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Assessment of buckling of piles due to soil liquefaction for a coal-fired power station Tanjung Jati B, Central Java, Indonesia

Muhammad Hamzah Fansuri[1], M. Chang[2], R. Kusumawardani[3], and P. Purwanta[4]

[1] Planning Engineer, MES-MIS JO – Tanjung Jati Expansion (JAWA-4) Coal-Fired Steam Power Plant, Jepara 59453, Indonesia.
[2] Professor, Dept. of Civil and Cons. Engrg., Nat'l Yunlin U. of Sci. & Tech. No. 123, Sec. 3 University Rd, Yunlin, Taiwan 640, ROC.
[3] Assoc. Prof., Soil Mechanics Laboratory, Dept. of Civil Engineering, Universitas Negeri Semarang, Semarang 50229, Indonesia.
[4] Planning Manager, MES-MIS JO – Tanjung Jati Expansion (JAWA-4) Coal-Fired Steam Power Plant, Jepara 59453, Indonesia.

ABSTRACT

Indonesia is facing rapid growth in economy and results in increasing demand for electricity. To supply sufficient electricity, Indonesian Government has constructed a coal-fired power station (CFPS) Tanjung Jati Units 5 & 6 with a capacity of 2 x 1000 MW. The site of this study is in Tubanan Village, Kembang District, Jepara Regency, Central Java Province. Development planning of the project should consider the risk caused by earthquakes. Accordingly, the potentials for liquefaction of foundation soils and buckling of piles due to loss of lateral support in liquefied soils become the main concerns of this project. Liquefaction analysis of foundation soils is performed based on a SPT-N approach. A depth-weighted procedure is also applied for assessment of liquefaction potential for the boreholes of the site. Results of liquefaction assessment indicate the project site is prone to the risk of soil liquefaction due to the design earthquake. Buckling of piles subjected to seismic loading is assessed for the cases of liquefied soils in both dry and wet seasons, while only the wet season scenario is the main focus of this paper. A buckling index G is adopted as the difference between the critical pile length (H_C) for buckling and the unsupported pile length (D_L) due to liquefaction of foundation soils. If the G is greater than zero, then the pile is considered safe; otherwise, the pile will buckle. Results of buckling assessment show $G > 0$ for the piles at the project site (Units 5 & 6 and Central Control Building) with an average G value of about 15 when the foundation soils are liquefied during the design earthquake, indicating the pile foundation of the site should be safe from buckling failure due to soil liquefaction.

Keywords: soil liquefaction; pile buckling; assessment; pile foundation; seismic loading

1 INTRODUCTION

Indonesia is known as one of the most seismically active countries in the world. The country is surrounded by three major active tectonic plates, namely, Eurasian, Indo-Australian, and Philippine Plates.

Central Java Province situating in an area adjacent to the plate collision is prone to tectonic earthquakes. An earthquake with a magnitude of 5.9 SR (M_b) or 6.3 (M_w) occurred in the Special Region of Yogyakarta and Central Java on May 27, 2006, with its center at 8.03°(S) / 110.32°(E) and a depth of 11.8 km. Soil liquefaction has been observed in several locations in the north of Sleman and Klaten (Unjianto, 2006).

Historical review of the major earthquakes includes Jepara earthquake of 1821 with a Modified Mercalli Intensity (MMI) of VI-VII at the project site. According to McBirney et al. (2003), Pati earthquake of 1880 with a magnitude of 6.8 occurred at a distance of 45-50 km away and was estimated with a MMI scale of VII at the project site.

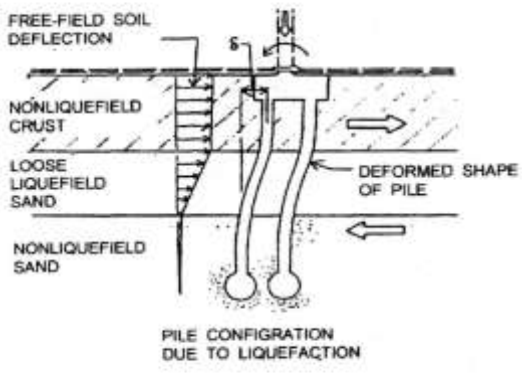

Fig. 1. Potential failure mode of piles due to seismic loading and soil liquefaction (Finn and Thavaraj, 2001).

Planning engineer needs to take into account the risks caused by seismic loading. These risks not only include the failure of building structures, but also the failure in the foundation soils that support the buildings above it. As illustrated in Fig. 1, the underlying sandy soils may liquefy during earthquake shaking, losing their shearing resistances, triggering lateral spreading or dragging on the upper non-liquefied crust, which would further cause bending or failure of the piles (Finn and Thavaraj 2001; Madabhushi et al. 2001). The effect of liquefied foundation soils might also lead to insufficient friction resistance and hence axial bearing failure of the piles. In addition, insufficient lateral supports of soils around the pile during liquefaction would result in an unexpected buckling failure of piles under the dynamic axial loading (Bhattacharya et al. 2005). Accordingly, the aim of this study is to examine the liquefaction potential of the foundation soils as well as the buckling stability of the pile foundation for the subject coal-fired power station.

2 LITERATURE REVIEW

Structure design with column buckling and beam bending can be approached in different ways. Design against bending or axial failures of piles, however, would not necessarily suffice the requirements for avoiding buckling (Bhattacharya et al. 2005). When ground shaking starts, the excess pore water pressure gradually increases, which will in turn decrease the effective stress in the soil. As the effective stress of soil approaches to zero, the soil will loss its strength and liquefaction occurs. Hence, the confining stress of soil around the pile will decrease drastically, and eventually might lead to bucking of the piles.

2.1 Liquefaction potential index (LPI)

A simplified SPT-N based method by Youd et al. (2001) is adopted for the liquefaction assessment of the foundation soils. The assessment is performed with a design earthquake for the site, where the magnitude ($M_w = 6.8$) and the peak ground acceleration (PGA; $a_{max} = 0.210g$) are selected based on the information provided by Ministry of Public Works, Indonesia (2011). Two groundwater senarios are considered, i.e., dry and wet groundwater tables, which are based on longterm monitoring data at the site. In the liquefaction assessment, the cyclic stress ratio (CSR) due to seismic shaking is estimated by Eq. (1):

$$CSR = 0.65(\frac{a_{max}}{g})(\frac{\sigma_v}{\sigma'_v})r_d \tag{1}$$

The cyclic resistance ratio of the soils, per Youd's approach, is determined by Eq. (2):

$$CRR = \left(\frac{1}{34-N_{1,60,FC}} + \frac{N_{1,60,FC}}{135} + \frac{50}{(10N_{1,60,FC}+45)^2} - \frac{1}{200}\right)MSF \tag{2}$$

where $N_{1,60,FC}$ is the SPT blow count normalized to an overburden pressure of 1 atm, a hammer efficiency of 60%, and with the correction of fines content (Youd et al. 2001). MSF is the magnitude scaling factor for the adjustment of an earthquake magnitude of 7.5 to the manitude of design earthquake of the site.

The factor of safety (F_L) against liquefaction can be thus evaluated by Eq. (3):

$$F_L = CRR/CSR \tag{3}$$

A depth-weighted procedure by Iwasaki et al. (1982) is then applied to assess the liquefaction potential index (LPI) of the entire borehole up to a depth of 20 m:

$$LPI = \int_0^{20m} F \cdot w(z)dz = \int_0^{20m} F \cdot (10 - 0.5z)dz \tag{4}$$

where $F = 1 - F_L$ if $F_L \leq 1.0$; otherwise, $F = 0$.

2.2 Unsurpported pile length (D_L) due to liquefaction

The unsurpported length of piles (D_L) indicates the extent along the pile where its lateral confining stress decreases significantly as a result of liquefaction of the surrounding soils. The unsurpported length of piles is assessed based on the thickness of liquefied soil layers plus additional distances necessarily for the fixity into the upper or lower non-liquefied soil layers. The fixity is typically 3~5 pile-diameters (Bhattacharya and Goda 2013). In this study, we adopt a fixity of 5 diameters of the pile.

2.3 Critical pile length (H_C) for buckling

The critical pile length (H_C) describes the minimum length that the pile will buckle due to axial loading based on Euler's theory. The critical length of piles is evaluated by considering the static and dynamic axial loads as well as the boundary conditions of the pile. The dynamic axial load ($P_{dynamic}$) is equal to the static axial load (P_{static}) plus additional load on the piles due to seismic shaking on the superstructure, and is given by Eq. (5):

$$P_{dynamic} = P_{static} + \Delta P = (1 + \alpha)P_{static} \tag{5}$$

where α is the dynamic axial load factor - a function of the type, geometry and mass of superstructure and the characteristics of earthquake shaking.

By assuming the actual buckling load equals 0.35 times the Euler's value and the dynamic axial load equals the actual buckling load, the critical length of piles can hence be evaluated by Eq. (6):

$$H_C = \sqrt{\frac{0.35\pi^2 EI}{K^2 P_{dynamic}}} = \sqrt{\frac{0.35\pi^2 EI}{K^2(1+\alpha)P_{static}}} \tag{6}$$

where EI is the bending stiffness of the pile and K is the effective column length factor depending on the boundary conditions of the pile. In this study, a K value of 1.0 is adopted in viewing that the pile head is fixed to the superstructure and the pile tip is embedded into the hard layer.

A buckling index G is adopted as the difference between the critical pile length (H_C) for buckling and the unsupported pile length (D_L) due to liquefaction of foundation soils, as indicated by Eq. (7):

$$G = H_C - D_L \tag{7}$$

If the G is greater than zero, then the pile is considered safe; otherwise, the pile will be buckling due to seismic loading and soil liquefaction.

3 CASE STUDY

The location of Tanjung Jati B Coal-Fired Power

Station Units 5 & 6 and the Central Control Building is located in Tubanan Village, Kembang District, Jepara Regency, Central Java Province, approximately 32 km to the north of Jepara. Results of subsurface exploration at the site generally show the foundation soils consist of soft sandy silts or clays interbedded with loose fine sands to a depth of about 9 m, and the deeper strata will be stiff to hard clayey soils.

The liquefaction potential of the site is evaluated based on a SPT-N approach by Youd et al. (2001) in association with a depth-weighted method by Iwasaki et al. (1982), for 16 boreholes that cover 3 main facilities (Units 5 & 6 and Central Control Building or CCB) of the site. Results indicate the site is prone to liquefaction due to the design earthquake, with more than 50% of the boreholes computed showing liquefaction potential indices ($LPIs$) of greater than 5 (i.e., high to very high liquefaction potential). Fig. 2 shows LPI contour plots for the groundwater senario during the wet season, indicating the areas with high liquefaction potential generally fall in the center and southern parts of the site (CCB and Unit 6) and to the north and east boundaries of Unit 5 (northern part of the site).

It is noted, however, the above assessment is based on the SPT data performed prior to the installation of the piles. The piles are formed by precast reinforced concrete with outside and inside diameters of 600 and 400 mm, respectively. The average pile length in Units 5 & 6 is 18 m, while in CCB is 12 m. Since foundation soils would have been compacted after pile installation, the above assessment is considered conservative on the computed liquefaction potential for the project site.

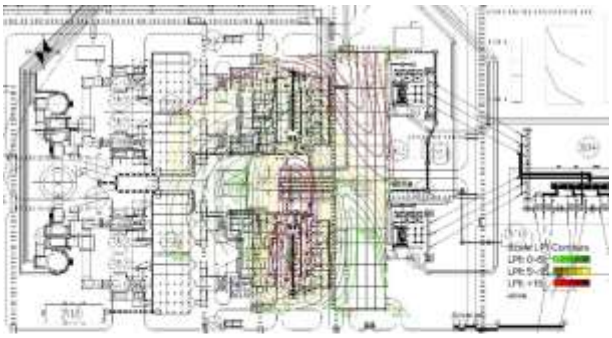

Fig. 2. LPI contours for groundwater senario in wet season.

3.1 Dynamic axial load of piles

In analyzing dynamic axial loads of pile ($P_{dynamic}$) as indicated in Eq. (5), we need the information of dynamic axial load factor α, which is a function of the type, dimension and mass of the superstructure, the characteristics of seismic shaking, as well as material properties and geometric factors of the pile foundation.

To estimate the additional dynamic axial load (ΔP) on each of the piles, the acting moment due to seismic shaking on the superstructure has to be computed, then the moment can be distributed onto all of the resisting piles and the additional dynamic axial load on each of the piles can therefore be calculated.

According to BIS (2002), the seismic base shear of the superstructure is computed by:

$$V_B = C_s W \tag{8}$$

where W is the building weight and C_s is the seismic response coefficient, for which C_s can be calculated by the four parameters: (1) Z - zone factor (0.16; Prakash 2004; Bhatia et al. 1999), (2) I - importance factor (1.5 for coal-fired power plants), (3) R - response reduction factor (4.0 for steel frame with concentric braces), and (4) S_a/g - response spetrum acceleration coefficient. The spectrum acceleration can be estimated based on the fundamental period of the structure.

For the post-liquefaction situation, the fundamental period of the structure can be calculated by:

$$T_{post} = 2\pi\sqrt{\frac{W/g}{N_p \times 12EI/{D_E}^3}} \tag{9}$$

where N_p is the total number of piles of the building, and D_E is the depth to the lower boundary of liquified soil layer plus an additional fixity. With the calculated fundamental period, the spectrum acceleration, and the base shear of the building as well, due to the design earthquake can then be obtained based on the design spectrum for the case of soft soil sites (BIS 2002).

To compute seismic moment on the superstructure, the arm where the inertial force acts can be assumed by the following:

$$ARM_{post} = D_E + \beta_3 H \tag{10}$$

where H is the height of the building and β_3 is the coefficient to account for the effective height where the inertia acts (typically, 0.5). With the base shear and the moment arm, the acting moment can be computed as:

$$M_{post} = V_B \ ARM_{post} \tag{11}$$

The overall moment is then distributed to all of the resisting piles of the superstructure for computing the additional dynamic axial load on each of the piles. To do this, the utmost dynamic load for piles located at the peripheral boundary of pile foundation needs to be calculated first, and then the additional dynamic loads of the inner piles can be estimated by assuming the dynamic load is proportional to the distance between the pile of concern and the axis of symmetry of the foundation area.

By assuming a rectangular arrangement of the piles (n rows $\times$ m columns) and the acting moment in the direction of row, the additional dynamic axial loads of the peripheral and inner piles can thus be computed respectively by:

$$\Delta P_{max} = \frac{x_{max} \ M_{post}}{2n\left(\sum_{i=1}^{max} x_i^2\right)} \tag{12}$$

$$\Delta P_i = \left(\frac{x_i}{x_{max}}\right)\Delta P_{max} \tag{13}$$

where x_{max} and x_i are the distances of the peripheral and inner piles, respectively, to the axis of symmetry of the foundation area. Finally, the dynamic axial load ($P_{dynamic}$) can then be calculated by Eq. (5).

3.2 Critical pile length (H_C)

With the evaluated dynamic axial loads, Eq. (6) can hence be applied to estimate the critical length for each of the piles. Fig. 3 shows results of H_C computations for 47 representative piles that cover 3 main facilities (Units 5 & 6 and CCB) of the site, indicating the minimum lengths that the on-site piles will buckle are all greater than 18 m, when the foundation soils are liquefied due to the design earthquake in wet season.

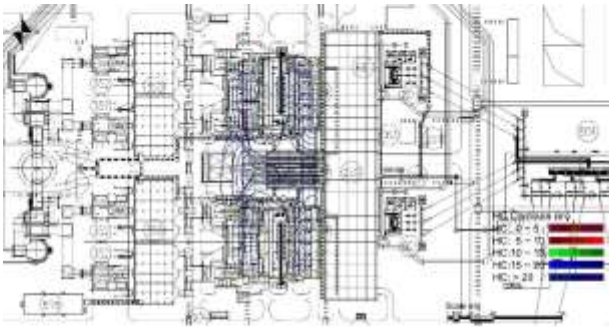

Fig. 3. H_c contours (post-liquefaction & wet season senario).

3.3 Buckling index (G)

Based on above analyses on the unsupported pile length (D_L) and the critical pile length (H_C), the buckling index (G) can be assessed to determine if the piles are adequate in resisting the buckling instability due to soil liquefaction by the design earthquake. As stated previously, a G value of greater than zero signifies the pile is safe; otherwise, the pile will be buckling. As shown in Fig. 4, the project site (Units 5 & 6 and CCB) will be safe from buckling instability, with calculated G values of greater than 10 (m), and an average G value of 15 (m), for all of the piles assessed at the project site.

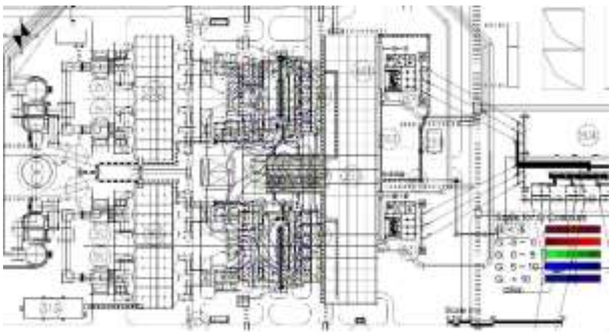

Fig. 4. G contours (post-liquefaction & wet season senario).

4 CONCLUSION

This paper discusses assessments of the liquefaction potential of foundation soils and the buckling instability of foundation piles for a coal-fired power station in Indonesia. Major findings are summarized as follows:

(1) Liquefaction analysis indicates the foundation soils are prone to liquefaction due to design earthquake, with $LPI > 5$ (high liquefaction potentials) for more than 50% of the boreholes assessed, although studies of the adjacent nuclear power station show the opposite (McBirney et al. 2003). It is noted, however, the SPT data adopted for liquefaction assessment was performed prior to pile installation. The compaction effect of soils due to pile installation is not considered and hence the assessment would be conservative on the liquefaction potential of the site.

(2) Buckling assessment indicates the foundation piles will be safe from buckling failure, with computed critical pile lengths (H_C) of greater than 18 m and buckling indecis (G) of greater than 10 (m), due to soil liquefaction by the design earthquake, for the piles assessed at the project site.

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