Determination of partial factors for a vertically loaded pile based on reliability analysis

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abstract: The aim of this study is to establish a procedure to rationally determine the partial factors for a vertically loaded pile in the limit state design format based on a sound reliability theory. The frequency of the usage of pile types and dimensions are investigated first. Several design examples are selected, and load ranges and combinations on typical piles are studied. Based on these results, typical load intensities, load combinations and soil profiles are set for the code calibration. Also uncertainties involved in seismic loading are investigated based on the historical seismic data using an extreme statistical analysis, so called POT (peaks over threshold) analysis. Uncertainties concerning resistances of piles are taken from a well known study by Okahara et.al.(1991). FORM (first order reliability method) analysis is carried out to find out the current level of reliability index. Finally, the design value method is employed to determine the partial factors.

Key words: pile design, vertical bearing capacity, partial factors, safety factor, reliability analysis, FORM, extreme statistics, POT analysis, code calibration

1. Introduction
1.1 Background

It is recognized widely that one of the origins of the introduction of the limit design concept started in Denmark in 1960th by the effort of Brick-Hansen(1967). By 1970th, Danish geotechnical design code has been based on the limit state design concept and partial factors were introduced. It is considered that Eurocode 7 (CEN,1994) was much influenced by this code (Mayerhof 1993; Ovesen, 1993).

The introduction of the limit state design concept, which is sometime called Load and Resistance Factors Design (LRFD), is also coming to be popular in North America. Such works includes Barker et al.(1991), AASHTO(1994), Becker(1996) and Phoon, Kulhawy and Grigoriu (1995 and 2000).

Gobel(1999) has summarized the works done in this area especially in North America and concluded that most of the works done in 1990th still contain some degree of vagueness in the determination of resistance factors. He further emphasizes the use of available data bases and rational probabilistic analysis to determine the partial factors.

Paikowsky and Stenersen (2000) who are working of revision of the AASHTO driven pile design procedure are proposing a method to design piles based on measurements during pile driving. They have carried out extensive statistical analysis on a large data base, and partial factors are proposed based on this result.

In Japan, conversions of the current design codes to ones based on the limit state design concept are very much activated (see Honjo et al., 2000). All the major foundation design codes seem to be aiming at the limit state design code and performance based design concepts.

1.2 Objectives of this study

The objective of this study is to establish a procedure to rationally determine the partial factors for a vertically loaded pile in the limit state design format based on the sound reliability theory. We would employ pile design method that is specified in "Specifications for Highway Bridges IV: Substructures"(JRA,1996), denoted as SHB(1996) hereafter.
This conventional design method can be rewritten in a simplified form as follows:

\[
\frac{1}{F_r} (R_t + R_s) \geq S_d + S_e \tag{1}
\]

where \( R_t \), \( R_s \), and \( R_{sc} \) are pile tip, pile side in sand and pile side in cohesive soil resistances, whereas \( S_d \) and \( S_e \) are vertical pile top forces induced by dead load and seismic force.

In the partial factor format for which we are going to carry out calibration, the partial factors are applied to dead and seismic loads as well as pile tip and side resistances which can be formulated as the follows:

\[
\gamma_{R_t} R_t + \gamma_{R_s} R_s \geq \gamma_{S_d} S_d + \gamma_{S_e} S_e \tag{2}
\]

where \( \gamma_{R_t} \), \( \gamma_{R_s} \), \( \gamma_{S_d} \) and \( \gamma_{S_e} \) are partial factors for the resistances and the forces respectively.

2. Frequent load ranges, combinations and soil profiles

2.1 Type and size of piles used in highway bridge foundations in Japan

Fukui et al. (1996) has carried out an investigation based on a questionnaire to study the details of highway bridge foundations in Japan. The investigation period was from November 1995 to January 1996. 4,995 bridge foundations were investigated of which 2,441 were ordinary pile foundations.

Presented in Fig. 1(a) is distribution of pile types among these 2,441 cases where about 70% consists of cast-in place piles, 24% steel piles and 5% PHC (pre-stressed high strength concrete) piles. The frequency of the cast-in place pile is considerably high due to the environmental restrictions during construction, such as noise and vibration. Due to the extraordinary high usage frequency of cast-in place pile, only this type of piles is considered in this study.

The distribution of pile diameters of the 1,734 cast-in place piles is shown in Fig. 1(b). Although some piles have diameter of 200 cm, more than 95% of them have diameter between 100 to 150 cm. Concerning the length of the cast-in place piles, about 85% of them distribute between 6 to 30 m as presented in Fig. 1(c).

2.2 Characteristics of the load ranges and combinations and the chosen cases

Based on the investigation on usage frequency of the piles for highway bridge foundations, 8 pile foundation design examples of cast-in place piles are selected for more detailed study on load ranges and combinations. The pile diameter ranges between 100 to 150 cm and the length 9.5 to 35.5 m. The ranges conveniently cover the most frequent dimensions of the commonly used cast-in place piles.

In 7 among the 8 examples chosen, the critical loading condition is seismic loading in L1 (Level 1) seismic situation 1. Only in one case, the pile diameter is determined by the L2 seismic force. Mainly for this reason, the partial factors are only calibrated for L1 case in this study.

Since L1 seismic loading situation is the most critical for vertical loading of the most of

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1 In SHB(1996), two seismic situations are considered: L1 (Level 1) earthquakes whose occurrences are expected to be several times during the service life of a structure. L2 (Level 2) earthquakes are much larger earthquakes whose occurrences are not certain during the service life. SHB specifies that seismic coefficient method be used for L1 earthquakes, whereas the ductility design method be employed for L2 (JSCE, 2000).
pile foundations, the vertical load intensity and load combinations of this situation for the most critical piles in each foundation example are plotted in Fig. 2. 2

It is a common practice to check the pile dimensions for two cases, namely case where buoyancy is considered (assuming ground water level coincide with the ground surface) and case where buoyancy is ignored. The load ranges and load combinations for the first case is shown in Fig. 2. It is observed in the figure that the range of the dead load lies from 500 to 2500 kN, so as the seismic vertical load from 500 to 3,500 kN. The regression line for the mean of the dead vs. the seismic load is $S_e = 1.2 S_d$ as indicated in the figure with the one standard deviation of $S_e = (1.2 \pm 0.3) S_d$. Essentially the same results are obtained for the case where buoyancy is ignored: in this case, the regression line is given as $S_e = (1.4 \pm 0.4) S_d$.

Based on the results of the load range and the combinations found in Fig. 2, 21 loading cases are set in the following way:
1. The basic vertical loads are chosen. In regard to the load intensity range observed in Fig. 2, vertical loads of 500, 1000, 1500, 2000 and 2500 kN are chosen.
2. For each vertical load, seismic loads are determined based on the regression lines presented in Fig. 2: 3 different kinds of seismic induced vertical forces are set which are denoted by u, a and l (upper, average and lower). Thus, the cases are denoted as, for example, 500.u, 2000.a etc.
3. For each case, a pile has to be designed for dead load and seismic induced vertical load under conditions considering and ignoring the buoyancy: they are indicated, for example, 500.u-bc (buoyancy considered), 2000.a-bn (buoyancy neglected).
4. Two cases are set for 1500 and 2000 considering the higher frequency of the usage of the piles in these, whereas only one case is set for the others, i.e. 500, 1000 and 2500.
5. The soil profiles are set for the loading cases based on the selected pile foundation design examples described previously. They are believed to represent the variety of profiles encountered in pile foundation design in Japan.

2.3 Designed pile lengths and diameters by the conventional method

Piles are designed for each case set in the previous section based on the design method specified in SHB(1996). The designed diameters of piles are not rounded off as are commonly done in practice, but left in the order of centimeters (cm). In the all cases, the diameters are determined for the cases pile buoyancy being neglected. In other words, L2 seismic situations are not critical in determining the dimensions of piles.

3. Uncertainties in basic variables
3.1 Uncertainties in the loads
3.1.1 Dead load

An investigation was carried out by Japan Road Association on uncertainty involved in the unit weight of reinforced concrete. It was concluded that the error in the unit weight follows a normal distribution with mean 1.0 and s.d. 0.015 (JRA, 1989). In this study, a normal distribution with mean 1.0 and s.d. 0.1 is assumed for the uncertainty of dead load. It will be seen later that with this magnitude of uncertainty, the dead load has very little influence on the results of the reliability analysis.

3.1.2 Seismic load

The seismic data are based on Usami catalog (Usami, 1997). All the earthquakes between 1600 and 1995 are considered.
Distances from the epicenters to the location of Tokyo Metropolis Hall are calculated. The accelerations for all earthquakes from the data at distances deduced above are calculated using Fukushima-Tanaka (1991) attenuation model.

The peaks over threshold analysis is employed in this study. In the POT analysis, data over a relatively high threshold value, say $u$, are fitted to an exceedance distribution of $u$, which can be well approximated by a General Pareto distribution (Reiss and Thomas, 1997). In the following descriptions, sometimes $k$ is used in place of $u$, where $k$ is number of data that has been taken into the analysis. Therefore, it implies that the larger $k$ data out of $n$ data are employed in the analysis.

We can derive the following equation as the distribution function of the annual maximum: 
$$F_1(x) = \frac{n+1-k}{n+1} + \frac{k}{n+1} \left( 1 - \left( 1 + \frac{x - \mu}{\sigma} \right)^{-1/\gamma} \right)^{n+1}$$

where the function is valid only for $x > \mu$. The estimated parameters finally introduced in this study are as follows: $n=396$, $k=99$, $\gamma=0.070$, $\mu=26.66$ and $\sigma=80.45$. The resulting 100, 500 and 1000 year expected values are 320, 490 and 570 (gal) respectively (Honjo and Amatya 2001).

The obtained 100 year expected maximum acceleration is assumed to induce the seismic force that is employed in SHB(1996) L1 situation. In the reliability analysis, reference period of a structure is assumed to be 100 years. Thus, the 100 year maximum seismic force distribution is obtained from Eq.(3) as follows:
$$G_{100}(s) = G_{100}(\frac{x}{x_{100}}) = F_{100}(\frac{x}{x_{100}}) = \left( F_1(\frac{x}{x_{100}}) \right)^{100}$$

where, $G_{100}(s)$ is the 100 year maximum seismic force distribution, $s$ is the seismic force, $x$ is the maximum ground surface acceleration, $x_{100}$ is the 100 year expected maximum ground surface acceleration, $s_{100}$ is the seismic force used in SHB(1996) for L1 seismic situation, $F_1$ is the distribution function of the annual maximum ground surface acceleration.

### 3.2 Uncertainties in the resistances

Okahara et.al. (1991) tried to separately evaluate uncertainties in SHB vertical pile bearing capacity calculation formula on unit pile tip resistance, $q_d$, and unit side resistance, $f$, based on the available pile loading tests data with respect to SPT $N$-values.

First they defined the failure load of a pile as the load when vertical displacement of the pile top reaches 10% of its diameter. They have carried out partial factor, the design value method is the most commonly used reliability tool today. Among the methods to determine the partial factors, the design value method is employed in this study (see for example Ditlevsen and Madison, 1996). The reasons

| Table 3 Uncertainties in the basic variables by Okahara et.al 1991 |

<table>
<thead>
<tr>
<th>Notation</th>
<th>Definition of variable</th>
<th>Mean value</th>
<th>c.o.v.</th>
<th>Distribution and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_f$</td>
<td>For side resistance estimated based on the following formula: $f = 4$ N (&lt;200) (kN/m²) for sand $f = 10$N (&lt;150) (kN/m²) for clay</td>
<td>1.07</td>
<td>0.46</td>
<td>Log-normal distribution.</td>
</tr>
<tr>
<td>$\delta_{qd}$</td>
<td>For tip bearing capacity based on the following formula: $qd = 100$ N (&lt;3,000) (kN/m²)</td>
<td>1.12</td>
<td>0.63</td>
<td>Log-normal distribution.</td>
</tr>
<tr>
<td>$\delta_N$</td>
<td>SPT $N$-value averaged over a homogenous soil layer thickness</td>
<td>1.0</td>
<td>0.05</td>
<td>Normal distribution.</td>
</tr>
</tbody>
</table>
for this selection are as follows:
1. The method is soundly based on the reliability theory, and is relatively simple and easy to calculate the partial factors.
2. It is empirically known that the results are reasonably stable and robust.
3. Due to the relatively simple structure of the method, the calculation process can be easily traced in reflecting the calculated partial factors.

4.2 Results of the analysis and partial factors

The performance function for this case is specified as follows:
\[ f(s_d, s_e, \delta_d, \delta_e) = \delta_d q_d A + \delta_e U \sum L(f_i - w - s_d - s_e) \]
\[ = \delta_d R_t + \delta_e R_s - w - s_d - s_e \quad (5) \]
where
- \( \delta_d \): A random variable indicating uncertainty involved in evaluation of pile tip resistance.
- \( \delta_e \): A random variable indicating uncertainty involved in evaluation of pile side resistance.
- \( W \): Effective weight of pile (kN).
- \( A \): Section area of pile tip (m²).
- \( q_d \): Limit bearing capacity of pile tip for an unit area (kN/m²).
- \( U \): Circumference of pile (m).
- \( L \): Thickness of soil layer that has side resistance (m).
- \( f_i \): Side resistance of each layer (kN/m²).

The calculated reliability index values are shown in Table 2 where \( \beta \) lie between 1.33 and 1.92, where the average is 1.73.

The sensitivity factors, \( \alpha 's \), for the dead load, \( S_d \), lies between -0.05 to -0.10, which essentially can be considered insensitive in the current reliability analysis. In contrast, those for the seismic load exhibit values between

### Table 2 Calculated partial factors by the design value method and the direct method

<table>
<thead>
<tr>
<th>Case</th>
<th>Vertical dead load</th>
<th>Vertical seismic load</th>
<th>Diameter of pile</th>
<th>Length of pile</th>
<th>Reliability Index</th>
<th>Partial factors by the design value method</th>
<th>Partial factors directly calculated from the design values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sd (kN)</td>
<td>Se (kN)</td>
<td>D (m)</td>
<td>L (m)</td>
<td>( \beta )</td>
<td>( \gamma_{Sd} )</td>
<td>( \gamma_{Ss} )</td>
<td>( \gamma_{Rs} )</td>
</tr>
<tr>
<td>500.0u</td>
<td>500</td>
<td>900</td>
<td>0.85</td>
<td>8</td>
<td>1.66</td>
<td>1.00</td>
<td>1.13</td>
</tr>
<tr>
<td>500.0a</td>
<td>700</td>
<td>700</td>
<td>0.77</td>
<td>8</td>
<td>1.77</td>
<td>1.01</td>
<td>1.13</td>
</tr>
<tr>
<td>500.0l</td>
<td>500</td>
<td>500</td>
<td>0.68</td>
<td>8</td>
<td>1.91</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>1000.0u.1</td>
<td>1000</td>
<td>1800</td>
<td>0.97</td>
<td>14</td>
<td>1.73</td>
<td>1.00</td>
<td>1.14</td>
</tr>
<tr>
<td>1000.0a.1</td>
<td>1400</td>
<td>1400</td>
<td>0.86</td>
<td>14</td>
<td>1.81</td>
<td>1.01</td>
<td>1.13</td>
</tr>
<tr>
<td>1000.0l.1</td>
<td>1000</td>
<td>1000</td>
<td>0.75</td>
<td>14</td>
<td>1.92</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>1500.0u.1</td>
<td>1500</td>
<td>2700</td>
<td>0.91</td>
<td>27.5</td>
<td>1.64</td>
<td>1.00</td>
<td>1.13</td>
</tr>
<tr>
<td>1500.0a.1</td>
<td>2100</td>
<td>2100</td>
<td>0.8</td>
<td>27.5</td>
<td>1.74</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>1500.0l.1</td>
<td>1500</td>
<td>1500</td>
<td>0.68</td>
<td>27.5</td>
<td>1.88</td>
<td>1.01</td>
<td>1.11</td>
</tr>
<tr>
<td>1500.0u.2</td>
<td>1500</td>
<td>2700</td>
<td>1.05</td>
<td>27</td>
<td>1.68</td>
<td>1.00</td>
<td>1.13</td>
</tr>
<tr>
<td>1500.0a.2</td>
<td>2100</td>
<td>2100</td>
<td>0.92</td>
<td>27</td>
<td>1.76</td>
<td>1.01</td>
<td>1.13</td>
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<tr>
<td>1500.0l.2</td>
<td>1500</td>
<td>1500</td>
<td>0.79</td>
<td>27</td>
<td>1.90</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>2000.0u.1</td>
<td>2000</td>
<td>3600</td>
<td>1.45</td>
<td>22.5</td>
<td>1.70</td>
<td>1.00</td>
<td>1.13</td>
</tr>
<tr>
<td>2000.0a.1</td>
<td>2800</td>
<td>2800</td>
<td>1.29</td>
<td>22.5</td>
<td>1.79</td>
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<td>2000</td>
<td>1.12</td>
<td>22.5</td>
<td>1.93</td>
<td>1.01</td>
<td>1.12</td>
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<tr>
<td>2000.0u.2</td>
<td>2000</td>
<td>3600</td>
<td>1.41</td>
<td>14</td>
<td>1.33</td>
<td>1.01</td>
<td>1.13</td>
</tr>
<tr>
<td>2000.0a.2</td>
<td>2800</td>
<td>2800</td>
<td>1.26</td>
<td>14</td>
<td>1.41</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>2000.0l.2</td>
<td>2000</td>
<td>2000</td>
<td>1.09</td>
<td>14</td>
<td>1.50</td>
<td>1.01</td>
<td>1.10</td>
</tr>
<tr>
<td>2500.0u</td>
<td>2500</td>
<td>4500</td>
<td>1.41</td>
<td>23.5</td>
<td>1.67</td>
<td>1.00</td>
<td>1.13</td>
</tr>
<tr>
<td>2500.0a</td>
<td>3500</td>
<td>3500</td>
<td>1.24</td>
<td>23.5</td>
<td>1.75</td>
<td>1.01</td>
<td>1.13</td>
</tr>
<tr>
<td>2500.0l</td>
<td>2500</td>
<td>2500</td>
<td>1.07</td>
<td>23.5</td>
<td>1.90</td>
<td>1.01</td>
<td>1.12</td>
</tr>
<tr>
<td>Mean</td>
<td>1.73</td>
<td>1.01</td>
<td>1.13</td>
<td>0.71</td>
<td>0.71</td>
<td>1.02</td>
<td>2.30</td>
</tr>
<tr>
<td>s.d.</td>
<td>0.16</td>
<td>0.00</td>
<td>0.01</td>
<td>0.06</td>
<td>0.03</td>
<td>0.00</td>
<td>0.17</td>
</tr>
</tbody>
</table>
-0.82 to -0.87, which are significant and fall in relatively narrow range. On the other hand, those of the pile tip and side resistance, $R_t$ and $R_s$, lie between 0.17 to 0.42 and 0.27 to 0.52 respectively.

In the present study, it is intended that the newly developed design formula should have equivalent safety margin compared to the conventional design formula. For this reason, the target reliability index, $\beta_T$, is set to 1.73, which is the average of the calculated reliability indices.

The design value method is used to calculate the partial factors for $R_t$ and $R_s$ because the basic variables $\delta_{qd}$ and $\delta_{t}$ are assumed to follow the lognormal distributions.

The calculated partial factors are presented in Table 2. In addition, the calculated partial factors, namely $\gamma_{Rt}$ and $\gamma_{Rs}$, are plotted against $R_s/R_t$ and $L/D$ in Fig. 6 to be used in the further consideration where $L$ and $D$ being the length and the diameter of each pile.

The following considerations can be made based on these results:

1. The range of partial factors calculated by the design value method are $1.00 \cdot 1.01$ for $\gamma_{Sd}$, $1.10 \cdot 1.14$ for $\gamma_{Se}$, $0.62 \cdot 0.80$ for $\gamma_{Rt}$ and $0.65 \cdot 0.79$ for $\gamma_{Rs}$.

2. As supporting information, the range of partial factors directly calculated from the design values are given. They are $1.01 \cdot 1.02$ for $\gamma_{Sd'}$, $1.91 \cdot 2.48$ for $\gamma_{Se'}$, $0.81 \cdot 0.96$ for $\gamma_{Rt'}$ and $0.81 \cdot 0.91$ for $\gamma_{Rs'}$.

3. The range of $\gamma_{Sd}$ is very narrow and can be essentially regarded as 1.0.

4. The ranges of $\gamma_{Se}$ are very different for the two method; the fluctuation, however, is relatively narrow considering the thick tail of the extreme value distribution. The discrepancy between the results by the two methods comes from the inaccuracy in approximating this distribution by a lognormal distribution. It is therefore speculated that the both methods fail to give appropriate partial factors for the seismic force. Some careful treatment need to be taken in determining the partial factor for the seismic force.

5. $\gamma_{Rt}$ and $\gamma_{Rs}$ varies rather widely. In order to understand reasons behind these scatters, they are plotted against $R_s/R_t$ as shown in Fig.3(a). It is very clear from the figure that $\gamma_{Rt}$ increases from 0.60 to 0.80 when $R_s/R_t$ increases from 1.0 to 5.0, whereas $\gamma_{Rs}$ decreases from 0.80 to 0.65 in the same range of $R_s/R_t$. Considering the convenience in the design calculation, $R_s/R_t$...
is replaced by L/D in Fig.3(b). Essentially the same characteristic mentioned above is observed.

6. In order to find reasons for the observed behavior above, the sensitivity factors of $R_t$ and $R_s$ are plotted against $R_s/R_t$ in Fig.4. $\alpha_{R_t}$ and $\alpha_{R_s}$ are exhibiting the same behavior as $\gamma_{R_t}$ and $\gamma_{R_s}$ in Fig.3; therefore it is appropriate to conclude that the sensitivity factors are responsible for the behavior.

6. Conclusions

Based on the results obtained in the reliability analysis, a proposal is made to determine partial factors for each basic variable:

Dead Load $S_d$: There is not much to discuss on determining the partial factor for $S_d$. All results support that 1.0 should be adopted for $\gamma_{S_d}$.

Seismic Load $S_e$: Due to the extraordinary shape of the extreme value distribution, it seems that neither the design value method nor the direct calculation give an appropriate partial factor value. For this reason, $\gamma_{S_e}$ is determined after the partial factors for $R_t$ and $R_s$ are fixed. The value of 1.6 is finally recommended for $\gamma_{S_e}$. The reason for this choice is to keep the ratio of the total force to the total resistance to be 1:2, which is presently adopted in SHB(1996).

Pile tip resistance $R_t$ and side resistance $R_s$. As discussed above, the partial factors are much influenced by the relative magnitude of $R_t$ and $R_s$, which is also almost proportional to L/D ratio. Based on this fact, L/D ratio should be introduced in determining the partial factors because L/D can be more conveniently used in design calculation than $R_s/R_t$.

The proposed partial factors are given as follows:

$$\gamma_{R_t} = 0.60 + 0.05 \frac{L}{D}$$

$$\gamma_{R_s} = 0.75 - 0.0025 \frac{L}{D}$$

$$(10 \leq L/D \leq 40)$$

(6)

The proposed definitions are superposed on Fig.3(b).

If one feels some hesitation in introducing such a new L/D dependent partial factors, an alternative is to use a constant partial factor of 0.70 for both $\gamma_{R_t}$ and $\gamma_{R_s}$. In this particular case, these two components happen to compensate each other, and the uniform factor of 0.70 can almost give uniform safety margin for considered combinations of $R_t$ and $R_s$.

Note that the views stated in this paper are those of the authors', and do not necessarily reflect the resolutions of the working group in JRA.

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