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Settlement of axially loaded piles entirely embedded in rock – analytical and experimental study

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Finite element analysis was performed to study the settlement behaviour of axially loaded piles entirely embedded in nonhomogenous rock. The elastic modulus of the rock mass was taken to increase linearly along the pile length starting from a nonzero value at the ground surface. Cases of pile-rock stiffness ratios that have not been considered in the literature were investigated. Such ratios are typical for reinforced concrete piles bored in sedimentary rocks. Results of the numerical analysis were verified through conducting static loading tests on full-scale piles in rock of well-defined physical and mechanical properties. Charts were developed to predict the elastic settlement of piles in rock. An equation was also introduced to incorporate the effect of rock nonhomogeneity in estimating the depth at which settlement becomes insensitive to the increase of pile length. A complementary numerical analysis utilizing a simple piled foundation system showed that the pile load share is sensitive to the rate of increase in rock stiffness along the pile length. The sensitivity is more pronounced for piled foundations resting on rock with high mass stiffness.

Keywords: elastic analysis; piles in rock; pile load share; pile settlement; piled raft; rock stiffness

1. Introduction

Structural design and settlement of piled foundations are influenced by the compressibility of the foundation bearing medium and the stress-settlement behaviour of piles. In the last two decades, several studies have been conducted on such soil-structure interaction especially for piled rafts (e.g., Russo and Viggiani 1998, Poulos 2001, Prakoso and Kulhawy 2001, El-Mossallamy et al. 2003, Sanctis and Mandolini 2006, Sanctis and Russo 2008, Eid 2011). In piled rafts, superstructure loads are shared by the raft through contacting the underlying ground and the piles through skin friction and/or end bearing. Load sharing can be more pronounced in piled rafts resting on extended rock where the raft base is in direct contact with the rock. The rock mass usually exhibits a considerable stiffness near the pile head and an increasing stiffness along the pile length.

Pile settlements at working loads have been commonly estimated on the basis of elastic theory as it has been shown by field tests that load-settlement curves are closely linear up to half of the peak capacity (Williams and Pells 1981). As a result, a pile spring constant is usually needed as an input for most of the commercially available software utilized to design piled foundations. Since pile loading tests, that can be used for a reliable assessment of pile elastic settlement, are usually conducted after the foundation design stage, the current practice in design of piled foundations is based on a crude estimation of the pile spring constant or elastic settlement behaviour.

Several studies have been conducted to predict the elastic behaviour of piles in homogenous rock through field testing and laboratory modelling (e.g., Carrubba 1997, Zhan and Yin 2000, Kim 2001, McVay and Niraula 2004, Ooi et al. 2004). Settlement of piles in homogenous rock or soil has been studied analytically assuming a linearly elastic half-space (e.g., Mattes and Poulos 1969, Poulos and Davis 1980) and utilizing simple but approximate closed-form solutions (e.g., Murff 1975, Randolph and Wroth 1978, Fleming et al. 1992, Kodikara and Johnston 1994, Motta 1994). Two-dimensional (2D) finite element (FE) approaches have been also used to estimate the settlement of piles in homogenous media (e.g., Osterberg and Gill 1973, Pells and Turner 1979, Donald et al. 1980). Conversely, the available investigations of the elastic settlement of piles in nonhomogenous media are few and mostly based on assuming a stiffness modulus that starts at zero near the pile head and increases linearly with depth (e.g., Banerjee and Davies 1977, Poulos 1979, 1989) or as a power of depth (e.g., Guo and Randolph 1997).

Based on a closed-form solution presented by Randolph and Wroth (1978), Fleming et al. (1992) developed charts to estimate the elastic settlement of piles in a soil with nonzero shear modulus at the ground surface and stiffness that increases linearly with depth. Only high pile subgrade stiffness ratios, that
are not applicable for piles in rock, were utilized in developing such charts. A closed-form solution was also presented by Guo (2000) to account for nonhomogeneity of soil with nonzero shear modulus at the ground surface and stiffness that increases as a power of depth. A more rigorous continuum-based numerical analysis, using a 2D finite difference program was used to verify the suggested solution. However, no charts were presented to predict the pile elastic settlement in terms of pile dimensions and pile-soil stiffness ratio. The lack of these charts along with the difficulty of the calculations involved has limited the use of such solution.

Mayne and Niazi (2009) also utilized a closed-form solution to introduce an approach for evaluating the axial elastic pile response from cone penetration tests. Results showed that the responses yielded from using this approach are in agreement with those previously reported in the literature for three pile loading tests. However, the use of such an approach is restricted to piles in soil because cone penetration tests are not executable in rock.

The main purpose of the work presented in this paper is to introduce a simple and reliable technique to predict the elastic settlement behaviour of piles entirely embedded in rock. This work is different from the previous studies that dealt with elastic settlement of piles in rock or soil in four main aspects: (1) utilizing a FE analysis that considers a nonzero rock stiffness near the pile head and an increase in stiffness of rock mass with depth; (2) introducing charts for estimating settlement in cases of typical pile-rock stiffness ratios that have not been covered in the literature for piles in nonhomogenous media; (3) verifying the numerical analysis results not only through comparisons with data presented in the literature based on analytical investigations, but also with results of static loading tests conducted for this study on full-scale piles entirely embedded in rock the physical and mechanical properties of which are well defined; and (4) performing a sensitivity study to show the importance of considering the change of rock stiffness along the pile length on the design of piled foundations.

2. Numerical analysis

Numerical analysis was carried out using SAP2000 three-dimensional FE software package to study the elastic behaviour of axially loaded piles in rock. One meter diameter piles with lengths (L) of 2, 4, 6, 10, 20, and 30 meters were considered in the analysis. Physical and mechanical properties of the piles and surrounding rock used in the analysis are shown in Figure 1. The elastic modulus of rock mass was taken to increase linearly along the pile length until it reaches a value of $E_m$ at the pile base and then continues to be constant downward. The modulus of rock mass below the pile base was considered to be constant because the stress bulb beneath the base – even for bearing piles- typically extends down about twice the pile diameter (Fleming et al. 1992). Assuming an increase in the modulus of rock mass in such short depth is not practical.

Poisson’s ratio for rock ($\nu_r$) and piles ($\nu_p$) were taken as 0.25. Value of Poisson’s ratio usually ranges from 0.1 to 0.3 and from 0.15 to 0.3 for rock and concrete, respectively. Variation in Poisson’s ratio of rock and concrete within these ranges has very little effect on elastic settlement of piles (Pells and Turner 1979). The ratio between the modulus of rock mass near the pile head and that at the base ($b$) was taken to be 0.2, 0.4, 0.6, 0.8, and 1.0. Pile-rock relative stiffness was expressed as the ratio of the pile Young’s modulus $E_p$ and the base level rock mass Young’s modulus $E_m$, i.e., $E_p/E_m$. Values of 10, 20, 50, 100, 200, and 500 were assigned to the $E_p/E_m$ ratio. Such values are typical for reinforced concrete piles bored in sedimentary rocks. However, no data are available in the literature on the elastic settlement of piles in nonhomogenous media with $E_p/E_m$ less than 100.

The three-dimensional (3D) FE mesh used in the analysis is shown in Figure 2. Eight-node solid elements were used to represent piles and rock. No special elements were utilized at the pile/rock interface because studying pile ultimate bearing capacity and consequently any relative movement at such interface is outside the scope of this research. To study the behaviour of side-resistance only piles, a gap with $E_p/E_m$ of 3000 was utilized to simulate rock immediately below the pile base and create a practically no-base support. The mesh shown in Figure 2 has been chosen after performing several refinement studies to avoid any mesh-size effect. Owing to the symmetry of the considered configuration, 2D axisymmetric analysis could have been used in this study. However, a 3D full domain mesh was utilized to be consistent with the technique used for the sensitivity study presented by the end of this paper. Capabilities of the utilized software and computers also helped in adopting such 3D mesh without a need for simplification.
Settlement values yielded from the numerical analysis were expressed in terms of the settlement influence factor ($I_p$) using the following equation:

$$ S = \frac{Q}{DE_m} I_p $$

(1)

where $S = $ settlement of pile head under a vertical load $Q$ and $D = $ pile diameter. Equation (1) has been used by several researchers (e.g., Pells and Turner 1979) to estimate the elastic settlement of isolated shear sockets at the surface of a semi-infinite elastic rock. The equation considers that the effects of end bearing and drilled shafts compression on the settlement of pile head are negligible. Such consideration can be valid for piles analyzed in this study since the results of several field tests and numerical simulations (e.g., Horvath et al. 1980, Carrubba 1997) have shown that the contribution of toe resistance is only significant at relatively large displacements, i.e., at the ultimate limit state. Consequently, under the working loads, forces transferred from the pile shaft to the surrounding rock through skin friction are the main contributor to the settlement of pile head.

Table 1 presents values of $I_p$ determined for side-resistance-only and side-resistance and bearing piles using the limit $E_p/E_m$ values and slenderness ratios ($L/D$) utilized in this study. It can be seen that the reduction in pile settlement, i.e., the reduction in $I_p$, due to the mobilization of end bearing, increases with decreasing $b$ and $L/D$ values, and increasing $E_p/E_m$ ratio. However, such reduction is insignificant and does not exceed 11% for all of the considered cases. Similar conclusion can be drawn from the data presented by Mattes and Poulos (1969), Pells and Turner (1979) and Donald et al. (1980) for piles entirely embedded in rock of constant mass modulus. As a result, no differentiation was made between the two pile types in presenting their predicted behaviour throughout the rest of this paper.

The rate of increase in the modulus of rock mass along the pile length, expressed in terms of $b$, influences the settlement of pile and consequently the calculated values of $I_p$. Such values are shown in Figure 3 as a function of the pile-rock stiffness and pile slenderness ratio. It can be seen that $I_p$ has a general tendency to decrease with increasing values of $b$, $L/D$, and $E_p/E_m$. However, the rate of such decrease diminishes for piles with high $L/D$ and $E_p/E_m$ ratios. Increasing pile slenderness ratios to values greater than a critical one, $(L/D)_c$, does not affect the pile head stiffness or settlement since almost no load reaches the pile lower end. Analyzing the data developed from this study and those presented in the literature shows that the magnitudes of $(L/D)_c$ increases with decreasing $v_r$ and increasing $E_p/E_m$ and $b$ values. Fleming et al. (1992) suggested an equation to approximately estimate the value of $(L/D)_c$ in terms of $v_r$ and $E_p/E_m$ only. This was based on studying the settlement behaviour of single piles with slenderness ratios as high as 100 socketed in rock with $v_r$ of 0.3, $E_p/E_m$ of 115, 385, 1154, and 3846, and $b$ of 0.25, 0.5, and 1.0.

To include the effect of increasing the modulus of rock mass with depth in estimating the magnitude of $(L/D)_c$, data yielded from this study and those available in the literature were reanalyzed. This has resulted in developing the following equation:

$$ \left( \frac{L}{D} \right)_c = 2b \left[ 2(1 + v_r) \frac{E_p}{E_m} \right]^{0.5} $$

(2)

Equation (2) is similar to that introduced by Fleming et al. (1992) but modified to incorporate the effect of rock nonhomogeneity, i.e., to include $b$. Determining the magnitude of $(L/D)_c$, helps in choosing the optimum pile length and designing pile rafts that are commonly used – as an alternative to rafts on grade- for settlement reduction.

It should be noted that for piles with slenderness ratios greater than $(L/D)_c$, using Equation (1) in calculating values of $I_p$ based on the settlements determined from the FE analysis leads to spurious results. Increasing $L/D$ values in such range

<table>
<thead>
<tr>
<th>$L/D$</th>
<th>$E_p/E_m$ = 10</th>
<th>Side-resistance-only</th>
<th>500</th>
<th>Side-resistance and bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b$</td>
<td></td>
<td></td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.593</td>
<td>0.470</td>
<td>0.457</td>
<td>0.367</td>
</tr>
<tr>
<td>30</td>
<td>0.543</td>
<td>0.377</td>
<td>0.110</td>
<td>0.079</td>
</tr>
<tr>
<td>0.2</td>
<td>0.550</td>
<td>0.446</td>
<td>0.407</td>
<td>0.337</td>
</tr>
<tr>
<td>1.0</td>
<td>0.543</td>
<td>0.377</td>
<td>0.109</td>
<td>0.078</td>
</tr>
</tbody>
</table>
is associated with an ineffective increase in $E_n$ that leads to a rise in the calculated $I_p$ values, even though the actual settlement is essentially constant. Similar trend can be noticed in data presented by Poulos and Davis (1980) and Guo and Randolph (1997) for piles embedded in uniform soils. These unrealistic values of $I_p$ were not utilized in developing the figures of the current study.

The influence of changing the values of $b$ on settlement of single pile is presented in Figure 4. It can be seen that, in case of having $E_p/E_m$ of 100 as a typical value, changing $b$ from 0.2 to 1.0 decreases the settlement by ratios of 22% and 30% for piles with $L/D$ of 4 and 30, respectively. This significant reduction shows the importance of considering the change in the stiffness of rock mass along the pile shaft specially in estimating settlement of long piles.

For verification, results of the presented FE analysis were compared with the elastic settlement data obtained by Pells and Turner (1979) and Donald et al. (1980) for a single compressible pile in homogenous elastic continuum (Figure 5). Both of these studies are based on two-dimensional FE analysis and assigning a constant mass modulus for rock surrounding the pile, i.e., $b = 1.0$ using the terminology of the present paper. Results of the numerical analysis were also compared with the

![Figure 3. Elastic settlement influence factors as a function of pile and rock parameters.](image)

![Figure 4. Effect of changing $b$ on the values of $I_p$.](image)
3. Field testing

Two static loading tests on full-scale piles were conducted for further verification of the numerical analysis results presented in this study. The testing was performed in a selected study area at Doha, capital of the state of Qatar. The typical subsurface profile in this area consists of a thin layer of sand underlain by an extended layer of fractured rock (Figure 7). The rock belongs to the Upper Dammam formation of the Middle Eocene age. Cavalier et al. (1970) described such rock as a fine to medium grained off-white, poorly bedded, chalky crystalline calcareous and dolomitic limestone with numerous irregular joints often filled with weathered siltstone. The limestone – in which the test piles were embedded – is moderately weathered.
and has a bimodal nature comprising predominantly dolomite and a variable percentage of secondary material. The engineering properties of the limestone are influenced by the percentage and type of the secondary material which is silt for the top five meters and clay at larger depths (Eid and Alansari 2004). The limestone layer is frequently underlain by Midra shale of the Lower Dammam formation. Midra shale consists mainly of attapulgitic clay with very high plasticity (Eid 2006).

Two boreholes with a depth of 22 meters were carried out at the pile testing location. Rock quality designation (RQD) was determined along the depth of each borehole. Unconfined compression tests on limestone specimens extracted from different depths were conducted to determine the strength and Young’s modulus of the intact rock ($q_u$ and $E_{lab}$, respectively). In situ pressuremeter tests were used to measure the rock mass modulus ($E_{max}$) at different depths. No significant change was detected in rock properties determined for the two boreholes. As a result, average properties were used to describe the limestone of the testing location. These properties —along with a representative borehole log— are shown in Figure 8.

For typical construction activities that have been taken place in the study area, RQD is the only parameter used in assessing rock mass quality. However, other classification schemes such as the Rock Mass Rating (RMR) and the Geological Strength Index (GSI) are used in this study to assess the quality of limestone at the testing location. Table 2 shows the results of such assessment for rock along the depth of the boreholes. It can be seen that the estimated values of the RMR parameter ranged between 38 and 61. Consequently, the limestone can be described as a poor to fair rock based on the RMR system developed by Bieniawski (1989).

Based on the results of the unconfined compression tests conducted in this study, values of the modulus ratio (MR) for the limestone were calculated and shown in Table 2. Most of such values fall below the ranges reported by several researchers for similar rock type (e.g., Hoek and Diederichs 2006). The

![Figure 8. Rock properties at the test area.](image-url)
Table 2. Properties of limestone along the depth of boreholes

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>RQD (%)</th>
<th>MR$^a$</th>
<th>$E_{\text{max}}$ (GPa)</th>
<th>RMR$^c$</th>
<th>GSI$^d$</th>
<th>$E_{\text{mass}}$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5</td>
<td>29</td>
<td>336</td>
<td>0.35</td>
<td>38</td>
<td>33</td>
<td>0.31</td>
</tr>
<tr>
<td>5.8</td>
<td>48</td>
<td>395</td>
<td>0.51</td>
<td>47</td>
<td>42</td>
<td>0.53</td>
</tr>
<tr>
<td>9.3</td>
<td>63</td>
<td>211</td>
<td>0.64</td>
<td>52</td>
<td>47</td>
<td>0.65</td>
</tr>
<tr>
<td>12.8</td>
<td>70</td>
<td>140</td>
<td>0.60</td>
<td>57</td>
<td>52</td>
<td>0.66</td>
</tr>
<tr>
<td>18.7</td>
<td>79</td>
<td>134</td>
<td>0.72</td>
<td>61</td>
<td>56</td>
<td>1.14</td>
</tr>
</tbody>
</table>

$^a$Defined as $E_{\text{lab}}/q_u$ (see Fig. 8 for measured values of $E_{\text{lab}}$ and $q_u$).
$^b$Measured using pressuremeter.
$^c$Estimated using the rock mass rating system presented by Bieniawski (1989).
$^d$Estimated using relation with $RMR$ suggested by Hoek and Brown (1997).
$^e$Estimated based on equation introduced by Hoek and Diederichs (2006).

deviation from the reported ranges increases with increasing the depth of the tested limestone. This can be interpreted in terms of the limestone bimodal nature described above. Increasing the percentage of the attapulgitic clay -as a secondary material of the limestone- with depth resulted in reducing the $MR$ values to be close to those of claystones or shales. The frequent encountering of shale layers beneath the studies limestone supports this interpretation.

Using the measured $E_{\text{lab}}$ values along with the calculated GSI ones, $E_{\text{mass}}$ of the limestone at different depths was estimated utilizing the equation developed by Hoek and Diederichs (2006). The disturbance factor needed to be used in such equation was assigned a value of 0 since the rock around the boreholes was not disturbed. The estimated $E_{\text{mass}}$ values as well as those measured using the in-situ pressuremeter are also listed in Table 2. It can be seen that, except for rock near the bottom of the boreholes, the estimated and measured $E_{\text{mass}}$ values are in good agreement. Such agreement supports the reliability of the conducted field measurements. As a result, the measured $E_{\text{mass}}$ values were adopted to represent the rock mass stiffness in this study. A linear increase of rock mass stiffness with depth was utilized for theoretical estimation of the piles elastic behaviour under axial loading. Such approximation reasonably represents the change in the measured values of rock mass stiffness with depth (Figure 8).

Test piles were drilled with bucket auger under water. Cages of high-strength steel bars were then placed followed by pouring concrete using the conventional tremie method. Concrete with an average 28 day-curing strength of 45 MPa was used. A sonic test was carried out on each test pile five days after concreting. The results verified the geometry and concrete homogeneity of the test piles.

Pile loading tests were carried out in accordance with ASTM D1143 (2007). Two cycles of loading were applied to each pile. In the first cycle, pile was loaded to the design load. After complete unloading, pile was reloaded to 1.5 times the design load. The loading and unloading processes were performed in increments of a quarter of the design load. Design loads were calculated based on the piles structural capacities to be 1.5 and 5.1 MN for test piles with diameters of 0.6 m and 0.9 m, respectively. A hydraulic jack was used to load the piles against a reaction system consists of anchors socketed in rock.

The arrangement used for pile testing is shown in Figure 9. Configurations and loads of tested piles along with the number and length of the reaction anchors are presented in Table 3.

Load-settlement curves resulted from the pile testing are presented in Figure 10. It can be seen that the two tests yielded a nearly elastic behaviour, even at the maximum testing loads.

![Figure 9. Arrangement of pile loading tests.](image)

Table 3. Configurations and loads of tested piles

<table>
<thead>
<tr>
<th>Diameter (“D”) (m)</th>
<th>Length (“L”) (m)</th>
<th>Reinforcement</th>
<th>Maximum Load (MN)</th>
<th>Reaction Anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1st cycle</td>
<td>2nd cycle</td>
</tr>
<tr>
<td>0.6</td>
<td>14</td>
<td>8 bars of 16 mm dia. $^a$</td>
<td>1.5</td>
<td>2.25</td>
</tr>
<tr>
<td>0.9</td>
<td>8</td>
<td>9 bars of 25 mm dia. $^a$</td>
<td>5.1</td>
<td>7.65</td>
</tr>
</tbody>
</table>

$^a$Main reinforcement in a full-length cage with 10 mm-diameter spiral stirrups of 200 mm pitch.
Figure 10. Load-settlement curves for the tested piles (elastic compression lines determined using the FE analysis are superimposed).

Permanent displacements were less than 12% and 18% of the displacement at maximum load of the first and second cycles, respectively. A similar elastic behaviour up to loads close to half the peak bearing capacity was reported by several researchers for piles socketed in rock (e.g., Williams 1980, Santiago and Fragio 1986, Carrubba 1997, Ooi et al. 2004).

Elastic responses estimated using the FE analysis (i.e., using Figure 3) are also shown in Figure 10 as the elastic compression lines. Parameters needed to represent variation of rock mass modulus along the pile length were taken from Figure 8 as $E_m$ and $b$ of 0.65 GPa and 0.59 for the 0.6 m diameter pile, and 0.54 GPa and 0.75 for the 0.9 m diameter pile. An average pile elastic modulus of 31 GPa was assigned to the tested piles. Such value was calculated using a formula proposed by Zhan and Yin (2000) to estimate the elastic modulus for the composite material of steel and concrete as

$$E_p = E_c (1 - r) + E_s r$$

(3)

Where $E_c$ and $E_s$ are the elastic moduli of concrete and steel, respectively; and $r$ is the percentage of steel reinforcement.

Figure 10 shows that the numerical simulation presented in this study reasonably predicted the relationship yielded from loading the test piles up to the design load. Such good prediction verifies the theoretical study results and validates the use of the charts presented in this paper for estimating settlement of single piles in rock.

4. Sensitivity study

To investigate the effect of considering the increase in rock stiffness along the pile length—and consequently utilizing the results of the current research—on design of piled foundations, a simple foundation system was analyzed (Figure 11).
The system consists of two piles entirely embedded in rock to support a concrete foundation with different foundation thicknesses ($t$). The study was conducted using the same 3D modelling technique, method of analysis, and parameters described in the numerical analysis section of this paper. Magnitudes of the pile load share, i.e., loads carried by piles, were calculated for cases of different $b$, $E_p/E_m$, and $t$ values.

Moments to be carried by a piled foundation significantly affect its structural design. Major errors can be made in estimating such moments if the pile load share is miscalculated (Eid 2011). Pile load share is sensitive to the pile-subgrade stiffness ratio and foundation rigidity. Results shown in Figure 12 for the concerned foundation system support this conclusion especially in the case of thin piled foundations resting on rock of high stiffness, i.e., low $E_p/E_m$ values. It can be seen that the pile share of the total imposed load decreases to less than 10% in cases of having $E_p/E_m = 10$ and foundation of 0.7 meter thickness. This share tripled when $b$ decreases from 1.0 to 0.2. On the other hand, piles carry most of the loads imposed on piled foundation resting on low-stiffness rock (Figure 12). This high sensitivity of the pile load share to the value of $b$ confirms the importance of considering the change in rock mass stiffness with depth, and the associated effect on the behaviour of axially loaded piles, in the design of piled foundations on rock.

5. Conclusions

Three-dimensional finite element analyses as well as full-scale pile loading tests were conducted to study the elastic settlement behaviour of axially loaded piles entirely embedded in nonhomogenous rock. Cases of typical pile-rock stiffness ratios that have not been previously explored were considered. The following conclusions are based on the analyses of this study’s results:

a) Reduction in the elastic settlement of single pile due to the mobilization of the end bearing is not significant.

b) Pile elastic settlement is influenced by the pile slenderness ratio, pile-rock stiffness ratio, and the rate of increase in rock stiffness along the pile length. Charts were presented to predict the elastic settlement of piles using such parameters. The charts uniquely cover cases of pile-rock stiffness ratios that represent reinforced concrete piles in nonhomogeneous sedimentary rock.

c) The depth at which settlement becomes insensitive to the increase in pile length depends on the rate of change in rock stiffness with depth as well as the pile-rock stiffness ratio and rock Poisson’s ratio. An equation was introduced to incorporate the effect of these parameters in estimating such critical depth.

d) The pile share of the loads imposed on foundations is sensitive to the accuracy of predicting the single pile settlement behaviour and consequently to the change in rock stiffness along the pile length. Such sensitivity is more pronounced for piled foundations resting on rock of high mass stiffness.

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