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Combined Horizontal-Vertical loading of Square Foundation in the Offshore Niger Delta of Nigeria

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ABSTRACT

The performance of a square foundation on clay in the offshore Niger Delta subjected to combined horizontal – vertical loading has been attempted .Horizontal forces were evaluated from the impact of varying wave heights on circular piles of 1.0 to 2.0m diameter using available wave height records, while concentric vertical loadings were evaluated from empirical methods of Skempton, Brinch Hanson, and Vesic. The results revealed that sliding failure commenced when the ratio of vertical load to footing area and undrained shear strength (V/As_u) assumes negative values (i.e. at $H/As_u > 3.427$) and for footing stability, the vertical load should be at least twice the anticipated horizontal load. For foundation with the ratio of footing depth to breadth greater than one (D_f /B>1.0), higher horizontal force of about 10.5 times greater than those for D_f /B<1.0 are required to initiate sliding.

Keywords: *Niger Delta. Impacted. Footing. sliding.*

1. INTRODUCTION

Foundations of offshore structures used for oil exploration and exploitation activities are impacted upon by gravity and environmental loads. Gravity loads are due to the self weight of the structure while environmental loads are wave induced, giving rise to vertical, horizontal and moment loading which are subsequently transferred to the foundation. The foundation response to these loads may be assessed under the impact of single loading; vertical, horizontal or moment but generally, offshore structures are being impacted upon by varying degree of combined loading. Studies on foundation subjected to combined loading have been reported by Bell (1991), Martin (1994), Ngo Tran (1996), Taiebat (1999), Zhang (2008) among others. The increasing level of exploration and production activities in the Niger Delta offshore environment calls for further understanding on offshore foundation performance to gravity and environmental loading. These loads are transferred to the offshore foundation generating lateral (u_h), rotational (θ_m) and vertical (u_v) displacement. Recent studies on vertical load induced displacement (Akpila, 2011), moment induced displacement (Ejezie and Akpila, 2011) and horizontal load induced displacement (Akpila and Ejezie, 2011) of offshore foundation in the Niger Delta have been reported. Horizontal forces are evaluated for varying pile diameters and wave heights using Morrison equation, adopting the linear wave theory of Airy (1845), while concentric vertical loading of offshore foundation are generally evaluated using the widely used methods of Skempton (1951), Brinch Hanson (1970) and Vesic (1975).

This paper attempts to present the performance of offshore foundation on clay in the Niger Delta subjected to combined horizontal-vertical loading.

2. MATERIALS AND METHODS

2.1 Wave Characteristics

The wave characteristics; wave height, wave period were deduced from relevant meteorological and oceanographic studies reported by Santala (2002), Cooper (2004) and Offshore Report (2005), while wave celerity, c, and wave length, L, were evaluated for conditions of shallow water waves [Sorenson,2006].

2.2 Hydrodynamic Coefficients

Haritos (2007) reported that inertia coefficients, C_m, and drag coefficient, C_d generally lie in the range of 0.8 to 2.0. However, these coefficients have been reported by Akpila and Ejezie (2011) in the offshore Niger Delta as having C_d of 0.7 and C_m of 1.5. The dimensionless parameters for maximum drag force, K_{Dm}, inertia force, K_{im}, maximum drag moment, S_{Dm}, and maximum inertia moment, S_{im}, were evaluated from standard charts.

2.3 Hydrodynamic Forces

The total instantaneous hydrodynamic force, F, on a submerged structure per elemental length ds of the cylinder (Sorenson, 2006) can be obtained from:

$$F = \frac{C_d}{2}\rho D^2 + C_m \rho \left(\frac{\pi D^2}{4}\right) \frac{2\pi^2 H}{T^2} \left[\frac{\cosh k(d+z)}{\sinh kd}\right] \sin(kx - \omega t) \quad (1)$$

While the maximum horizontal force is obtained by summing both the drag force and inertia force as follows:

$$F = \frac{C_d}{2}\gamma_w D H^2 K_{Dm} + C_m \gamma_w \pi \frac{D^2}{4} H K_{im} \quad (2)$$

where F is horizontal force, γ_w is unit weight of water, D is pile diameter, T is wave period, K is wave number, ω is wave angular frequency and H is wave height.

2.4 Concentric Vertical Loads

The conventional methods of Skempton (1951), Brinch Hanson (1970) and Vesic (1975) were used to evaluate concentric vertical loads for cases of $D_f/B \leq 1.0$ and $D_f/B \geq 1.0$. A recent study on the performance of offshore foundations under vertical load induced displacement in the Niger Delta has been reported by Akpila (2011).

Skempton (1951)

$$\frac{V}{A} = 6s_u(1 + 0.2\,D/2R) + \gamma' D \leq 9s_u + \gamma' D \quad (3)$$

Brinch Hansen (1970)

$$\frac{V}{A} = (\pi + 2)s_u(1.2 + 0.4\,D/B) + \gamma'\, D/B \leq 1 \quad (4a)$$

$$\frac{V}{A} = (\pi + 2)s_u(1.2 + 0.4\tan^{-1}(D/B)) + \gamma' D\; D/B > 1 \quad (4b)$$

Vesic (1975)

$$\frac{V}{A} = (\pi + 2)s_u\left(1 + \frac{1}{\pi+2}\right)(1 + 0.4\,D/B) + \gamma' D\; D/B \leq 1 \quad (5a)$$

$$\frac{V}{A} = (\pi + 2)s_u\left(1 + \frac{1}{\pi+2}\right)(1 + 0.4\tan^{-1}(D/B)) + \gamma' D\; D/B > 1 \quad (5b)$$

where shape factor, $s_c = 1 + \frac{1}{2+\pi}$, v = vertical load, A = footing area, c_u = undrained cohesion, $\frac{V}{A} =$ bearing capacity, D = foundation diameter, B = foundation breadth and $\gamma' =$ submerged unit weight of soil.

Vertical and Horizontal Load (M = 0)

Vesic (1975) expression for a rectangular foundation is given as;

$$\frac{V_u}{A} = (2 + \pi)s_u\left(1 + \frac{B'/L'}{2+\pi}\right)\left(1 - \frac{\frac{2+B'/L'}{1+B'/L'}H}{(2+\pi)s_u B' L'}\right) \quad (6)$$

where B' and L' are the dimension of the fictitious effective area A' of the foundation and v_u is ultimate vertical load. For the case of $B'=L'$, Equation (6) becomes;

$$\frac{V_u}{As_u} = 1.2\left[(2 + \pi) - \frac{3H}{2As_u}\right] \quad (7)$$

and maximum horizontal load (H_o) is given by

$$H_o = As_u = \left(\frac{1}{2+\pi}\right)V_o \quad (8)$$

where sliding failure is incipient at $V/V_o \leq 0.5$.

3. RESULTS AND DISCUSSION

3.1 Wave Characteristics

The meteorological and oceanographic data from the offshore Niger Delta were analysed and a maximum directional wave height, H_{max} of approximately 7.0 m, mean wave period of 17 sec and average wind speed of 14.1m/s were obtained.

3.2 Hydrodynamic Coefficients

Hydrodynamic coefficients, C_d, and C_m, assumed a constant value of 0.7 and 1.5 respectively for wave heights varying from 3.0 – 12.0m, pile diameter of 1.0 – 2.0m, wind speed, u of 12 m/s and kinematic viscosity, υ of 9.5×10^{-7} m^2/s. The dimensionless parameters of inertia and drag forces (K_{im} and K_{Dm}) had constant values for a given wave height on pile diameter ranging from 1.0 – 2.0m. These parameters also reduced with wave height and generally, for a given wave height, K_{im} assumes lower values compared to K_{Dm}. A similar trend is also observed between S_{im} and S_{Dm}.

3.3 Horizontal-Vertical Loading

3.3.1 Square Footing with $D_f/B<1.0$

For $D_f/B<1.0$, the undrained shear strength of soil is generally found to be 2.0kN/m^2. Typical failure envelopes (Figures 1 and 2) of dimensionless plots of V/As_u and H/As_u revealed that a constant V/As_u value of 6.168 occurred at H/As_u equal to 0. Subsequently, V/As_u values decreased with increase in H/As_u and at V/As_u equal to 0, the value of H/As_u is 3.427. When H/As_u exceeded 3.427, negative values of V/As_u occurred. Foundation area used varies from 98-314m^2 while horizontal force varies from 23kN to 938kN. Higher magnitudes of horizontal forces are required to cause sliding when footing area exceeds 113m^2, hence, for stability requirement the maximum vertical load is found not exceeding V= 6.168As_u and the failure envelope lies within $0 \leq V/As_u \leq 6.168$, representing zone of footing stability. Sliding failure commenced when V/As_u assumes negative values (i.e. at $H/As_u > 3.427$) and for footing stability, the vertical load should be at least twice the anticipated horizontal load. The predictive model for horizontal – vertical load case for $D_f/B<1.0$ is given by;

$$\frac{V}{As_u} = -1.8\frac{H}{As_u} + 6.168 \qquad (9)$$

3.3.2 Square Footing with $D_f/B>1.0$

The failure envelope is depicted in Figures 2 and 3 where a maximum vertical load, V of 6.168As_u occurred at H/As_u equal to zero as in the case of $D_f/B<1.0$. Subsequently, V/As_u values decreased with increase in H/As_u. However, within the range of horizontal forces used in this study, V/As_u did not assume negative value. Higher horizontal force of about 10.5 times greater than those for $D_f/B<1.0$ are required to initiate sliding for footing with $D_f/B>1.0$. The predictive model for horizontal – vertical load is also giving by Equation (9).

4. CONCLUSION

Based on this study, the following conclusions can be drawn;

a. For Foundation with $D_f/B<1.0$, a constant value of V/As_u equal to 6.168 occurred when H/As_u is 0.0. Subsequently, V/As_u values decreased with increase in H/As_u and at V/As_u equal to 0.0, the value of H/As_u is 3.427.

b. Sliding failure begins when V/As_u assumes negative values (i.e. at $H/As_u > 3.427$) and for footing stability, the vertical load should be at least twice the anticipated horizontal load.

c. For foundation with $D_f/B>1.0$, higher horizontal force of about 10.5 times greater than those for $D_f/B<1.0$ are required to initiate sliding.

d. The generated models can be used for preliminary design of offshore foundations under combined horizontal-vertical loading in the Niger Delta

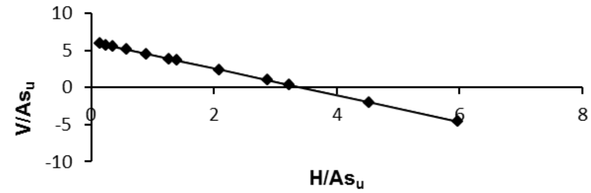

Figure 1: Horizontal- Vertical loading on square footing ($D_f/B<1$, Area=78m^2)

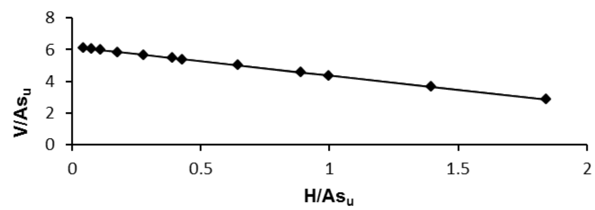

Figure 2: Horizontal- Vertical loading on square footing ($D_f/B<1$, Area=201m^2)

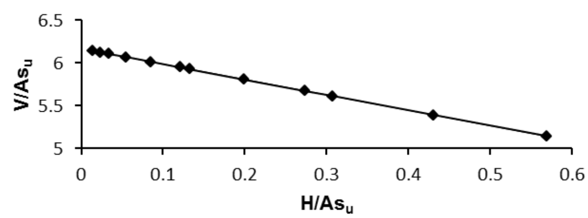

Figure 3: Horizontal- Vertical loading on square footing ($D_f/B>1$, Area=78m^2)

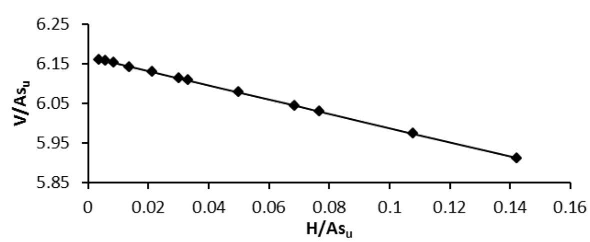

Figure 4: Horizontal - Vertical loading on square footing ($D_f/B>1$, Area=314m^2)

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