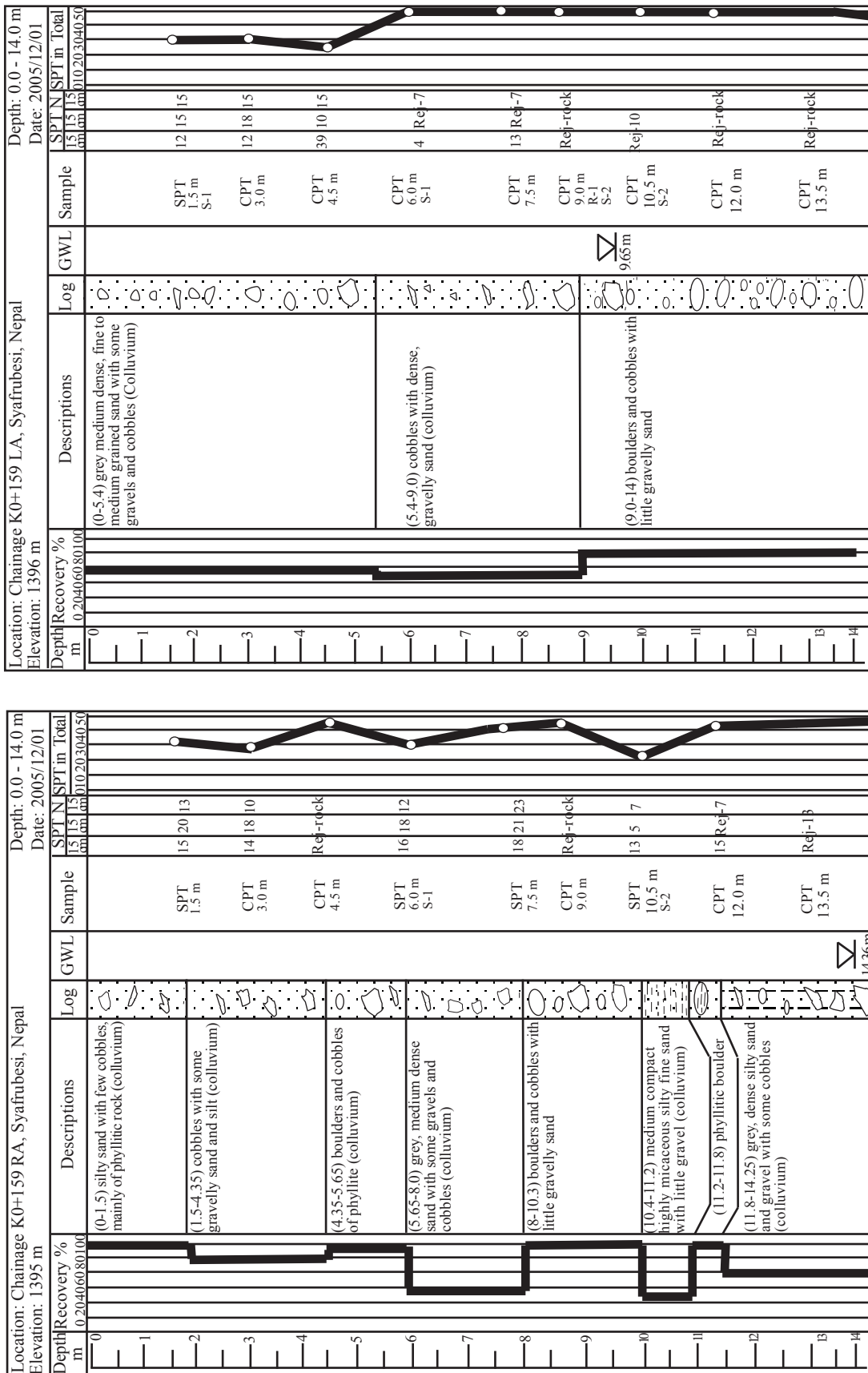
Fig. 3 A geological map and cross-section

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

Borehole	Depth (m)	Sample No.	Description	w (%)	G_s	ρ t/m³	ρd t/m³ $\rho dmax$	$\rho dmin$	Angle of Repose °	Shear Results c, N/cm²	ϕ°
BHN K0+159 LA	0.0–1.5	S0	Gap graded sandy gravel	0.879	2.67	1.257	1.018	0.924	36.3	-	-
	3.0–4.5	S0	Gap graded sandy gravel	3.816	2.676	1.423	1.253	1.174	37.3	-	-
	1.5–1.95	S1	Well graded sand (SW)	13.983	2.686	1.6	1.389	1.292	33.7	0	32
	10.3–10.65	S2	Gap graded gravelly sand	19.857	2.665	1.469	1.287	1.204	33.3	0	33
BHN K0+159 RA	0.0–1.5	S0	Gap graded sandy gravel	3.365	2.67	1.257	1.018	0.924	-	-	-
	3.0–4.5	S0	Gap graded sandy gravel	1.135	2.665	1.354	1.24	1.185	-	-	-
	6.0–6.45	S1	Gap graded sand with some fines	25.202	2.658	1.43	1.1	0.965	33.7	0	35
	10.5–10.95	S2	Uniformly graded sand (SP)	29.748	2.682	1.433	1.179	1.08	36.3	0	38
	16.5–16.95	S3	Gap graded sand	19.118	2.697	1.481	1.283	1.201	33.7	0	34
	30.6–33.35	S4	Gap graded sand	21.738	2.655	1.445	1.165	1.066	35.3	0	34

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.

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

(b)

Location: Chainage K0+159 LA, Syafrubesi, Nepal
Elevation: 1396 m
Depth: 0.0 - 14.0 m
Date: 2005/12/01

Depth m	Recovery %	Descriptions	Log	GWL	Sample	SPT N	SPT in Total
		(0-5.4) grey medium dense, fine to medium grained sand with some gravels and cobbles (Colluvium)			SPT 1.5 m S-1	12 15 15	
					CPT 3.0 m	12 18 15	
					CPT 4.5 m	39 10 15	
		(5.4-9.0) cobbles with dense, gravelly sand (colluvium)			CPT 6.0 m S-1	4 Rej-7	
					CPT 7.5 m	13 Rej-7	
					CPT 9.0 m R-1 S-2	Rej-rock	
		(9.0-14) boulders and cobbles with little gravelly sand		▽ 9.65 m	CPT 10.5 m S-2	Rej-10	
					CPT 12.0 m	Rej-rock	
					CPT 13.5 m	Rej-rock	

(a)

Location: Chainage K0+159 RA, Syafrubesi, Nepal
Elevation: 1395 m
Depth: 0.0 - 14.0 m
Date: 2005/12/01

Depth m	Recovery %	Descriptions	Log	GWL	Sample	SPT N	SPT in Total
		(0-1.5) silty sand with few cobbles, mainly of phyllitic rock (colluvium)			SPT 1.5 m	15 20 13	
		(1.5-4.35) cobbles with some gravelly sand and silt (colluvium)			CPT 3.0 m	14 18 10	
		(4.35-5.65) boulders and cobbles of phyllite (colluvium)			CPT 4.5 m	Rej-rock	
		(5.65-8.0) grey, medium dense sand with some gravels and cobbles (colluvium)			SPT 6.0 m S-1	16 18 12	
		(8-10.3) boulders and cobbles with little gravelly sand			SPT 7.5 m	18 21 23	
					CPT 9.0 m	Rej-rock	
		(10.4-11.2) medium compact highly micaceous silty fine sand with little gravel (colluvium)			SPT 10.5 m S-2	13 5 7	
		(11.2-11.8) phyllitic boulder			CPT 12.0 m	15 Rej-7	
		(11.8-14.25) grey, dense silty sand and gravel with some cobbles (colluvium)		▽ 14.36 m	CPT 13.5 m	Rej-13	

Fig.4 Borehole log of (a) left abutment and (b) right abutment redrawn after Anon (2007)

Table 3: Bearing Capacity of foundation of 6 m size K0+159 LA

D	[3]GWCF		[4]C-SPT,	q_{ult}	[5]q_a`	[6]q_s
m	R_w	R'_w	N-Value	t/m²	t/m²	t/m²
3	1.00	1.00	33	184	70	70
4.5	1.00	0.92	25	136	60	60
6	1.00	0.80	50	559	116.70	117
7.5	1.00	0.67	50	578	110.00	110
9	1.00	0.55	50	572	100.30	100
10.5	0.96	0.50	50	591	100.30	100
12	0.90	0.50	50	643	109.40	109

[3]GWCF (Ground water correction factor), [4]C-SPT (corrected-SPT), [5]qa`(Allowable bearing capacity based on 40 mm settlement), [6]qs (Safe bearing capacity)

Table 4: Bearing Capacity of foundation of 6 m size K0+159 RA

D	[3]GWCF		[4]C-SPT,	q_{ult}	[5]q_a`	[6]q_s
m	R_w	R'_w	N-Value	t/m²	t/m²	t/m²
3	1.00	1.00	19	67.4	37.20	37
4.5	1.00	1.00	50	534.9	127.60	128
6	1.00	1.00	30	250.0	83.80	84
7.5	1.00	1.00	46	668.0	150.00	150
9	1.00	0.88	50	819.0	160.00	160
10.5	100.00	0.82	22	192.0	66.50	66
12	1.00	0.68	50	839.0	148.80	149
13.5	1.00	0.58	50	812.0	137.50	138

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.

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.

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

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