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Design of composite well-pile foundation (CWPF): A case study of bridge foundation in Nepal

Conception d'une fondation sur pieux composites (CWPF): Une étude de cas de fondation de pont au Népal

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ABSTRACT: A bridge designed initially with well foundation may encounter unexpected boulder strata while sinking at a certain depth causing a hurdle for further sinking. Construction of over three major bridges is being delayed and costly in Nepal due to unexpected hard started within design depth of well foundation. Due to difficulties in well sinking, several contractors left the job and some contractors took more than ten years to complete the well foundations. To overcome such a peculiar situation by taking advantages of well foundation, a composite well-pile foundation (CWPF) is proposed. The CWPF is a hybrid foundation where a well foundation rests on grouped piles. This is a unique technique to be used for the first time in Nepal. The CWPF can be constructed by driving piles from the inside of the well after it sinks to the favourable depth. This study proposes a capacity-based design and numerical analysis identifying the integral behaviour of CWPF. The proposed capacity-based design is further illustrated with an example. A CWPF for a five spanned, 175 m long bridge, consisting of post-tensioned cast-in-situ superstructure was designed using the proposed capacity-based design method. A large portion of the load is beard by well; however, the proportion of load carried by well depends on the depth of well foundation below the scoured depth. A finite element simulation showed that both stress and strain in the soil around the CWPF foundation are within the permissible range. Overall the results indicate satisfactory behaviour of hybrid foundation.

RÉSUMÉ : Un pont conçu initialement avec des fondations de puits peut rencontrer des strates de blocs inattendues en s'enfonçant à une certaine profondeur, provoquant un obstacle à l'enfoncement ultérieur. La construction de plus de trois grands ponts est retardée et coûteuse au Népal en raison d'un démarrage difficile inattendu dans la profondeur de conception de la fondation du puits. En raison des difficultés de fonçage du puits, plusieurs entrepreneurs ont quitté le travail et certains entrepreneurs ont mis plus de dix ans pour terminer les fondations du puits. Afin de surmonter une telle situation particulière en profitant des avantages de la fondation de puits, une fondation composite sur pieux de puits (CWPF) est proposée. Le CWPF est une fondation hybride où une fondation de puits repose sur des pieux groupés. C'est une technique unique à utiliser pour la première fois au Népal. Le CWPF peut être construit en enfonçant des pieux depuis l'intérieur du puits après son enfoncement à la profondeur favorable. Cette étude propose une conception basée sur la capacité et une analyse numérique identifiant le comportement intégral de la CWPF. La conception proposée basée sur la capacité est illustrée plus en détail par un exemple. Un CWPF pour un pont à cinq travées de 175 m de long, constitué d'une superstructure coulée sur place post-tendue, a été conçu à l'aide de la méthode de conception basée sur la capacité proposée. On a observé qu'une grande partie de la charge est barbe par puits; cependant, la proportion de charge portée par le puits dépend de la profondeur de la fondation du puits sous la profondeur décapée. Une simulation par éléments finis a montré que la contrainte et la déformation dans le sol autour de la fondation de la CWPF se situent dans la plage autorisée. Dans l'ensemble, les résultats indiquent un comportement satisfaisant de la fondation hybride.

KEYWORDS: bridge foundation, composite well-pile foundation, capacity based design, finite element simulation

1 INTRODUCTION

Under the different budget head, Sixteen hundred bridges in both the strategic road network and local road network are under construction by the Department of Roads, Nepal. Similarly, around four hundred new bridges designs are carried out by the department almost every year. The Geology of Nepal is quite dynamic as a result of active lithospheric plate tectonics forming high Himalayas collision of Indian plate with the Eurasian plate, which began about 55 million years ago and the process is still going on (Dhakal 2012). As a result of the unique geology of Nepal, we found frequent variable in geotechnical parameters in short length.

A major portion of the capital budget for the infrastructure sector being invested by the government is in the transport sector, being road transport prominent, some bridges goes under construction, maybe without proper geotechnical investigation resulting in hurdles in foundation construction, and the project period extended to years.

2 CASE STUDY HISTORY

Bridge located in Strategic Road Network, Lumbini Parikrama Road, Manigram Motipur section at ($27^0$37’54.46’’N, $83^0$26’19. 72’’E). The proposed bridge is over Tinau River, one of the maj or rivers in the Lumbini region having a design discharge of 26 13.37 m^3/s (50-year return period) at the bridge site location.

Agreement for the construction work was made on 9th July 2010. The bridge consists of five-span 35 meter c/c of the length of each, simply supported Post Tensioned Prestressed I girder. Well foundation with 17.0 m depth (below pile cap) and 7 m outer diameter with steining thickness of 0.9m both for abutment and pier were designed. Due to the encounter of 2.5 m to 3.0 m depth conglomerate rock on Pier 3, Pier 4 and Abutment A2 (Figure 1), further sinking of well with available technology and contractor capacity did not become possible and took years for the solution of the problem. Table 1 describes the sinked well condition in detail of the bridge.

Table 1. Detail of sinked well

Description	Pier 3	Pier 4	Abutment 2
Total Depth of sinked Well below Pile Cap (m)	9.0	7.2	9.3
Depth of Well below Scour level (m)	5.6	3.8	9.3
Depth of conglomerate layer (m)	2.5	2.8	3.0

$$\sigma_x = K_h\left(\frac{z}{D}\right)(D-z)\theta \; Or \quad \sigma_x = mK_v\left(\frac{z}{D}\right)(D-z)\theta$$

$$P = \frac{2mK_v\theta}{D} I_v \tag{2}$$

$$m = \frac{K_H}{K_v}$$

I_v = Moment of Inertia about the axis passing through the c.g of the vertical projected area

Moment of P at the base level:

$$M_p = mK_v\theta I_v \tag{3}$$

Pressure Distribution at Base:
The vertical deflection at distance ($x+x_c$) from the centre of

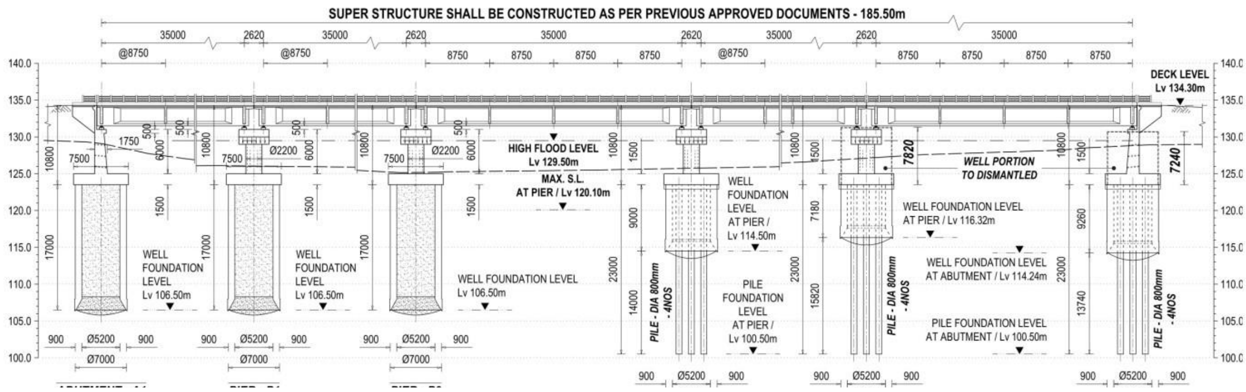


Figure 1. General feature of the bridge

rotation (figure 2, c) $\rho = (x + x_c)\theta$ and vertical soil reaction $\sigma_z = K_V(x + x_c)\theta$

Moment at Base:

$$M_B = \int_{-\frac{B}{2}}^{+\frac{B}{2}} K_V(x + x_c)\theta x dA \tag{4}$$

Applying Equilibrium condition,

$$\frac{M}{r}(1 + \mu\mu') - \mu W < H < \frac{M}{r}(1 - \mu\mu') + \mu W \tag{5}$$

Where,
μ = Coefficient of friction between base and the soil = Tan Φ
μ' = Coefficient of friction between sides and the soil = Tan δ
W = Total downwards loads acting at the base of the well
$r = \frac{D}{2} \cdot \frac{I}{mI_V}$

3 STABILITY ANALYSIS OF EXISTING WELLS

Calculation of allowable bearing pressure and lateral stability analysis under static load combination becomes vital for the stability analysis of well. Well sunk to certain depth below the scour level known as grip length, should be such that the resistance from side is able to resist lateral forces and the base pressures under worst loading condition should be within permissible limit. Indian Road Congress (IRC:45-1972) recommendation for the estimation of resistance of soil below the respective scour level of abutment and pier, for the analysis of well foundation in cohesionless soil is based on elastic theory to estimate the pressure at side and base under design load but to determine the actual factor of safety against shear failure, ultimate soil resistance need to be calculated.

3.1 *Lateral stability analysis*

Pressure Distribution on sidewalls: well sunk to depth D below the scour level.

Horizontal soil reaction acting on the sides:

$$P = \int_0^D L\sigma_x dz \tag{1}$$

Where,

3.2 *Elastic state condition*

The maximum soil reaction from the sides cannot exceed the maximum passive pressure at any depth, if soil remains in an elastic state. This accounts to the condition that at depth y:

$$m * \frac{M}{I} \leq (K_P - K_A) \tag{6}$$

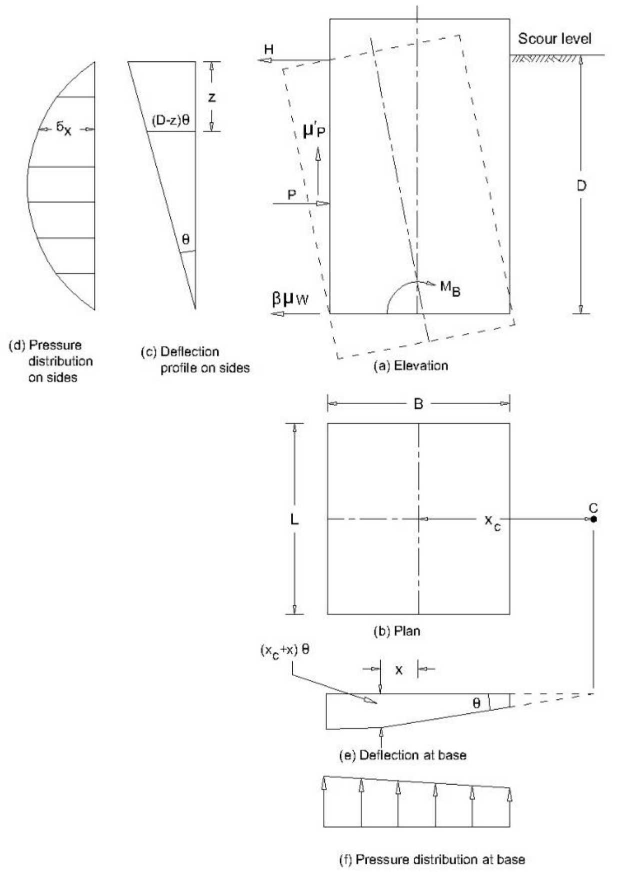


Figure 2. Deflection and pressure distribution on side and base

3.3 *Allowable bearing capacity calculation*

The maximum soil reaction from the sides cannot exceed the maximum passive pressure at any depth, if soil remains in an Wells embedded in a soil medium due to its own weight and generally are rested on a hard stratum. Although the embedded portion of the foundation under maximum scouring scenario provides the side resistance against lateral loading, the skin friction due to this embedded part is generally ignored in bearing capacity calculation. Teng (1962) suggested that settlement can be a guiding factor of the bearing pressure of well foundations embedded in sand and proposed an equation of bearing pressure for a permissible settlement. According to Indian design code IS:3955 (1967), allowable bearing pressure for wells in cohesionless soils can be evaluated from the standard penetration number, N. As per this codal provision, the allowable bearing pressure, Qa (in kN/m^2), for a foundation of width B (in m) and depth (below maximum scour level) D (in m) can be expressed as:

$$Q_a = 0.054N^2B + 0.16(100 + N^2)D \tag{6}$$

These critical three well foundations are checked for the above condition as per the load combinations specified in IRC: 78 (2000) Clause 706.1.1. Pier 4, as mentioned in table 1, sinked to least depth below scour level is insufficient for withstanding the necessary design load. The summary of stability in elastic state and bearing pressure under respective critical load combinations is presented in table 2 as under these two condition pier 4 seems critical.

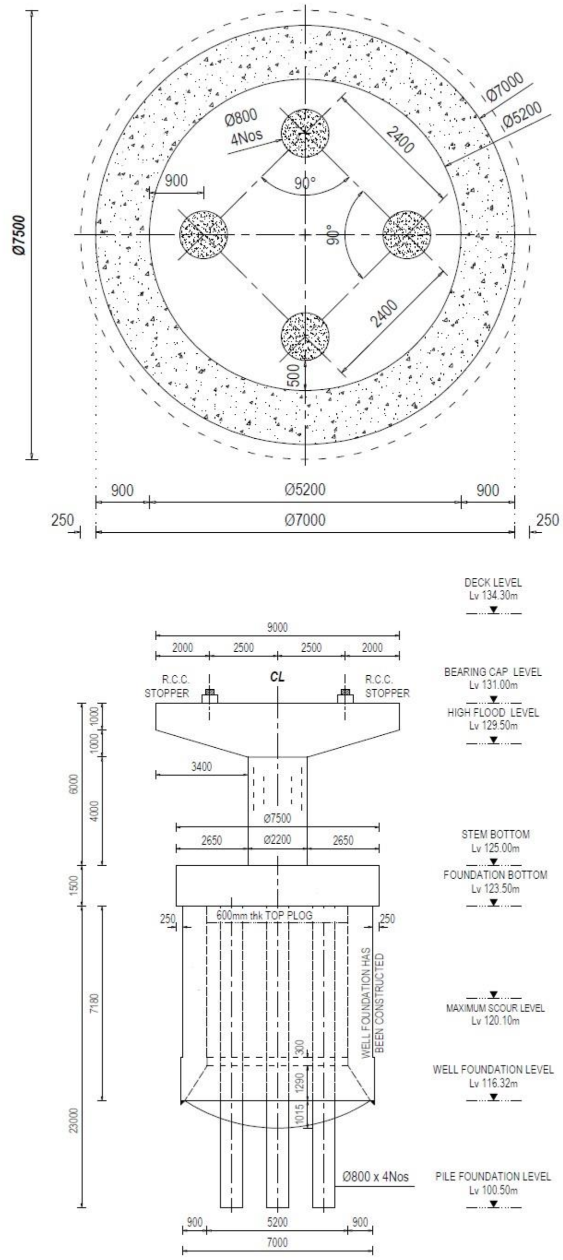


Figure 3. Detail of Pile and well at Pier 4

From the above table it is concluded that the well foundation is not stable by the calculation based on elastic theory. It also represent that maximum bearing pressure exceeds the allowable bearing capacity as the allowable bearing capacity of the well in pier 4 is less than that of Pier 3 and abutment 2.

Table 2. Stability and bearing pressure of foundation

Description	Pier 3	Pier 4	Abutment 2
Check for the stability (Elastic theory) : m*M/I ≤ Υ^1 *(kp-ka), under Critical load combination			
m*M / I, kN/m^3	134.2	*166.5*	92.9
Υ *(kp-ka), kN/m^3	161.5	*161.5*	161.5
Check for the bearing pressure:			
σ_1 =[(W - μ' P)/A] + MB/2I , kN/m^2	531.7	*554.6*	672.1
σ_2 = [(W - μ' P)/A] + MB/2I, kN/m^2	200.2	*121.5*	379.1
Allowable Bearing Capacity (kN/m$^{2)}$	719.0	*537.00*	1086.0

Υ^1 = Density of the soil (submerged density if below water table)

equally at the nodes of the pier column below high flood level. The well has been modelled as shell element and the soil reactions below the scour level are simulated by using area spring. Linear spring was modelled at the bottom of the well. Pile is modelled as the line element discredited at one meter interval and the proper spring coefficient has been assigned.

4.1 *Determination of spring constant*

The load displacement responses of the springs are assumed to be elastopastic, and the SPT-N value of the stratum has been used to calculate the stiffness of the springs.

A horizontal load on a vertical pile is transmitted to the subsoil primarily by horizontal subgrade reaction generated in the upper part of the shaft. A single pile is normally designed to carry load along its axis. Transverse load bearing capacity of a single pile depends on the soil reaction developed and the structural capacity of the shaft under bending.

In case the horizontal loads are of higher magnitude, it is essential to investigate the phenomena using principles of horizontal subsoil reaction adopting appropriate values for horizontal modulus of the soil. In this study, piles are analyzed using modulus of subgrade reaction and lateral resistance offered by soil is modeled by providing springs having stiffness derived using modulus of subgrade reaction. The modulus of subgrade

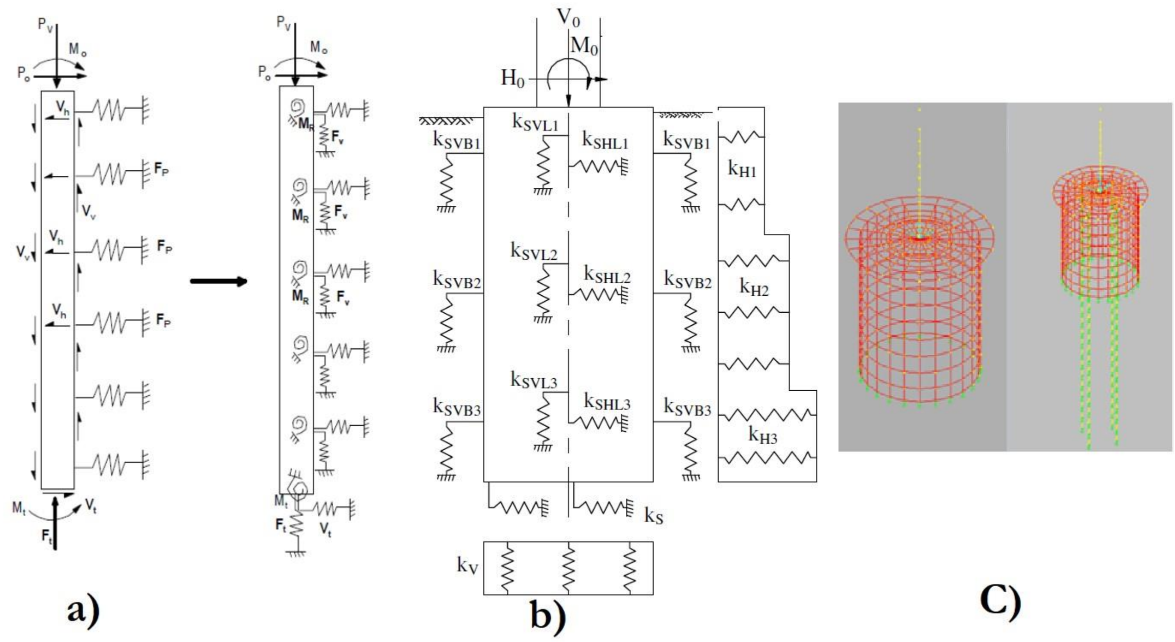


Figure 4. (a) Spring model for the soil reaction on Pile, (b) Spring model for the soil reaction on well, and (c) Numerical model of well foundation without Pile and with Pile

4 PROPOSED SOLUTION

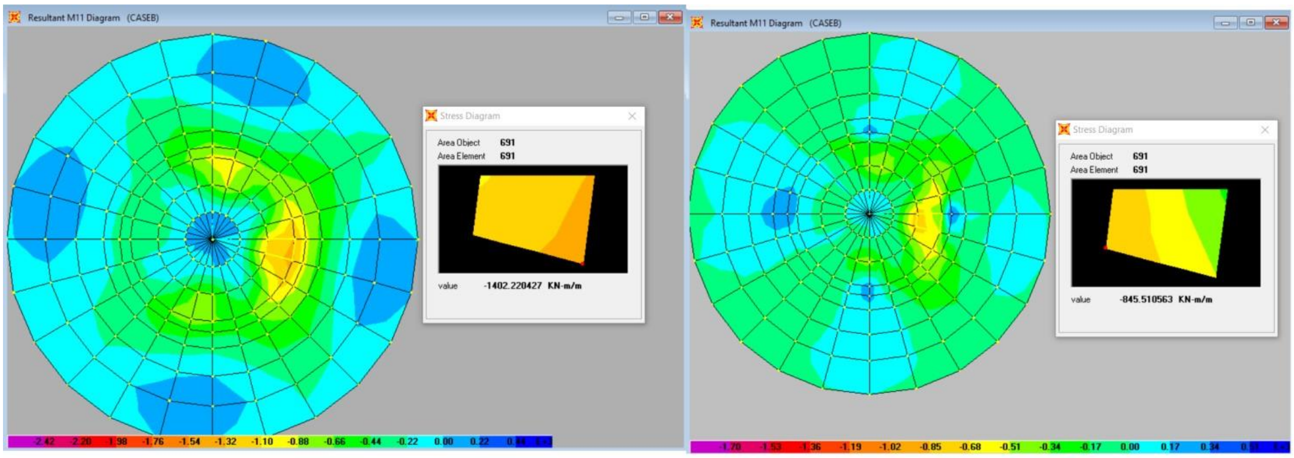
Four bored pile of 23 m length inside well is proposed for improving the structural behaviour of the well importing hybrid behaviour. The piles are arranged as shown in figure 3. Structural behaviour of the proposed model is carried out using SAP2000 (Computer & Structures Inc. 2002). The behaviour of the foundation is examined for the case of Pier 4 with pile ie of hybrid foundation and without pile. The pier column was modelled as frame element. The vertical load of 6960 kN and 2846 kN was applied on the top of the column to replicate the self weight of the superstructure and vehicular live load reaction under critical combination respectively. Similarly breaking load (horizontal load) of 140 kN is applied 1.2m above the road level as prescribed by IRC 6:2000 creating a body constrained node. Frictional force of 53.43 kN applied at the top of the pier. Water current load of of total 231.45 kN is applied by distributing

reaction is seldom measured in lateral pile load test. Node values of "K_s" are required in FEM solution for lateral piles.

However in absence of test results, this value may be approximated as per procedure given below: As per Vesic (1961), modulus of subgrade reaction can be computed using stress-strain modulus Es based on as,

$$k'_s = 0.65 \sqrt[12]{\frac{E_s B^4}{E_f I_f}} \frac{E_s}{1-\mu^2}$$

Where E_s, E_f = modulus of soil and footing respectively, in consistent units B, I_f = footing width and its moment of inertia



a) b)

Figure 5. Bending moment distribution along radial unit strip of the cap (a) well without pile and (b) well with pile

based on cross section in consistent units.

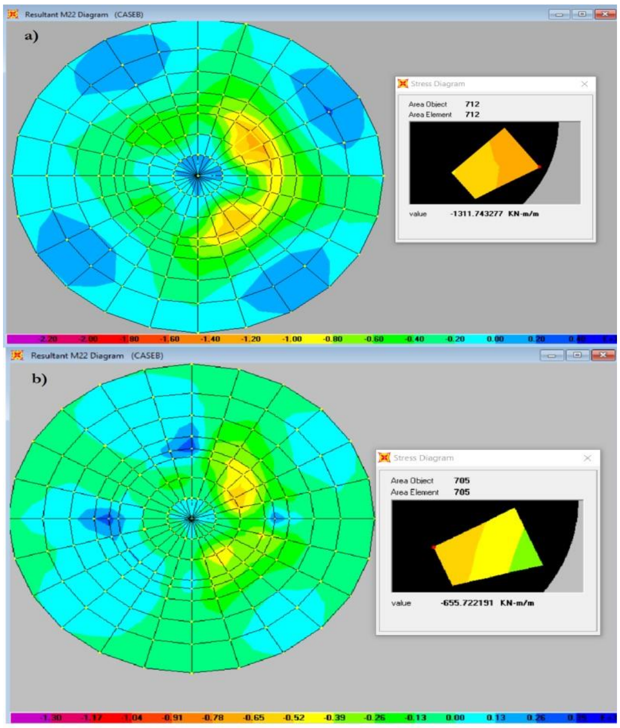


Figure 6. Bending moment distribution along circumferential unit strip of the cap a) Well without pile b) Well with Pile

One can obtain ks from k_s' as,

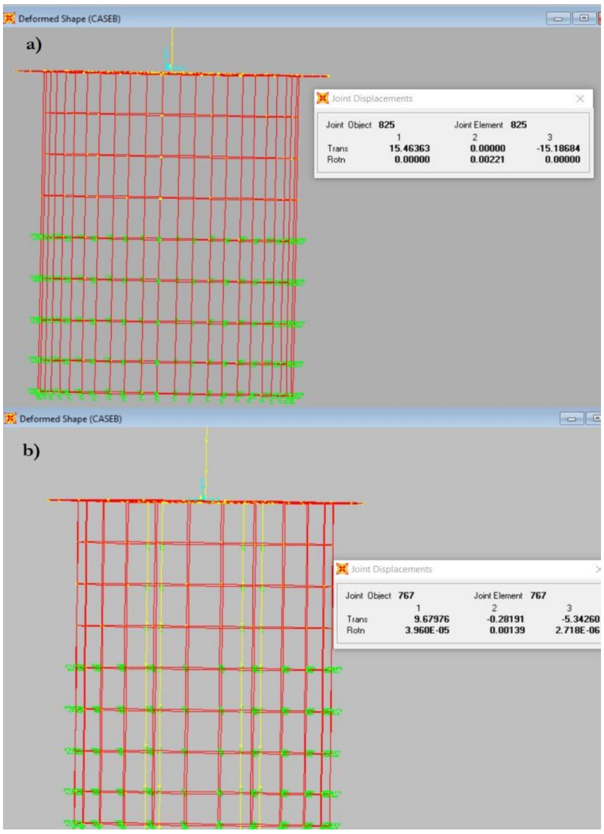
$$k_s = \frac{k'_s}{B}$$

Since the twelfth root of any value multiplied by 0.65 will be close to 1, for practical purposes the Vesic's equation reduces to,

$$k_s = \frac{E_s}{B(1-\mu^2)}$$

But since one does not often have values of E_s, other approximations are useful and quite satisfactory if the computed deflection is reasonable. It has been found that bending moments and computed soil pressure are not very sensitive to what is used for 'K_s' because the structural member stiffness is usually 10 or more as great as soil stiffness as defined by 'K_s'.

Figure 7. Maximum deflection in cap centre a) Well without pile b)



Well with Pile

Vesic has suggested the following for approximating 'K_s' from the allowable bearing capacity qa based on geotechnical data:

$$k_s = 40\ (SF)\ q_a\ \mathrm{kN/m^3}$$

Where, q_a is in kPa. This equation is based on assumption that ultimate soil pressure occurs at a settlement of 0.0254 m. For other values of ΔH = 6, 12, 20 mm etc., the factor 40 can be adjusted to 160, 83, 50 etc. 40 is reasonably conservative but smaller assumed displacement can always be used.

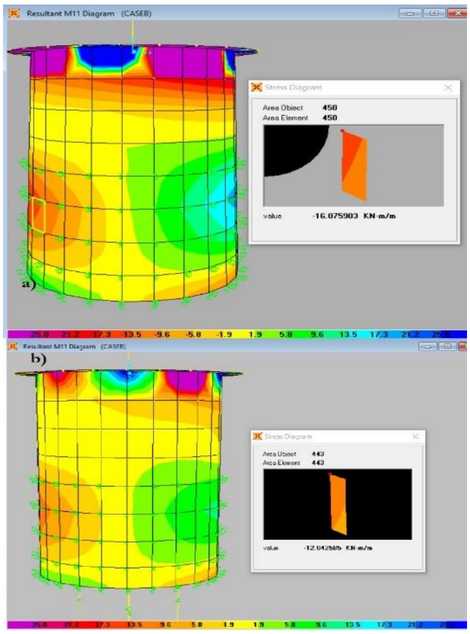


Figure 8. Bending moment distribution along circumferential unit strip of well stem a) Well without pile b) Well with Pile

The most general form for either horizontal or lateral modulus of subgrade reaction is,

$$k_s = A_s + B_s Z^n$$

A_s = constant for either horizontal or vertical members
B_s = coefficient based on depth variation
Z = depth of interest below ground
n = exponent to give ks the best fit.

5 ANALYSIS AND RESULTS

5.1 *Well cap moment distribution*

Figure 5 shows the bending moment distribution along radial unit strip (M_{11}) and figure 6 indicates bending moment distribution along circumferential unit strip for the both, well foundation without pile and with pile of the well cap. Max Induced bending moment for critical load along radial unit strip in case of no pile is observed as 1402.22 KNm/m and that of case with pile is observed as 845.5 KN-m/m. Similarly circumferential direction unit strip moment is 1311.74 KN-m/m and 655.72 KN-m/m for the condition well without pile and with pile respectively under critical load condition.

Similarly from figure 7 Deflection of well cap centre is reduced from 15 mm to 5.3 mm indicating significant influence of composite behaviour for deflection control.

5.2 *Well stem moment distribution*

Bending moment in with influence of pile is reduced by factor 0.25 (Figure 8) for circumferencial strip (M11) and 0.34 for vertical indicating significant improvement with the composite behaviour of pile and well for the reduction of well stem bending moment. Well end veritical settlement due to induced case loading is decreased from 20 mm to 7 mm with the introduction of pile.

5.3 *End bearing pressure*

End bearing pressure is observed critical at link id 682, The maximum bearing pressure in model without pile is 1025 kN/m^2. While bearing pressure with pile model is reduced to 460 kN/m^2. It indicates significant end bearing pressure reduction due to composite behaviour of well and pile.

6 CONCLUSION

Bridge designed initially with well foundation may encounter unexpected boulder strata while sinking at certain depth causing hurdle for further sinking. Construction of over three major bridges is being delayed and costly in Nepal due to unexpected hard started within design depth of well foundation. Due to difficulties in well sinking, several contractors left the job and some contractors took more than ten years to complete the well foundations. In order to overcome such a peculiar situation by taking advantages of well foundation, a composite well-pile foundation (CWPF) is proposed. The CWPF is a hybrid foundation where a well foundation rests on grouped piles. This is a unique technique to be used for the first time in Nepal. The CWPF can be constructed by driving piles from the inside of the well after it sinks to the favourable depth. This study proposes a capacity-based design and numerical analysis identifying integral behaviour of CWPF. The proposed capacity-based

design is further illustrated with an example. A CWPF for a five spanned, 175 m long bridge, consisting of post tensioned cast in-situ superstructure was designed using proposed capacity-

based design method. It was observed that large portion of the axial load is beard by well; however, the proportion of axial load carried by well is depends on the depth of well foundation below the scoured depth. A finite element simulation showed that both stress and strain in soil around the CWPF foundation are in permissible range. Overall the results indicate satisfactory behaviour of hybrid foundation. In general, the caisson could bear the top load and could be used for construction platform in the construction of bridges. The piles under the caisson could control the settlement after the bridges were built.

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