CHAPTER 8: DRILLED PIERS AND CAISSONS

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8.1 Introduction

Drilled pier foundations, the subject matter of this chapter, belong to the same category as pile foundations. No sharp deviations can be made between piers and piles because both of them serve similar purpose. The distinctions are based on the method of installation. A pile is installed by driving, whereas a pier is installed by excavating. Thus, a foundation unit installed in a drill-hole may also be called a bored cast-in-situ concrete pile. Here, distinction is made between a small diameter pile and a large diameter pile. A pile, cast-in-situ, with a diameter less than 0.75 m (or 2.5 ft) is sometimes called a small diameter pile. A pile greater than the above size, is called a large diameter bored-cast-in-situ pile. The latter definition is used in most non-American countries whereas in the USA, such large diameter bored piles are called drilled piers, drilled shafts, and sometimes drilled caissons.

Well foundations also called as Caissons, have been in use for foundations of bridges. Well foundation has its origin in India. Well foundations have been used in India for hundreds of years for providing deep foundation below the water level for monuments, bridges and aqueducts.

Caisson foundations have been used for most of the major bridges in India. Materials generally used for construction are reinforced concrete, brick or stone masonry. United States of America and other western countries are equally popular for use of well or caisson foundations. Well foundation is a type of deep foundation which is generally provided below the water level for bridges.

The term ‘Caisson’ is originally derived from the French word “Caisse” which means box or chest. Hence caisson means box like structure, rectangular or round, which is sunk from the surface of either land or water to the desired depth.
8.2 Types of Drilled Piers

Drilled piers may be described under four types. All four types are similar in construction technique, but differ in their design assumptions and in the mechanism of load transfer to the surrounding earth mass. These types are illustrated in Fig. 8.1.

i) *Straight-shaft end-bearing piers* develop their support from end-bearing on strong soil, "hardpan" or rock. The overlying soil is assumed to contribute nothing to the support of the load imposed on the pier (Fig. 8.1(a)).

ii) *Straight-shaft side wall friction piers* pass through overburden soils that are assumed to carry none of the load, and penetrate far enough into an assigned bearing stratum to develop design load capacity by side wall friction between the pier and bearing stratum (Fig. 8.1(b)).

iii) *Combination of straight shaft side wall friction and end bearing piers* are of the same construction as the two mentioned above, but with both side wall friction and end bearing assigned a role in carrying the design load. When carried into rock, this pier may be referred to as a socketed pier or a "drilled pier with rock socket" (Fig. 8.1(c)).

iv) *Belled or under reamed piers* are piers with a bottom bell or under ream (Fig. 8.1(d)). A greater percentage of the imposed load on the pier top is assumed to be carried by the base.
(a) straight-shaft end-bearing pier

(b) straight-shaft sidewall-shear pier

(c) straight-shaft pier with both sidewall-shear and end bearing

(d) Under-reamed (or belled) pier
8.3 Advantages and Disadvantages of Drilled Pier Foundations

8.3.1 Advantages

1. Pier of any length and size can be constructed at the site.
2. Construction equipment is normally mobile and construction can proceed rapidly.
3. Inspection of drilled holes is possible because of the larger diameter of the shafts.
4. Very large loads can be carried by a single drilled pier foundation thus eliminating the necessity of a pile cap (in case of a pile foundation).
5. The drilled pier is applicable to a wide variety of soil conditions.
6. Changes can be made in the design criteria during the progress of a job.
7. Ground vibration that is normally associated with driven piles is absent in case of drilled pier construction.
8. Bearing capacity can be increased by under-reaming the bottom (in non-caving materials).

8.3.2 Disadvantages

1. Installation of drilled piers needs a skillful supervision and quality control.
2. The method is incommodious. Sufficient storage space is needed for all the equipments and materials used in the construction.
3. In case of driven pile, there is an added advantage of increased bearing capacity due to compaction in granular soil, which cannot be obtained by drilled pier construction.
4. Construction of drilled piers is very difficult at places where there is a heavy current of ground water flow due to artesian pressure.
8.4 Design Considerations

The process of the design of a drilled pier generally involves the following:

1. The objectives of selecting drilled pier foundations for the project.
2. Analysis of loads coming on each pier foundation element.
3. A detailed soil investigation and determining the soil parameters for the design.
4. Preparation of plans and specifications which include the methods of design, tolerable settlement, methods of construction of piers, etc.
5. The method of execution of the project.

In general the design of a drilled pier may be studied under the following headings.

1. Allowable loads on the piers based on ultimate bearing capacity theories.
2. Allowable loads based on vertical movement of the piers.
3. Allowable loads based on lateral bearing capacity of piers.

In addition to the above, the uplift capacity of piers with or without under-reams has to be evaluated.

The following types of strata are considered.

1. Piers embedded in homogeneous soils, sand or clay.
2. Piers in a layered system of soil.
3. Piers socketed in rocks.

It is better that the designer selects shaft diameters that are multiples of 150 mm (6 in) since these are the commonly available drilling tool diameters.

8.5 Load Transfer Mechanism

Fig.8.2 (a) shows a single drilled pier of diameter $d$, and length $L$ constructed in a homogeneous mass of soil of known physical properties. If this pier is loaded to failure under an ultimate load $Q_u$, a part of this load is transmitted to the soil along the length of the pier and the balance is transmitted to the pier base. The load transmitted to the soil along the pier is called the *ultimate friction load or skin load*, $Q_f$, and that transmitted to the base is the ultimate base or point load $Q_b$. The total ultimate load, $Q_u$, is expressed as (neglecting the weight of the pier)

$$Q_u = Q_b + Q_f = q_b A_b + \sum_{i=1}^{N} f_i P_i \Delta z_i$$

.....Eq.8.1

where
\[ q_b = \text{net ultimate bearing pressure} \]
\[ A_b = \text{base area} \]
\[ f_{si} = \text{unit ultimate skin resistance of layer } i \]
\[ P_i = \text{perimeter of pier in layer } i \]
\[ \Delta z_i = \text{thickness of layer } i \]
\[ N = \text{number of layers} \]

If the pier is instrumented, the load distribution along the pier can be determined at different stages of loading. Typical load distribution curves plotted along a pier are shown in Fig. 8.2 (b) (O'Neill and Reese, 1999). Since the load transfer mechanism for a pier is the same as that for a pile, no further discussion on this is necessary here. However, it is necessary to study in this context the effect of settlement on the mobilization of side shear and base resistance of a pier. As may be seen from Fig. 8.3, the maximum values of base and side resistance are not mobilized at the same value of displacement. In some soils, and especially in some brittle rocks, the side shear may develop fully at a small value of displacement and then decrease with further displacement while the base resistance is still being mobilized (O'Neill and Reese, 1999). If the value of the side resistance at point \( A \) is added to the value of the base resistance at point \( B \), the total resistance shown at level \( D \) is over-predicted. On the other hand, if the designer wants to take advantage primarily of the base resistance, the side resistance at point \( C \) should be added to the base resistance at point \( B \) to evaluate \( Q \). Otherwise, the designer may wish to design for the side resistance at point \( A \) and disregard the base resistance entirely.

### 8.6 Vertical Bearing Capacity of Drilled Piers

For the purpose of estimating the ultimate bearing capacity, the subsoil is divided into layers (Fig. 8.4) based on judgment and experience (O'Neill and Reese, 1999). Each layer is assigned one of four classifications.

1. Cohesive soil [clays and plastic silts with undrained shear strength \( c_u < 250 \text{ kN/m}^2 \)]
2. Granular soil [cohesionless geo-material, such as sand, gravel or non-plastic silt with uncorrected SPT(N) values of 50 blows per 0.3m or less].
Fig 8.2: Typical set of load distribution curves (O'Neill and Reese, 1999)

1 kips = 4.4484 kN
1 ft = 0.3048 m
3. Intermediate geo-material [cohesive geo-material with undrained shear strength $cu$ between 250 and 2500 kN/m$^2$, or cohesionless geo-material with SPT (N) values $> 50$ blows per 0.3 m]

4. Rock [highly cemented geo-material with unconfined compressive strength greater than 5000 kN/m$^2$].

The unit side resistance $f_s (= f_{max})$ is computed in each layer through which the drilled shaft passes, and the unit base resistance $q_b (= q_{max})$ is computed for the layer on or in which the base of the drilled shaft is founded.

The soil along the whole length of the shaft is divided into four layers as shown in Fig. 8.4.
8.7 Effective Length for Computing Side Resistance in Cohesive Soil

O'Neill and Reese (1999) suggest that the following effective length of pier is to be considered for computing side resistance in cohesive soil.

*Straight shaft:* One diameter from the bottom and 1.5 m from the top are to be excluded from the embedded length of pile for computing side resistance as shown in Fig. 8.5(a).

*Belled shaft:* The height of the bell plus the diameter of the shaft from the bottom and 1.5 m (5 ft) from the top are to be excluded as shown in Fig 8.5(b).
8.8 Bearing Capacity Equation for the Base Resistance

The equation for the ultimate base resistance may be expressed as

\[ q_b = s_c N_c d_c c + s_q d_q \left( N_q - 1 \right) q_0' + \frac{1}{2} \gamma d_s d_d N_\gamma \]  

\text{Eq. 8.2}

where,

- \( N_c, N_q \) and \( N_\gamma \) = bearing capacity factors for long footings
- \( s_c, s_q \) and \( s_\gamma \) = shape factors
- \( d_c, d_q \) and \( d_\gamma \) = depth factors
- \( q_0' \) = effective vertical pressure at the base level of the drilled pier
- \( \gamma = \) effective unit weight of the soil below the bottom of the drilled shaft to a depth = 1.5 \( d \) where
- \( d = \) width or diameter of pier at base level
c = average cohesive strength of soil just below the base.

For deep foundations the last term in Eq. (8.2) becomes insignificant and may be ignored. Now Eq. (8.2) may be written as

\[ q_b = s_c d_c N_c c + s_q d_q \left( N_q - 1 \right) q_0^* \]  
Eq. 8.3

8.9 Bearing Capacity Equations for the Base in Cohesive Soil

When the Undrained Shear Strength, \( c_u < 250 \text{ kN/m}^2 \)

For \( \Phi = 0, \) \( N_q = 1 \) and \( (N_q - 1) = 0, \) here Eq. (8.3) can be written as (Vesic, 1972)

\[ q_b = N_c^* \times c_u \]  
Eq. 8.4

in which \( N_c^* = \frac{4}{3} \left( \ln I_r + 1 \right) \)  
Eq. 8.5

\( I_r = \) rigidity index of the soil

Eq. (8.4) is applicable for \( c_u < 96 \text{ kPa} \) and \( L > 3d \) (base width)

For \( \Phi = 0, I_r \) may be expressed as (O'Neil and Reese, 1999)

\[ I_r = \frac{E_s}{3c_u} \]  
Eq. 8.6

where \( E_s = \) Young's modulus of the soil in undrained loading.

Table 8.1 gives the values of \( I_r \) and \( N_c^* \) as a function of \( c_u. \)

If the depth of base (\( L < 3d \) (base))

\[ q_b \left( = q_{\text{max}} \right) = \frac{2}{3} I_r + \frac{L}{6d} N_c^* c_u \]  
Eq. 8.7

When \( c_u > 96 \text{ kPa}, \) the equation for \( q_b \) may be written as

\[ q_b = 9c_u \]  
Eq. 8.8

for depth of base (\( L > 3d \) (base width)).

\[ \begin{array}{|c|c|c|}
\hline
\text{\( c_u \)} & \text{\( I_r \)} & \text{\( N_c^* \)} \\
\hline
24 \text{ kPa} & 50 & 6.5 \\
\hline
48 \text{ kPa} & 150 & 8.0 \\
\hline
\geq 96 \text{ kPa} & 250-300 & 9.0 \\
\hline
\end{array} \]

Table 8.1 Values of \( I_r = E_s/3c_u \) and \( N_c^* \)
8.10 Bearing Capacity Equation for the Base in Granular Soil

Values \( N_C \) and \( N_q \) in Eq. (8.3) are for strip footings on the surface of rigid soils and are plotted as a function of \( \Phi \) in Fig. 8.6. Vesic (1977) explained that during bearing failure, a plastic failure zone develops beneath a circular loaded area that is accompanied by elastic deformation in the surrounding elastic soil mass. The confinement of the elastic soil surrounding the plastic soil has an effect on \( q_b (= q_{\text{max}}) \). The values of \( N_C \) and \( N_q \) are therefore dependent not only on \( \Phi \), but also on \( I_r \). They must be corrected for soil rigidity as given below.

\[
N_C(\text{corrected}) = N_C C_C \\
N_q(\text{corrected}) = N_q C_q \quad \text{Eq. 8.9}
\]

where, \( C_C \) and \( C_q \) are the correction factors.

As per Chen and Kulhawy (1994) Eq.8.3 may now be expressed as

\[
q_b = cN_c s_c d_c C_C + (N_q - 1)H_0 s_q d_q C_q \quad \text{Eq. 8.10}
\]

\[
C_C = C_q = \frac{1 - C_q}{N_c \tan \phi} \quad \text{Eq. 8.11 (a)}
\]

\[
C_q = \exp \left\{ -3.8 \tan \phi + \frac{(3.07 \sin \phi) \log_{10} 2I_r}{1 + \sin \phi} \right\} \quad \text{Eq. 8.12 (b)}
\]

where, '\( \phi \)' is the effective angle of internal friction. \( I_r \) is the reduced rigidity index

\[
I_r = \frac{I_r}{1 + \Delta I} \quad \text{Eq. 8.12}
\]

\[
I_r = \frac{E_d}{2(1 + \mu_d)q_0 \tan \phi} \quad \text{Eq. 8.13}
\]

By ignoring cohesion, where,

\[
E_d = \text{drained Young's modulus of the soil} \\
\mu_d = \text{drained Poisson's ratio} \\
\Delta = \text{volumetric strain within the plastic zone during the loading process}
\]

The expressions for \( \mu_d \) and \( \Delta \) may be written as (Chen and Kulhawy, 1994)

\[
\mu_d = 0.1 + 0.3 \phi_{rel} \quad \text{Eq. 8.14}
\]

\[
\Delta = \frac{0.005(1 - \phi_{rel})q_0'}{p_a} \quad \text{Eq. 8.15}
\]
where, $\phi_{rel} = \left(\frac{\phi^0 - 25^0}{45^0 - 25^0}\right)$ for $25^0 \leq \phi^0 \leq 45^0$ ……Eq. 8.16

$\phi_{rel}$ = relative friction angle factor, $p_a$ = atmospheric pressure = 101 kPa.

Chen and Kulhawy (1994) suggest that, for granular soils, the following values may be considered.

Loose soil, $Ed = 100$ to $200$ p$a$ ……Eq. 8.17
Medium dense soil, $Ed = 200$ to $500$ p$a$
Dense soil, $Ed = 500$ to $1000$ p$a$

The correction factors $C_c$ and $C_q$ indicated in Eq. (8.9) need be applied only if $I_{rr}$ is less than the critical rigidity index $(I_r)_{crit}$ expressed as follows

$$(I_r)_{critical} = \frac{1}{2} \exp\left\{2.85 \cot\left(\frac{45^0 - \phi^0}{2}\right)\right\}$$ ……Eq. 8.18

The shape and depth factors in Eq. (8.3) can be evaluated by making use of the relationships given in Table 8.2.

**Table 8.2 Shape and depth factors (Eq. 17.3) (Chen and Kulhawy, 1994)**

<table>
<thead>
<tr>
<th>Factors</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_c$</td>
<td>$1 + \frac{N_d}{N_c}$</td>
</tr>
<tr>
<td>$s_d$</td>
<td>$d_q = \frac{1 - d_q}{N_c \tan \phi}$</td>
</tr>
<tr>
<td>$s_q$</td>
<td>$1 + \tan \phi$</td>
</tr>
<tr>
<td>$d_q$</td>
<td>$1 + 2 \tan \phi(1 - \sin \phi)^2 \frac{\pi}{180} \tan^{-1} \frac{L}{d}$</td>
</tr>
</tbody>
</table>

**Base in Cohesionless Soil**

The theoretical approach as outlined above is quite complicated and difficult to apply in practice for drilled piers in granular soils. Direct and simple empirical correlations have been suggested by O’Neill and Reese (1999) between SPT N value and the base bearing capacity as given below for cohesionless soils.

$q_b (= q_{max}) = 57.5N \text{ kPa} \leq 2900 \text{ kN} / \text{m}^2$ ……Eq. 8.19 (a)
where \( N = \text{SPT value} < 50 \text{ blows} / 0.3 \text{ m.} \)

**Base in Cohesionless IGM**

Cohesionless IGM's are characterized by SPT blow counts of more than 50 per 0.3 m. In such cases, the expression for \( q_b \) is

\[
q_b (= q_{\text{max}}) = 0.60N_{60} \left( \frac{P_a}{q_0'} \right)^{0.8} q_0' \quad \cdots \text{Eq. 8.20}
\]

where \( N_{60} = \text{average SPT corrected for 60 percent standard energy within a depth of } 2d \text{ (base)} \) below the base. The value of \( N_{60} \) is limited to 100. (No correction for overburden pressure)

\( p_a = \text{atmospheric pressure in the units used for } q_o'(=101 \text{ kPa}) \)

\( q_0' = \text{vertical effective stress at the elevation of the base of the drilled shaft.} \)

**8.11 Bearing Capacity Equations for the Base in Cohesive IGM or Rock (O’neill And Reese, 1999)**

Massive rock and cohesive intermediate materials possess common properties. They possess low drainage qualities under normal loadings but drain more rapidly under large loads than cohesive soils. It is for these reasons undrained shear strengths are used for rocks and IGMs. If the base of the pier lies in cohesive IGM or rock \((RQD = 100 \text{ percent})\) and the depth of socket, \( D_s \), in the IGM or rock is equal to or greater than \( 1.5d \), the bearing capacity may be expressed as

\[
q_b (= q_{\text{max}}) = 2.5q_u \quad \cdots \text{Eq. 8.21}
\]

where \( q_u = \text{unconfined compressive strength of IGM or rock below the base} \)

For \( RQD \) between 70 and 100 percent,

\[
q_b (= q_{\text{max}}) = 4.83 \left( q_u \right)^{0.51} \text{ MPa} \quad \cdots \text{Eq. 8.22}
\]

For jointed rock or cohesive IGM

\[
q_b (= q_{\text{max}}) = \left[ s^{0.5} + (ms^{0.5} + s)^{0.5} \right] q_u \quad \cdots \text{Eq. 8.23}
\]

where, \( q_u \) is measured on intact cores from within \( 2d \) (base) below the base of the drilled pier. In all the above cases \( q_b \) and \( q_u \) are expressed in the same units and \( s \) and \( m \) indicate the properties of the rock or IGM mass that can be estimated from Tables 8.3 & 8.4.
Table 8.3 Descriptions of rock types

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Carbonate rocks with well developed crystal cleavage (e.g. dolostone, limestone, marble)</td>
</tr>
<tr>
<td>B</td>
<td>Lithified argillaeous rocks (mudstone, siltstone, shale, slate)</td>
</tr>
<tr>
<td>C</td>
<td>Arenaceous rocks (sandstone, quartzite)</td>
</tr>
<tr>
<td>D</td>
<td>Fine-grained igneous rocks (andesite, dolerite, diabase, rhyolite)</td>
</tr>
<tr>
<td>E</td>
<td>Coarse-grained igneous and metamorphic rocks (amphibole, gabbro, gneiss, granite, norite, quartz-diorite)</td>
</tr>
</tbody>
</table>

Table 8.4 values of \( s \& m \) (dimensionless) based on rock classification (Carter and Kulhawy, 1988)

<table>
<thead>
<tr>
<th>Quality of Rock mass</th>
<th>Joint Description &amp; Spacing</th>
<th>( s )</th>
<th>Value of ‘m’ as a function of rock types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td>Excellent</td>
<td>Intact (Closed); ( \text{spacing} &gt; 3\text{m}(10\text{ft}) )</td>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td>Very good</td>
<td>Interlocking; ( \text{spacing of 1 to 3m} )</td>
<td>0.1</td>
<td>3.5</td>
</tr>
<tr>
<td>Good</td>
<td>Slightly Weathered; ( \text{Spacing of 1 to 3m} )</td>
<td>( 4 \times 10^{-2} )</td>
<td>0.7</td>
</tr>
<tr>
<td>Fair</td>
<td>Moderately weathered; ( \text{Spacing of 0.3 to 1m} )</td>
<td>( 10^{-4} )</td>
<td>0.14</td>
</tr>
<tr>
<td>Poor</td>
<td>Weathered with gouge(soft material); ( \text{Spacing of 30 to 300 mm} )</td>
<td>( 10^{-5} )</td>
<td>0.04</td>
</tr>
<tr>
<td>Very Poor</td>
<td>Heavily weathered; ( \text{Spacing of less than 50 mm} )</td>
<td>0</td>
<td>0.007</td>
</tr>
</tbody>
</table>
8.12 Estimation of Settlements of Drilled Piers at Working Loads

O'Neill and Reese (1999) suggest the following methods for computing axial settlements for isolated drilled piers:

1. Simple formulas
2. Normalized load-transfer methods

The total settlement $S_t$ at the pier head at working loads may be expressed as (Vesic, 1977)

$$S_t = S_e + S_{bb} + S_{bs} \quad \text{……Eq. 8.24}$$

where,

- $S_e$ = elastic compression
- $S_{bb}$ = settlement of the base due to the load transferred to the base
- $S_{bs}$ = settlement of the base due to the load transferred into the soil along the sides.

The equations for the settlements are

$$S_e = \frac{L(Q_a - 0.5Q_{fm})}{A_b E} \quad \text{……Eq. 8.25}$$

$$S_{bb} = C_p \frac{Q_{mb}}{dq_b} \quad \text{……Eq. 8.25}$$

$$S_{bs} = 0.93 + 0.16 \sqrt{\frac{L}{d}} C_p \frac{Q_{fm}}{Lq_b} \quad \text{……Eq. 8.26}$$

where

- $L$ = length of the drilled pier
- $A_b$ = base cross-sectional area
- $E$ = Young's modulus of the drilled pier
- $Q_a$ = load applied to the head
- $Q_{fm}$ = mobilized side resistance when $Q_a$ is applied
- $Q_{hm}$ = mobilized base resistance
- $d$ = pier width or diameter & $C_p$ = soil factor obtained from Table 8.5
Table 8.5 Values of $C_p$ for various soils (Vesic, 1977)

<table>
<thead>
<tr>
<th>Soil</th>
<th>$C_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (dense)</td>
<td>0.09</td>
</tr>
<tr>
<td>Sand (loose)</td>
<td>0.18</td>
</tr>
<tr>
<td>Clay (stiff)</td>
<td>0.03</td>
</tr>
<tr>
<td>Clay (soft)</td>
<td>0.06</td>
</tr>
<tr>
<td>Silt (dense)</td>
<td>0.09</td>
</tr>
<tr>
<td>Silt (loose)</td>
<td>0.12</td>
</tr>
</tbody>
</table>

8.12.1 Normalized Load-Transfer Methods

Reese and O'Neill (1988) analyzed a series of compression loading test data obtained from full sized drilled piers in soil. They developed normalized relations for piers in cohesive and cohesionless soils. Figures 8.6 and 8.7 can be used to predict settlements of piers in cohesive soils and Figs. 8.8 and 8.9 in cohesionless soils including soil mixed with gravel. The boundary limits indicated for gravel in Fig. 8.9 have been found to be approximately appropriate for cemented fine-grained desert IGM's (Walsh et al., 1995). The ranges of validity of the normalized curves are as follows:

Fig 8.6: Normalized side load transfer for drilled shaft in cohesive soil
Fig 8.7: Normalized base load transfer for drilled shaft in cohesive soil

(O'Neill and Reese, 1999)

Fig 8.8: Normalized side load transfer for drilled shaft in cohesionless soil

(O'Neill and Reese, 1999)
Figures 8.6 and 8.7
Normalizing factor = shaft diameter $d$
Range of $d = 0.46$ m to 1.53 m

Figures 8.8 and 8.9
Normalizing factor = base diameter
Range of $d = 0.46$m to 1.53m
The following notations are used in the figures:

\[ S_R = \text{Settlement ratio} = \frac{S_a}{d} \]
\[ S_a = \text{Allowable settlement} \]
\[ N_{fm} = \text{Normalized side load transfer ratio} = \frac{Q_{fm}}{Q_f} \]
\[ N_{bm} = \text{Normalize base load transfer ratio} = \frac{Q_{bm}}{Q_b} \]

8.13 Lateral Bearing Capacity of Drilled Piers

It is quite common that drilled piers constructed for bridge foundations and other similar structures are also subjected to lateral loads and overturning moments. The methods applicable to
piles are applicable to piers also. This chapter deals with the method as recommended by O'Neill and Reese (1999). This method is called Characteristic load method and is described below.

8.13.1 Characteristic Load Method (Duncan et al., 1994)

The characteristic load method proceeds by defining a characteristic or normalizing shear load ($P_c$) and a characteristic or normalizing bending moment ($M_c$) as given below.

For Clay

$$ P_c = 7.34d^2 \left( \frac{ER_t}{c_u} \right)^{0.68} \quad \ldots \ldots \text{Eq. 8.27} $$

$$ M_c = 3.86d^3 \left( \frac{ER_t}{c_u} \right)^{0.46} \quad \ldots \ldots \text{Eq. 8.28} $$

For Sand

$$ P_c = 1.57d^2 \left( \frac{\gamma' d \phi' K_p}{ER_t} \right)^{0.57} \quad \ldots \ldots \text{Eq. 8.29} $$

$$ M_c = 1.33d^3 \left( \frac{\gamma' d \phi' K_p}{ER_t} \right)^{0.4} \quad \ldots \ldots \text{Eq. 8.30} $$

where $d =$ shaft diameter

$E =$ Young's modulus of the shaft material

$R_t =$ ratio of moment of inertia of drilled shaft to moment of inertia of solid section (= 1 for a normal un-cracked drilled shaft without central voids)

$c_u =$ average value of undrained shear strength of the clay in the top 8 $d$ below the ground surface

$\gamma' =$ average effective unit weight of the sand (total unit weight above the water table, buoyant unit weight below the water table) in the top 8$dbelow the ground surface

$\phi' =$ average effective stress friction angle for the sand in the top 8$d$ below ground surface

$K_p =$ Rankine's passive earth pressure coefficient $= \tan^2 \left( 45^\circ + \phi' / 2 \right)$

In the design method, the moments and shears are resolved into groundline values, $P_t$ and $M_t$, and then divided by the appropriate characteristic load values [Equations (8.27) through (8.30)]. The lateral deflections at the shaft head, $y$, are determined from Figures 8.10 and 8.11, considering the conditions of pile-head fixity. The value of the maximum moment in a free- or fixed-headed
drilled shaft can be determined through the use of Fig 8.12 if the only load that is applied is ground line shear. If both a moment and a shear are applied, one must compute \( y_t^{(\text{combined})} \), and then solve Eq.8.31 for the "characteristic length" \( T \) (relative stiffness factor).

\[
y_t^{(\text{combined})} = 2.43 \frac{P_t T^3}{EI} + 1.62 \frac{M_t T^2}{EI}
\]  
(Eq. 8.31)

where, \( I \) is the moment of inertia of the cross-section of the drilled shaft.

Fig 8.10: Groundline shear-deflection curves for (a) clay and (b) sand (Duncan et al., 1994)

Fig 8.11: Groundline moment-deflection curves for (a) clay and (b) sand (Duncan et al., 1994)
The principle of superposition is made use of for computing ground line deflections of piers (or piles) subjected to groundline shears and moments at the pier head. The explanation given here applies to a free-head pier. The same principle applies for a fixed head pile also. Consider a pier shown in Fig. 8.13(a) subjected to a shear load $P_t$ and moment $M_t$ at the pile head at ground level. The total deflection $y_t$ caused by the combined shear and moment may be written as

$$y_t = y_p + y_m$$  \hspace{1cm} (Eq. 8.32)

where $y_p$ = deflection due to shear load $P_t$ alone with $M_t = 0$
$y_m$ = deflection due to moment $M_t$ alone with $P_t = 0$
Again consider Fig. 8.13(b). The shear load $P_t$ acting alone at the pile head causes a deflection $y_p$ (as above) which is equal to deflection $y_{pm}$ caused by an equivalent moment $M_p$ acting alone.

In the same way Fig. 8.13(c) shows a deflection $y_m$ caused by moment $M_t$ at the pile head. An equivalent shear load $P_m$ causes the same deflection $y_m$ which is designated here as $y_{mp}$.

Based on the principles explained above, groundline deflection at the pile head due to a combined shear load and moment may be explained by Duncan et al., (1999) method for a free-head pier as follows.

1. Use Figs 8.10 and 8.11 to compute groundline deflections $y_p$ and $y_m$ due to shear load and moment respectively.
2. Determine the groundline moment $M$ that will produce the same deflection as by a shear load $P_t$ (Fig. 8.13(b)). In the same way, determine a groundline shear load $P_m$ that will produce the same deflection as that by the groundline moment $M_t$ (Fig. 8.13(c)).
3. Now the deflections caused by the shear loads $P_t + P_m$ and that caused by the moments $M_f + M_p$ may be written as follows:

\[ y_{tp} = y_p + y_{mp} \]  
\[ y_{tm} = y_m + y_{pm} \]  
(Eq. 8.33)  
(Eq. 8.34)

4. Lastly the total deflection $y_t$ is obtained as

\[ y_t = \frac{y_{tp} + y_{tm}}{2} = \frac{(y_p + y_{mp}) + (y_m + y_{pm})}{2} \]  
(Eq. 8.35)

The distribution of moment along a pier may be determined using Fig. 8.14.

8.14 Types of Caissons

i. Box caisson: This type of caisson is open at the top and closed at the bottom and is made of reinforced concrete, steel or timber. It is generally recommended when bearing stratum is available at shallow depth.
ii. Open caisson (wells): Open caisson is a box open both at top and bottom. It is made up of timber, concrete or steel. The open caisson is also called well.

iii. Pneumatic caissons: It has lower end designed as a working chamber in which compressed air is forced to prevent the entry of water and thus excavation can be done in dry condition.

8.15 Different Shapes of Well

The common types of well shapes are;

1. Single circular
2. Dumb well
3. Twin circular
4. Rectangular
5. Twin octagonal
6. Twin hexagonal
7. Double-D

The choice of a particular shape of well depends upon the size of the pier, the considerations of tilt and the shift during sinking and the vertical and horizontal forces to which well is subjected.

Fig 8.15: Different shapes of well foundation
A circular type well has the minimum perimeter for a given dredge area. Since, perimeter will be equidistant at the points from the centre of dredge-hole; the sinking is more uniform as compared to the other shapes. In circular well a disadvantage is that in the direction parallel to the span of bridge, the diameter of the well is much more than required to accommodate minimum size of pier and hence circular well obstruct water way much in comparison to other shapes.

The following are components of well foundation:

1. Well curb and cutting edges
2. Steining
3. Bottom plug
4. Well cap

8.15.1 Construction of Well Foundation

Well foundations can be constructed on dry bed or after making a sand island. At locations where the depth of water is greater than 5m to 6m and the velocity of water is high, wells can be fabricated at the river bank and then floated to the final position and grounded. Great care is to be exercised while grounding a well to ensure that its position is correct. Once
the well has touched the bed, sand bags are deposited around it to prevent scour. The well may sink into the river bed by 30 to 100cm under its own weight. The well is sunk into the ground to the desired level by extracting soil through the dredge holes. A strong cutting edge is provided to facilitate sinking. The tapered portion of the well above the cutting edge is known as curb. The walls of the well are known as steining. After the well has been sunk to the final position, the bottom plug is formed by concreting. The bottom plug serves as the base of the well. The well is filled with sand partly or completely. At the top of the well, a top plug is formed by concreting & R.C.C well cap is provided at the top to transmit both vertical and lateral loads. The vertical loads comprise the dead and live loads. The live load is brought on to the structure due to the passing of the vehicles over the bridge. The lateral loads are caused due to breaking or traction of vehicles, water current, wind, earth quakes etc. The lateral forces might act at different points on a pier, but their effect can be simulated by considering equivalent force acting at bearing level.

8.15.2 Forces Acting on a Well Foundation

1. Braking and tractive effort of the moving vehicles.
2. Force on account of resistance of the moving vehicles.
3. Force on account of water current.
4. Wind forces.
5. Seismic forces.
6. Earth pressure.
7. Centrifugal forces.

8.15.3 Depth of Well Foundation and Bearing Capacity

The depth of well foundation is based on the following 2 criteria.

1. There should be adequate embedded length of well, called the grip length below the lowest scour level.
2. The well should be taken deep enough to rest on strata of adequate bearing capacity in relation to the loads transmitted.

In North Indian rivers usually we meet with alluvial soils. The normal scour depth can be calculated by Lacey’s formula.
\[ R_L = 1.35 \left( \frac{q^2}{f^2} \right)^{1/3} \]  
(Eq.8.36)

where

\( q \) = discharge in cumecs per linear meter of water way

\( f \) = Lacey’s silt factor = \( 1.76 \sqrt{m_d} \)

\( m_d \) = mean weighted diameter in mm.

The maximum depth of scour, at the nose of pier is found to be twice the Lacey’s value of normal scour depth.

\( R = 2R_L \)

where, \( R \) is measured below the high flood level (HFL)

scour level = HFL - \( R \) = HFL - 2R_L

The grip length is taken as \( 1\frac{1}{3} R \) below the HFL according to IRC code.

It is further recommended that the minimum depth of embedment below the scour level should not be less than 2.0m for piers and abutment with arches and 1.2m for piers and abutments supporting other types of superstructures. Terzaghi and Peck have suggested the ultimate bearing capacity can be determined from the following expression.

\[ Q_f = Q_p + 2\pi R f D_f \]  
(Eq.8.37)

\[ Q_p = \pi R^2 \left( 1.2 c N_c + \gamma D_f N_q + 0.6 \gamma R N_f \right) \]  
(Eq.8.38)

where \( N_c, N_q, N_f \) = Terzaghi’s bearing capacity factors.

\( R \) = radius of well

\( D_f \) = depth of well

\( f_s \) = average skin friction

**8.16 Analysis of Well Foundation**

**8.16.1 Design of well cap**

The well cap supports the substructure of the bridge by spanning the dredge hole of the well and in case of more than one separate well, by spanning the distance between the wells. The top of well caps are usually kept at low water level or low tide level for general appearance and reducing obstruction to the flow. The piers or substructure transmit to the well cap not only
the direct loads but also moments caused by the various horizontal force. The well cap is
designed as a slab resting over the top of well.

For circular well, the design of the cap may be done
Consider the forces on the superstructure and substructure, calculate the resultant vertical load
(V), moment $M_{xx}$ and moment $M_{yy}$ at the top of well cap.
Compute $M = \sqrt{M_{xx}^2 + M_{yy}^2}$, & e = $M/V$

$$P_{1,2} = \frac{V}{A} \left( 1 + \frac{6e}{B} \right)$$

A is area of cross-section of well steining = $\pi(r_1^2 - r_2^2)$
Critical section for finding B.M will be a-a.
Determine pressure intensity at section a-a say $P_3$.
Assume that the shaded portion of the cap which is rested on steining is acted upon by a uniform
pressure intensity of magnitude $(P_1+P_2)/2 = P_4$
Area of segment of a circle = \[ \frac{1}{2} \left[ R^2 (\theta - \sin \theta) \right] \]
Distance of c.g. of segment from center of circle = \(\frac{4R\sin^3 \theta}{2}/3(\theta-\sin \theta)\)
Determine the areas of segments DEF and ABC with their center of gravities
Moment about section a-a

$$= P_4 A_1 \left( X_1 - \frac{B_p}{2} \right) - P_4 A_2 \left( X_2 - \frac{B_p}{2} \right) - \text{Weight of well cap/Area of well cap}(X_1-B_p/2) \quad (\text{Eq.8.39})$$

8.16.2 Design of Well Steining

The well steining is the main body of the well. After determining the maximum moments
and loads, the design of well steining through which the forces acting on the bridge are
transmitted to the base of well requires to be considered. The moments will go on reducing due
to the passive resistance offered by scour level.
The section of well steining just below the well cap has least direct load but is subjected to a
considerable moment and therefore, this section is critical for tensile and shear stresses.
At a level below the maximum scour level where the horizontal force gets neutralized by passive pressure of the earth, i.e., where the shear becomes zero, the moments are the maximum and the direct loads are also considerable.

When the well is circular and practically watertight, it is subjected to hoop compression during rising floods.

This hoop compressive stress varies depending on the flood level. Hoop compression in the steining is uniform up to the maximum scour level.

If wells are not circular, the stresses in the steining should be calculated taking the moments caused by the pressure due to differential head in such cases.

**8.16.3 Design of Well Curb and Cutting Edge**

Well curb now-a-days is usually made of reinforced concrete, with a steel cutting edge. The inner face of the curb is generally sloped two vertical to one horizontal.

The cutting edge of a well is almost made of steel. As it cuts through the bed, it must be extremely strong and rigidly tied to the well curb to withstand distortion, warping, twisting, shearing, crushing and spread out.

The well curb has a shape offering the minimum resistance during sinking, and should be strong enough to be able to transmit superimposed loads from the steining to the bottom plug. The curb should invariably be reinforced concrete of mix not leaner than M30 with minimum reinforcement of 72kg per cum excluding bond roads.

This quantity of steel should be suitably arranged to prevent spreading and splitting of the curb during sinking and in service.

**8.16.4 Design of Curb for Sinking**

The curb cuts through the soil by the dead weight of the well steining and kentledge, if any, when the inside of the well is dredged. After the well has penetrated the soil to a considerable depth, the forces acting on the curb will be as shown in Fig 8.17.
Fig 8.17: Force on well curb during sinking

D = Mean dia of curb in m,
N = Weight of steining in kN per m run,
θ = Angle in degrees of beveling face with the horizontal
µ = Coefficient of friction between soil and concrete of curb
P = Force in kN per m run of curb acting normal to the level surface.
Q = Force in kN per m length of curb acting tangentially to the level surface,
H = Horizontal resultant force in kN per m of curb
Q = Pµ

Resolving vertically

\[ \mu P \sin \theta + P \cos \theta = N \]

\[ \Rightarrow P = \frac{N}{\mu \sin \theta + \cos \theta} \]  \hspace{1cm} \text{(Eq.8.40)}

Resolving horizontally
\[ P \sin \theta - \mu P \cos \theta = H \]

\[ \Rightarrow H = P(\sin \theta - \mu \cos \theta) \]

\[
H \text{ per m run} = N \left( \frac{\sin \theta - \mu \cos \theta}{\mu \sin \theta + \cos \theta} \right) \quad \text{(Eq.8.41)}
\]

Total hoop tension = \( \frac{Hd}{2} \)

While sinking, active earth pressure of soil or external compression may not develop fully at the curb due to unsettled conditions.

Sometimes, during sinking, sand blow in case of deep dredge may result in sudden descent of well. To account for these eventualities, hoop tension reinforcement is increased by 50\% and vertical bond rods are provided.

\[
\text{Total hoop tension} = 0.75N \int \left( \frac{\sin \theta - \mu \cos \theta}{\mu \sin \theta + \cos \theta} \right) d \quad \text{(Eq.8.42)}
\]

8.16.5 Design of Curb Resting on the Bottom Plug

When the cutting edge is prevented from moving down by the reaction developed at the interface of the curb and the bottom plug, the reaction, neglecting cumulative effect of skin friction, could be resolved into horizontal and vertical components by assuming formation of a two-hinged parabolic arch within the thickness of the bottom plug. The weight of the material filled in the well and the bottom plug will be transmitted to the bed directly.
Fig 8.18: Force on well curb resting on bottom plug

For the condition assumed, the hoop tension H is given by the equation.

\[
H = \left( \frac{qd^2}{8r} \right) \frac{d}{2}
\]

(Eq.8.43)

where,

q = Total weight on the base/Area of plug
r = vertical height of imaginary inverted arch

In granular soils, the hoop tension H is relieved by the active pressure around the curb

Hoop compression \( C = \frac{1}{2} (P_1 + P_2) \frac{bd}{2} \)

(Eq.8.44)

\( P_1 \) = Active earth pressure at depth \( D_r \)
\( P_2 = \) Active earth pressure at depth \( (D_r - b) \)
\( \gamma = \) submerged unit weight of soil
Thus net hoop tension = (H – C)

At the junction provided at the corner to take care of this moment should be taken along the level base and anchored well into the steining.

Design of bottom plug

Based on theory of elasticity, the thickness of the seal t is given by

For circular well: 

\[ t^2 = \left( \frac{3W}{8f_c\pi} \right) (3 + \nu) \]  

(Eq.8.45)

If, \( W = q\pi r^2 \) and \( \nu = 0.5 \) equation reduce to 

\[ t^2 = \frac{1.18r \cdot q}{f_c} \]

\( f_c \) = flexure strength of concrete seal

\( W \) = Total bearing pressure on the base of well.