Two extreme conditions are normally considered for design and analysis.

DRAINED AND UNDRAINED STRENGTH

Soil Strength \(\rightarrow\) depend highly on pore pressure during loading

Two extreme conditions are normally considered for design and analysis.

1. Drained Strength \(; \text{when excess pore pressure } (\Delta u) \neq 0\)
   during loading or after fully dissipation of \(\Delta u\) 2 typical cases are;
   1) Sand or gravel layers of high \(k\) \(\rightarrow\) \(\Delta u\) dissipate fast
   2) Clayey soil with slow rate of loading \(\rightarrow\) small increase of \(\Delta u\) and longer time of dissipation
2. Undrained Strength; when $\Delta u$ remained during loading
   - $\Delta u$ fully developed and remained  “Fully undrained condition”
   - $\Delta u$ partially is dissipated  “Partially drained”

**Ex.**
1) Saturated clay with high rate of loading
2) Silt and fine sand with seismic or repeated load → Accumulation of pore pressure → Boiling

Mohr – Coulomb’s Effective Strength Equation

$$ \tau = \bar{c} + (\sigma - u_s - \Delta u) \cdot \tan \varphi $$

$u_s$ = Constant
$\Delta u$ = varied during loading

Drained or Effective strength concept
- usually applied for $\Delta u = 0$ or known $\Delta u$

Undrained or Total strength concept
- for $\Delta u$ is fully developed or unknown $\Delta u$ (high)

→ “Stress-path method” is normally used to explain drained and Undrained soil strength.
Actual Behaviors

1. Confined Compression / Consolidation
2. Triaxial Compression

Ex. At the edge of embankment

At Pt. A. For Normally Consolidated Clay at $K_o$-Condition

- $AB$ = Triaxial Undrained Loading (ESP)
- $AC$ = Triaxial Undrained Loading (TSP)
- $BC$ = Dissipation of Excess pore pressure

\[
\text{Fig. 20.7 Stress paths for oedometer test.}
\]
STRENGTH TEST BY TRIAXIAL COMPRESSION
**STRENGTH TEST BY TRIAXIAL COMPRESSION**

**Advantages**
1. Closely simulate the actual field stress condition.
2. Fully control the drainage condition in the sample.
3. Get more design informations. (Design Parameters)
   - $c$, $\phi$
   - $\Delta u$
   - $\Delta v$
   - $E$, $K_o$
4. Automatic control and monitoring capability.

**Disadvantages**
1. Require qualified technician
2. Costly

**SOIL SAMPLE**

1. Natural Soil (Saturated)
   - UCS \( \sigma_3 = 0 \rightarrow C_u \)
   - $\phi = 0$ Concept \( \rightarrow T_3 > 0 \rightarrow C_u \)
2. Compacted Soil (Unsaturated)
   - \( T_3 > 0 \rightarrow C_u, \Delta u \)
   - Very difficult to obtain
3. Gronular Soil
   - \( \tau_c \phi, \Delta = 0 \)
   - \( \bar{E}, \bar{\Delta v} \)
Types of Triaxial Test

1. True Triaxial Test
   - When $\sigma_1 > \sigma_2 > \sigma_3$
   - Usually occur in plane-strain condition in the field
   - Laboratory Test is quite complicated
     - need special equipment

2. Conventional Triaxial Test
   - $\sigma_2 = \sigma_3$
     - $\sigma_3 = \sigma_3$
     - $\sigma_3 = \sigma_3$

3. Repeated or Cyclic loading Triaxial Test
   - Simulate - earthquake
     - repeated load (pavement)
     - machine foundation
     - Oil drilling platform

4. Low strain Triaxial Test
   - $\frac{\Delta L}{L} = 1 \text{ to } 2\%$
   - Special Strain Gage
   - For
     - Tunneling work
     - Retaining wall
Simulation of triaxial Test

**Field**
- Before Loading
- Loading
- Consolidation
- Shearing
- Drainage

**Lab**
- Installation / Saturation
- Consolidation
- Shearing

### Field
- \( \sigma_{00} = k \sigma_{00} \)
- \( \Delta \sigma \)
- \( \sigma_{HH} \)

### Lab
- \( \sigma_i \)
- \( \sigma_i \)
- \( \sigma_i \)

### Simulation Details
- **Application of load**
- **Bring back to in situ condition**

### Table: UU-Test vs Consolidated Tests

<table>
<thead>
<tr>
<th></th>
<th>UU-Test</th>
<th>Consolidated Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unconsolidated</td>
<td>CU-Test</td>
</tr>
<tr>
<td></td>
<td>Undrained Test</td>
<td></td>
</tr>
<tr>
<td>( \sigma_1 )</td>
<td>Held Constant</td>
<td>Held Constant</td>
</tr>
<tr>
<td>( \sigma_3 )</td>
<td>Gradually Increased From ( \sigma_3 )</td>
<td>Equal to ( \sigma_3 )*</td>
</tr>
<tr>
<td>( u )</td>
<td>Drainage Lines Closed</td>
<td>Drainage Lines Open **</td>
</tr>
</tbody>
</table>

* Unless anisotropic consolidation is to be effected
** In back pressured tests, pressure is supplied to pore lines, but drainage is permitted
HUAI PA THAO DAM
STRENGTH CHARACTERISTIC, (C-ø DIAGRAM)

DRAINED STRENGTH
สภาพการเกิดความดันน้ำในมวลดิน ขณะที่มีการรับน้ำหนักจากภายนอกมีผลต่อ
กำลังรับน้ำหนักของดิน

Drained Strength เมื่อ Excess pore pressure (Δu) ≅ 0
ตลอดช่วงระยะเวลาการรับน้ำหนัก หรือในช่วงเวลาที่ Δu ระบบออกหมดแล้วจะ
เกิดขึ้นใน 2 กรณี คือ
1. ดินทราย หรือ กรวด เมื่อ k สูง → Δu ระบบออกได้รวดเร็ว
2. ดินเหนียวที่มีการบรรทุกน้ำหนักข้า → Δu เกิดขึ้นน้อยและมีเวลาระบายนิด

Undrained Strength เมื่อ Excess pore pressure (Δu) ≠ 0
ในช่วงที่เกิดการส่งน้ำหนักสูงจนเกิดการพิบัติ หรือเสี่ยงต่อการพิบัติ
1. ดินเหนียว ที่มีการบรรทุกน้ำหนักข้า → Δu เกิดขึ้นและสะสม
2. ดินทรายละเอียด (Silt) มีการสั่นสะเทือนหรือมีแรงแผ่นดินไหว
Δu เกิดในลักษณะ repeated load และสะสมจนเกิด Boiling

Mohr-Coulomb’s Effective Strength Equation

\[ \tau = \sigma + (\sigma - u - \Delta u) \cdot \tan \phi \]

การศึกษา Drained และ Undrained Strength คือการพิจารณาช่วงที่ Δu นิสัยการ
เปลี่ยนแปลงในช่วงระหว่างการรับน้ำหนักหรือขณะระบบฐานแล้วทำให้ τ (strength) ของดิน
เปลี่ยนแปลงได้ด้วย

การอธิบายพฤติกรรมนี้สามารถทำได้สะดวกโดยใช้ชื่อ “Stress path”
CONDITIONS OF STRESS CHANGE COMPARING TO TRIAXIAL TESTS

Path 1: Compression loading
\( \sigma_3 = \text{const.}, \ \sigma_1 = \text{increase} \)

Path 2: Extension loading
\( \sigma_3 = \text{increase}, \ \sigma_1 = \text{const.} \)

Path 3: Compression unloading
\( \sigma_3 = \text{decrease}, \ \sigma_1 = \text{const.} \)

Path 4: Extension unloading
\( \sigma_3 = \text{const.}, \ \sigma_1 = \text{decrease} \)

Case 1

Case 2

\[ \Delta \sigma_1 \]

Case 3

\[ \Delta \sigma_3 \]

Stress paths for triaxial tests.

CD-TEST

Fig. 38.3 Consolidated-drained (CD) triaxial test.
CU-TEST

1. Obtain specimen

2. Apply standard pressure e_s, permitting drainage

3. Increase soil stress \( \sigma \), permitting drainage

UU Test

1. Obtain specimen

2. Apply standard pressure \( e_s \), permitting drainage

3. Increase soil stress \( \Delta e_s \), permitting drainage

Fig. 18.5 Consolidated-drained (CU) triaxial test.

Fig. 18.4 Unconsolidated-drained (UU) triaxial test.
**Triaxial Tests.**

- **Initial Consolidation?**  
  - No  
  - Yes  

- **Cell Pressure?**  
  - No  
  - Yes  

- **Isotropic Consolidation?**  
  - No  
  - Yes

- **Unconfined Compression?** (U)  
  - No  
  - CAU

- **Drained Shearing?** (CD)  
  - No  
  - Yes

- **Shearing in Extension**  
  - CAUE

- **Shearing in Compression**  
  - CAUC

- **Drained Shearing?** (CD)  
  - No  
  - Yes

- **Shearing in Extension**  
  - CADE

- **Shearing in Compression**  
  - CADC

**UU** – Unconsolidated Undrained  
**CU** – Consolidated Undrained with pore pressure measurement  
**CD** – Consolidated Drained.

---

**DRAINED SHEAR STRENGTH**
DRAINED SHEAR STRENGTH

I. FIELD CONDITION
- Granular materials, dry or partially saturated.
- Cohesive materials, slow rate of loading

II. LABORATORY CONDITION
- Slow rate of loading, drainage permitted, excess pore pressure ($\Delta u$) = 0

CD-Test or Slow Test

Case Study  Triaxial Test on "Remolded Weald Clay" by Henkel (1956)

| LL. = 43% | Percent clay (<0.002) = 40% |
| PL. = 18% | Activity = 0.6 |
| PI. = 25% | Specific gravity (G) = 2.74 |

Case I. Normally Consolidated Clay (NC), 3 Samples

| Sample No. | 1 | 2 | 3 |
| Confining pressure (psi) | 10 | 30 | 100 |

Case II. Over Consolidated Clay (OC), 7 Samples

- Consolidated To = 120 psi
- Then rebound to test at

| Sample No. | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Confining p. (psi) | 5 | 8 | 15 | 25 | 45 | 60 | 70 |
| OCR. | 24 | 15 | 8 | 4.8 | 2.7 | 2.0 | 1.7 |
Normalized test results.
Test results on normally consolidated Weald clay.
Results of CD tests on overconsolidated Weald clay. $\bar{p} = 120 \text{ lb/in}^2$

Failure envelope of a clay with preconsolidation pressure $= \sigma_c$. 

\[ p = \frac{(\sigma + \pi)}{2} \text{(lb/in}^2) \]

\[ \bar{p} = 120 \text{ lb/in}^2 \]
Stress-volume relations for overconsolidated Weald

\[
\sigma_1 - w_f (OC) = (p_m)_{120 \text{ psi}}
\]

After drained shearing of soil samples, the final water contents \((W_f)\) at failure are linearly proportioned to \(g_f\) and \(P_f\) (in log. Scale)

Summary of CD-Test Results of Weald Clay

**Unique Characteristic of \( q_f - \bar{P}_f - w_f \) Relationship**

For Normally Consolidated Clay (NC)

For each soil, when CD-Test was test then the following results can be summarized:

1. The water content after consolidation is linearly proportioned to \( \log p_0 \), Consolidation pressure.
   
   Curve : \( W_o - \log (p_o) \) is straight line

2. After drained shearing of soil samples, the final water contents \((W_f)\) at failure are linearly proportioned to \( g_f \) and \( P_f \) (in log. Scale)

\[
\begin{align*}
W_o - \log(g_o) & \quad \text{are straight lines} \\
W_f - \log(p_f) & \quad \text{Parallel to each others}
\end{align*}
\]
For Overconsolidated Clay (O.C)

3. After preconsolidation pressures, the samples were rebound to a curtain confining pressure. The $w_o$ will show the lower value than $w_o$ of N.C. due to OC. Clay has inelastic property of soil.

4. The unique but not linear relationships between

$$w_o - \log(p_o), \quad w_f - \log(q_f), \quad w_i - \log(p_i)$$

were obtained for each soil at each $P_m$. 

Stress-volume relations for overconsolidated Weald $P_m = 120$ psi
Hvorslev's parameters

Hvorslev's Strength Parameter (1937)
- Combine effect of NC. And OC. to $\sigma_f$ and $\tau_f$

$$\phi = \frac{(\sigma_{av} + \tau_{av})}{2} \text{ (lb/in.}^2)$$

Hvorslev's failure lines

MOHR’s (NC.)
MOHR’s (OC.)
NC

$\sigma_f = e^\phi$ and $\tau_f$ for NC
$\sigma_f = e^\phi$ for NC
$\tau_f = e^\phi$ for OC

$\sigma_f = f(w_f) + f(\sigma_f)$

$\tau_f = \sigma_f + \phi - \tan \phi \cdot W_p$

Relationship between $\sin \phi$ and plasticity index for normally consolidated soils (From Kenney, 1959).

Soft Bangkok Clay
Fig. 7.24 Modified Mohr's failure envelope for quartz and clay minerals. (Note: 1 lb/in² = 6.9 kN/m²) (Replotted after R. E. Olson, Shearing Strength of Kaolinite, Illite and Montmorillonite, J. Geotech. Eng. Div., ASCE, vol. 100, no. GT11, 1974.)

Fig. 11. Relationship between void ratio and tan ϕ of various rock materials.
Relationship between peak and ultimate conditions.
Apparent Cohesion due to capillary

Effect of capillary tensions on effective stress and strength. (a) Sand. (b) Clay.
Effect of variable ground water conditions.

UNDRAINED SHEAR STRENGTH
Undrained Shear Strength

Sort strength is highly depended on the drainage condition in soil mass. During undrained condition when excess pore pressure is fully developed and no time to dissipate, the strength is called "undrained strength". Normally, the undrained strength is lower than drained strength due to the present of pore pressure. The behavior of undrained strength can also explain by the theory of effective stress and represented by "Stress path".

1. Theoretical or laboratory
   Fully undrained conditions can be simulated.

2. Practical or field
   Partially drained condition is usually occurred.

We should consider the worst case for the design.

Loading → drained strength higher then undrained strength is used.
Unloading → drained strength is lowest.
Fig. 28.6 Typical stress-strain curves from CU tests on Weald clay. (a) Specimens normally consolidated to 30 lb/ft², \( P_n = 120 \text{ lb/ft}^2 \); \( P = 10 \text{ lb/ft}^2 \). OCR = 12

---

Fig. 28.7 Effective stress paths from CU tests on normally consolidated Weald clay.

---

Fig. 28.8 Effective stress paths from CU tests on over-consolidated Weald clay, \( P_n = 120 \text{ lb/ft}^2 \).
Example 28.1

Given. Normally consolidated Weald clay with $p_0 = 30$ psi.

Find. $\sigma_1, \sigma_2, \sigma_3, u$, and $A$ when

a. $q = 3$ psi.

$A = \frac{\Delta u}{2q}$

Solution. Figure E28.1 is a blow-up version of the $q$ versus $p$ diagram of Fig. 28.7, using an effective stress path interpolated between those for $p_0 = 16$ lb/ft$^2$ and $p_0 = 39$ lb/ft$^2$.

\[
\sigma = \Delta \mu + \Delta u
\]

Fig. E28.1

<table>
<thead>
<tr>
<th>Stress (lb/ft$^2$)</th>
<th>(a)</th>
<th>(b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q$</td>
<td>5</td>
<td>8.7</td>
</tr>
<tr>
<td>$p$</td>
<td>35</td>
<td>38.7</td>
</tr>
<tr>
<td>$\sigma_1$</td>
<td>40</td>
<td>47.4</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>30</td>
<td>30.0</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>30</td>
<td>22.3</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>30</td>
<td>31.0</td>
</tr>
<tr>
<td>$\mu$</td>
<td>25</td>
<td>14.6</td>
</tr>
<tr>
<td>$A$</td>
<td>0.50</td>
<td>0.89</td>
</tr>
</tbody>
</table>

Example 28.2

Given. A specimen of Weald clay with $p_0 = 120$ psi and $p_0 = 10$ psi.

Find. $\sigma_1, \sigma_2, \sigma_3, u$, and $A$ at failure.

Solution. Figure E28.2 is a blow-up version of Fig. 28.8.

\[
\begin{align*}
q_f &= 10.2 \text{ lb/ft}^2 \\
p_f &= 20.2 \\
\sigma_1 &= 20.4 \\
\sigma_2 &= 10.0 \\
\sigma_3 &= 26.0 \\
\sigma_f &= 36.2 \\
\mu_f &= 15.8 \\
\mu &= -5.8 \\
A &= -0.38
\end{align*}
\]

Fig. E28.2
Fig. 28.9 Pore pressure parameter $A_f$ for Weald clay.

Fig. 28.10 Stress-volume relationships for normally consolidated Weald clay.

$A_f = 0.94$
Example 28.3

Given: Normally consolidated Well day with $p_0 = 30$ psi.
Find: $q_f$ and $w_f$ for both drained and undrained shear tests with $w_o$ increasing while $p_f$ remains constant.

Solution:
- Drained shear: Construct the effective stress path and find $p_f$ and $q_f$. Then find $w_f$ using the $w_f$ versus $w_o$, $q_f$ versus $w_o$, relations (see Fig. 28.2). $p_f = 48$ kPa, $q_f = 18$ kPa, $w_f = 20.6\%$.
- Undrained shear: Enter the stress-volume diagram with the given $p_0$ and find $w_f$. From $p_f$ versus $w_f$ find $p_f$. $p_f$ can be found from $q_f$ versus $p_f$. $w_f = 23.6\%$, $q_f = 8.7$ kPa, $p_f = 23.5$ kPa.

CD Test
$q_f = 17.5$ psi
from $w_f - q_f$
find $w_f = 20.6\%$
$w_o = 23\%$

CU Test
$q_f = 8.5$ psi
$w_f = 23\%$

Example 28.4

Repeat Example 28.3 with $q_f$ decreased and $w_o$ constant. Solution: Follow same steps as in Example 28.3. The undrained strength is the same for both examples. Note also that $q_f = 0.29 p_f$.

<table>
<thead>
<tr>
<th>$p_f$ (kPa)</th>
<th>$w_f$ (%)</th>
<th>$q_f$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained loading</td>
<td>48</td>
<td>20.6</td>
</tr>
<tr>
<td>Undrained loading and unloading</td>
<td>23.5</td>
<td>23.0</td>
</tr>
<tr>
<td>Drained unloading</td>
<td>22</td>
<td>23.2</td>
</tr>
</tbody>
</table>

AB = ESP for Undrained T.A. (CU) - Loading and Unloading.
AC = TSP for CU – Loading
AE = TSP = ESP for CD – Loading
AF = TSP = ESP for CD – Unloading
AD = TSP for CU – Unloading.
Example 28.5

**Given.** Overconsolidated Weald clay with $P_c = 120$ psi and $P_i = 30$ psi

**Find.** $q_f$ and $w_f$ for both drained and undrained shear with $\sigma_1$, increasing while $\sigma_3$ remains constant.

**Solution.** Follow same steps as in Example 28.3. The diagrams are given in Fig. E28.5 and the answers appear in the table in Example 28.6.
**Stress-volume relations for overconsolidated Weald clay**

\[
P_{oc} = 120 \text{ lb/ft}^2
\]

If \( P_o = 10 \text{ psi} \) → Construct ESP, TSP, Find \( \Delta u, A_f \)

**Relative Magnitude of Drained and Undrained Strength**

<table>
<thead>
<tr>
<th></th>
<th>Normally Consolidated Clay</th>
<th>Heavily Overconsolidated Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial compression loading ( (\sigma_3 \text{ constant with } \sigma_1 \text{ increasing}) )</td>
<td>1 CD &gt; CU</td>
<td>2 CU &gt; CD</td>
</tr>
<tr>
<td>Triaxial Compression unloading ( (\sigma_3 \text{ constant with } \sigma_1 \text{ decreasing}) )</td>
<td>3 CU = CD</td>
<td>4 CU &gt;&gt; CD</td>
</tr>
</tbody>
</table>

Note: These comparisons apply for specimens with the same initial effective stress.
การเปรียบเทียบ Shear Strength ของการทดสอบแบบ CU และ CD

Relative of Drained and Undrained Stress – Strain Curve.

Drained – Undrained Modulus

\[
E = \frac{3}{2(1+\mu)} E
\]

if \( \mu = 0.3 \),
\( E \geq 1.15 E \)

Actual \( E \approx 1 - 3 E \)

Stage Loading
หาความสัมพันธ์ของ Drained และ Undrained Modulus

If  
\[ E = \text{Undrained Young's Modulus} \]
\[ E = \text{Drained Young's Modulus} \]

While is  
\[ E = f(\bar{e}) \]

From Eq. 12.5 (a) for Triaxial Loading
\[ E_s = \frac{1}{E} \left[ \sigma_i - u (\sigma_i + \sigma) \right] \]

For Total Stress - Vertical loading  \( \sigma_i = \sigma_z, \sigma_y = \sigma_x = 0 \)
\[ E_s = \frac{1}{E} \sigma_z \]
...(1)

For Effective Stress - vertical loading  \( \bar{\sigma}_i = \bar{\sigma}_z, \bar{\sigma}_y = \bar{\sigma}_x = -\mu \)
\[ E_s = \frac{1}{E} (\sigma_z - 2\mu \bar{\sigma}_z) \]
...(2)

When  \( \bar{\nu} = \text{Poisson's ratio of mineral skeleton.} \)

For Isotropic mineral skeleton and axial loading from Eq. 26.5
\[ A = \frac{1}{1+2(\bar{C}_v/\bar{C})} \rightarrow A = \frac{1}{3} \]

Then
\[ \bar{\sigma}_v = \sigma_z - A \sigma_z = \sigma_z - \frac{1}{3} \sigma_z = \frac{2}{3} \sigma_z \]
...(3)
\[ \bar{\sigma}_y = 0 - A \sigma_z = -\frac{\sigma_z}{3} \]
...(4)

From Eq. (2)
\[ G_v = \frac{1}{E} \left( \frac{2}{3} \sigma_z + \frac{2}{3} \mu \sigma_z \right) = \frac{2}{3} \frac{\sigma_z}{E} (1+\mu) \]
...(5)

Or (1) = (5) Then
\[ E = \frac{3}{2(1+\mu)} \]
...(6)
Final Strains are depended on stress-path

1. Larger $p$ during loading $\Rightarrow$ cause smaller final strain ($E_a$)
2. Stress-path (ESP) closer to $K_f$ - line $\Rightarrow$ cause larger final strain due to plastic + yielding behaviors.

Fig. 29.15 Axial strains as a function of stress path. (a) Non-failure loading. (b) Elastic response to non-failure loading. (c) Nonelastic response to non-failure loading. (d) Failure loading. (e) Response to failure loading.
\[ \bar{\tau} = \bar{c} + \bar{c} \tan \bar{\phi} \]

**Unconsolidated undrained tests on partially saturated soil**

- **Strength**
- **Deformation**
- **Consolidation**
- **% Saturation**

**Application**
1. Compacted embankment
2. Earth dam
3. Landfill

Fig. 28.16  Compression of partially saturated soil in an oedometer.
Undrained Shear Strength

1. Undrained Shear Strength of Saturated Sand
   Undrained loading on saturated sand during fast or repeated loading conditions

   **Generally**
   1. Loose sand similar to N.C. or Soft Clay
   2. Dense sand similar to O.C. or Stiff Clay

   **Exception**
   For loose sand after peak, soil can maintain it failure condition and pore pressure start to decrease due to dilatency effect.

2. Cavitation of Pore Water
   For dense sand during searing, soil mass tend to expound and develop negative pore pressure.
   If \( u < -1.0 \text{ ATM.} (-14.4 \text{ psi}) \), then pore water will cavitate

   **Solution**
   Back pressure of about 1 ATM (or more) is applied in soil sample and confining pressure.
Medium Dense Sand --> No Cavitation

Dense Sand --> Cavitation @ Low Confining Pressure

Unsaturated due to cavitation

\[ \phi = 0 \]

Fig. 29.1 Stress-strain curves for undrained triaxial compression of a saturated sand (from Leondards, 1962).

Fig. 29.2 Stress paths for undrained triaxial compression of a saturated sand. (a) Loose, \( \epsilon_0 = 0.85 \). (b) Moderately dense, \( \epsilon_0 = 0.75 \).

The collapsing soil structure results in the rearrangement of soil particles after peak strength. Then, the excess pore pressure continues to increase and causes the ultimate K-line larger than peak K-line.
Fig. 29.5 Behavior of sensitive clay during undrained shear (after Crawford, 1959).

More Meta Structure

Unstable Structure

 Stable Structure

Fig. 29.7 Comparison of friction angles mobilized at peak resistance in drained and undrained tests.
4. Strength after repeated load or seismic load

Fatigue strength
(cycle / repeated loading) < static strength
(Single loading)

Due to:

1. Accumulation of pore pressure
2. Rearrangement of soil particles.
3. Reduction of cementing bonds.

Generally we want to find the “Fatigue Limit” which is (the number of load application (cycle) until) the failure strength below peak single strength occurred.

Fig. 29.8  Effect of repeated loading on undrained strength of very loose saturated sand. Specimen consolidated to 10 psi; void ratio = 0.834. (From Henly, 1963.)
Fig. 29.9. Pore pressures and axial strain versus number of cycles during repeated triaxial loading of loose saturated sand. (a) Axial strain versus number of cycles. (b) Observed change in pore water pressure versus number of cycles. (From Soed and Lee, 1985.)

Photo 1. Sand boil in Port Island

Photo 2. Extraction of the Pier Foundation in Port Island
Fig. 29.12 Relationship between pulsating deviator stress and number of cycles required to cause failure Sacramento River sand initial void ratio = 0.87; initial confining stress = 1.0 kPa/m². (From Seed and Lee, 1967.)
5. Anisotropic Consolidation before Shearing.

Generally

1. Typical isotropic T.A. Test
2. Anisotropic T.A. Test

Both tests will show the different stress paths, but the same failure line (same kf-line)
6. Remolding and Disturbance.

Undisturbed strength → soil in natural forming structure

remolded strength → Soil is completely disturbed (or destroy the original structure.)

disturbed strength → partially disturbed to some certain degree.

\[
\text{Sensitivity} = \frac{\text{Undisturbed strength}}{\text{Remolded strength}}
\]
7. Stress History
- Overconsolidated OC
- Normally Consolidated NC

Normalized undrained strength by SHANSEP

\[ \frac{q_f}{q_{fm}} \]

When clayey soil is naturally sedimented or artificially sedimented in laboratory. The undrained shear strength \((S_u)\) is proportioned to \(P_o\) as

\[ S_u = S'_{o}(OCR)^m \]  \(\ldots(1)\)

When \( S_u = S'_{o} \) @ Normally Consolidation State \((OCR = 1)\)

For Bangkok Clay \( S_u \approx 0.25 \)

\( m = \text{Constant} \approx 0.8 \pm 0.05 \)

\( OCR \) = Overconsolidation Ratio
Laboratory Procedures

1. Obtain soil sample (Undisturbed or slightly disturbed)
2. Consolidate in triaxial to $P_a > P_m$
3. Release consolidation pressure to test at different OCR.
4. Obtain $Su$ (or $q_o$) at various $P_o$ and OCR, Then solve for $m$ and $S_o$

Application

1. Use for stability analysis for large excavation
2. Use for field test quality control.
Table : Common Methods for Measuring undrained Strength

<table>
<thead>
<tr>
<th>Method</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>In-situ measurements</strong></td>
<td></td>
</tr>
<tr>
<td>1. Vane test</td>
<td>Usually considered to give best result, but is limited as to strength of soil with which it can be used</td>
</tr>
<tr>
<td>2. Soft to Medium Penetration test (SPT)</td>
<td>Gives crude correlation to strength</td>
</tr>
<tr>
<td>3. Static Conre Pure Test (CPT)</td>
<td></td>
</tr>
<tr>
<td><strong>Measurements upon undisturbed samples</strong></td>
<td></td>
</tr>
<tr>
<td>1. Unconfined compression</td>
<td>Best general purpose test; underestimates strength because disturbance decreases effective stress</td>
</tr>
<tr>
<td>2. UU test at in situ confining pressure</td>
<td>Most representative of laboratory tests, because of compensating errors.</td>
</tr>
<tr>
<td>3. CU test at in situ confining pressure</td>
<td>Overestimates strength, because disturbance leads to smaller water content upon reconsolidation</td>
</tr>
</tbody>
</table>

**STRESS – STRAIN RELATIONSHIP FOR CU**

**Applications**

1. Immediate Settlement of Loaded Area
2. Movement of Tunnel in soil
3. Excavation heaving and lateral movement

**Parameters**

- Young’s Modulus, $E$ (Elastic modulus)
- Shear modulus, $G$
- Poisson’s Raton, $\mu$ or $\nu$

From Eq. 12.4

$$G = \frac{E}{2(1 + \mu)} \quad \text{...(1)}$$
For saturated soil in undrained condition \( \mu = 0.5 \), then

\[
G = \frac{E}{3} \quad \cdots (2)
\]

Shear modulus \( G \) can be measured quite accurate by shear wave propagation velocity. Ex. Shear column, seismic survey etc.

Hardin and Black (1968) estimated \( G \) for sand and clay.

\[
G = 1230 \left( \frac{2.973 - e}{1 + e} \right) \sqrt{\sigma'} \quad \cdots (3)
\]

\( G \) and \( \sigma' \) in psi

Fig. 12.1 Various types of modulus.
<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>V (G)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, Saturated</td>
<td>0.4 – 0.5</td>
</tr>
<tr>
<td>Clay, Unsaturated</td>
<td>0.1 – 0.3</td>
</tr>
<tr>
<td>Sandy Clay</td>
<td>0.2 – 0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.3 – 0.35</td>
</tr>
<tr>
<td>Sand, Gravelly Sand</td>
<td>0.3 – 0.4</td>
</tr>
<tr>
<td>Rock</td>
<td>0.1 – 0.4</td>
</tr>
<tr>
<td>Loess</td>
<td>0.1 – 0.3</td>
</tr>
<tr>
<td>Ice</td>
<td>0.36</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.15</td>
</tr>
</tbody>
</table>

สูตร: Bowles, J.E. (1968)

\[
\frac{\Delta x}{x_0} = 0.5 \frac{\Delta y}{H}
\]

\[
\frac{\Delta y}{y_0} = 0.5 \frac{\Delta \psi}{H}
\]

Young’s Modulus flow stress – strain Carve

\[ E_i = \text{Initial Tangential Modulus} \]

\[ E_s = \text{Secant Modulus} \]

\[ E_t = \text{Tangential Modulus} \]
Test for small strain modulus

1. Repeated load test

2. Using low strain strain - gage

\[ \sigma_a \frac{\Delta h}{h} = L \]

\[ E = \frac{\sigma_a}{\Delta L} \rightarrow \Delta L. \]

Duncan's Stress – Strain Model (FEM)

Due to non-linearity of soil stress-strain curve, Duncan and Kulhawy (1969) proposed hyperbolic model for stress-strain

\[ \cdot E_i = f \left( E_i, R_f, K, n... \right) \]

\[ R_f = \text{Failure ratio} \]
\[ K = \text{Axial modulus member} \]
\[ N = \text{Modulus exponent} \]
During series of triaxial test, if strain levels C% axial strain are plotted along in stress – paths. The radial lines of the same strain can be drawn as "strain – contours".

Applications

1. Prediction of undrained settlement. (Elastic sett)
2. Estimation of consolidation settlement after dissipation
   Using $E_v$ v.s. $\sigma_v$ Conservation pressure $w_o$ v.s. $\bar{p}_c$
3. Estimation of failure vertical strain during loading
Methods for prediction of settlement from Triaxial Test.

1. Direct method.

- Predict stress – path due to in situ and construction loadings.
- Obtain the undisturbed representative soil samples.
- Run triaxial test follow the predicted stress – path.
- Direct measure the vertical strain in the sample

\[
E_v = E_v \text{ (undrained)} + E_v \text{ (Consolidation)}
\]

\[
S = E_v \frac{H}{L}
\]

When \( H \) = soil layer thickness
\( L \) = sample height.
2. Indirect Method

- Test a series of triaxial \((c'v')\) tests
  
  Ex. @ \(\sigma_c = 50, 100, 150\) kN/m\(^3\) as in Fig

- Establish the strain contours

- Establish the relationship between \(P_v\) v.s. \(e_v\) (or \(w_v\))

- Construct stress path follow actual loading.

- \(E_v\) (undrained) is obtained from strain contour.

- \(E_v\) (consolidation) \(\equiv \frac{1}{3} E_{(vol)}\)

\[E_{vol} \text{ calculate from } P_v \text{ v.s. } e_v\]  

\[E_v = E_v \text{ (undrained)} + E_v \text{ (consolidation)}\]
Normalized Deviator Stress

\[
\frac{\sigma_1 - \sigma_3}{\sigma_c}
\]
Factors stress – strain behaviors (E)

1. Consolidation pressure $\frac{P}{\sigma}$, $P_o$

   N.C. Clay $\rightarrow$ E is proportional to $\frac{P}{\sigma}$ or $\sigma$. Then the plot of normalized stress – strain curves is unique for each clay. Figure 30.4

   O.C. Clay $\rightarrow$ E is depended on OCR and stress level (F.S.) Figure 30.6

2. Loading Rate and Loading cycle

   - Faster loading rate $\rightarrow$ higher (on table 30.3)
   - Second or subsequent loading cycles $\approx 1.5 E$ (first loading)

3. Time and Aging Effects

   - Thixotropic Effects $\rightarrow$ Soil stronger with time for remolded and high liquidity index.
   - Consolidation Time $\rightarrow$ longer time $\rightarrow$ secondary consolidation $\rightarrow$ stronger soil.
   - Strain Rate Effect $\rightarrow$ Dynamic stronger than Static $E$ (dynamic) $= 1.5$ to $2.0 E$ (Static)

Note: without considering of excess pore pressure.

Fig. 30.7 Strength tests on Boston blue clay.
4. Loading patterns

Stress – strain (E) behaviors of soil are highly depended on loading pattern (or stress paths)

Ex.

Test 1.  \( \rightarrow \) Compression loading from \( K_o \)
(Spread footing, embankment ...)

Test 2.  \( \rightarrow \) Unloading from \( K_o \)
(Excavation pit, deep foundation ...)

Test 3.  \( \rightarrow \) unloading from sampling and compression loading in laboratory
(Soil sampling and compression Test)
**Fig. 30.8** Stress-strain data on Boston blue clay. Note: Zero strain on CA-UU test taken at $e_1 = e_2 = 0$. (From Ladd, 1964.)

---

**Diagram**

**Technicians**
- Thesis Research
- Perturbation of failure

**Experts / Researchers**
- Terzaghi or Muller
- Applied/Theoretical Researchers

**Engineers**
- Simple Models
  - Simplicity of engineer's mastery
  - Technical simulation models, work done & accuracy comply with reality substitute & not portray reality
  - Calculations & Test results may not agree but model suitable for design if experience indicates representation of reality
- Simplified things invested by field experience

**Work Done / Work Necessary**
- Primitive Models
  - Rules of thumb
  - Reality unknown

**Phases in Development of Balanced Design**
- Learning from failure
- Time

(Adapted from Duddeck, 1981)
Application of Advanced soil Mechanics

1. Soil Investigation
2. Pile Foundation
3. Excavation
4. Land Reclamation
5. Slope Stability
6. Embankment and Dam
7. Tunneling
8. Geotechnical Monitoring
9. Seepage and Filter
10. Soil Dynamic and Earthquake
11. Soil Engineering Database

Soil Investigation (Site Characterization)
เพื่อประเมินสภาพชั้นดินและคุณสมบัติดิน (และหิน) เพื่อการศึกษาวิเคราะห์ออกแบบ และแก้ไขปัญหาทางวิศวกรรมปฐพี
เนื้อหาจาก Advanced Soil Mechanics ที่เกี่ยวข้อง

1. Soil Formation, Weathering Processes, Sedimentation, Deposition
2. Soil Physical Properties and Classification
3. Soil and Clay Mineralogy
5. Laboratory Test ➔ Physical, Strength, Compressibility
6. Database, zoning, GIS ➔ Statistic, Computer Graphic, Soil profile Model.

Pile Foundation (Deep foundation)

Problems concerned

1. Pile Capacity
   - Strength
   - Stress - Strain

2. Consolidation – Settlement
   - Stress - Strain (Elastic)
   - Consolidation
   - Stress - distribution

3. Method of Installation
   - Driver ➔ pore pressure, soil displacement
   - Bored
   - Pre bored

4. Pile group / Mat foundation
   - Group efficiency ➔ Stress overlapping
   - Relative stiffness
   - Differential Settlement
5. Caisson / Shaft
   - End bearing
   - Seepage
   - Horizontal pressure, Friction

6. Bored pile / Barrette Wall (Slurry)
   - Bentonite properties
   - Trench Stability
   - Seepage.

7. Tunneling
   *Problem*
   1. Strength
   2. Insitu – Stress, Stress Release
   3. Seepage
   4. Rock bolting – Soil nailing, living
   5. Blasting, Tunneling machine

8. Geotechnical monitoring
   *Problem*
   1. Pore pressure
   2. Stress, load
   3. Movements
   4. Temperature
   5. Permeability

9. Excavation
   *Problem*
   1. Soil Strength - Stability
   2. Seepage
   3. Lateral earth pressure – lateral movement.
   4. Pressure relief – heaving

10. Land Reclamation
    *Problem*
    1. Settlement
    2. Soil Strength - Stability
    3. Compaction - Soil improvement

11. Slope Stability (Natural – Manmade)
    *Problem*
    1. Soil Strength
    2. Pore pressure – drainage - infiltration
    3. Erosion

12. Embankment and Dams
    *Problem*
    1. Soil strength
    2. Compaction
    3. Seepage – drainage
    4. Settlement
13. Seepage and Filter

Problem 1.

14. Soil Dynamic and Earthquake

Problem 1.

15. Soil Engineering Database

Problem 1.

---

Design Pile Foundation

Pile Types

1. Short pile (3-12 m)
   Timber, R/C

2. Long P/C pile
   Section 15 $\rightarrow$ 62.5 cm.
   Length 26 m. Maximum

3. Bored Pile
   - Dry Process $d = 35$ $\rightarrow$ 200 cm.
   - Wet Process
   - Micro pile (grouting technique)

4. Steel Pile
1. Static method from soil strength

\[ Q_j = P \cdot \sum_{i=1}^{n} (\beta_i \cdot C_i \cdot L_i) \]
\[ Q_p = A_p \cdot \bar{\sigma}_v \cdot N_q \]

Where
- \( P \) = Skin friction parameter length
- \( \beta_i \) = adhesion factor
- \( C_i \) = cohesion
- \( L_i \) = thickness of soil layer
- \( A_p \) = section area of pile
- \( \bar{\sigma}_v \) = effective overburden pressure

2. Dynamic method

- for construction monitoring and cross-checking only

3. Static method from Dutch Cone

\[ P_c = \sum_{i=1}^{n} (\alpha_i \cdot q_{fi} \cdot L_i \cdot P) + \lambda \cdot A_p \cdot q_c \]

Where
- \( n \) = no. of soil layer
- \( \alpha_i \) = adhesion factor
- \( q_{fi} \) = local friction from Dutch Cone
- \( L_i \) = thickness of soil layer
- \( \lambda \) = point bearing factor
- \( q_c \) = point cone resistance within 4D
4. Design by B.M.A. Code

\[ P_z = q_f \cdot P \cdot L \]

When

\[ q_f = \text{skin friction equal to } 800 \text{ kg/m}^2 \quad (0-7) \text{ m.} \]
\[ 800 + 200L_1 \quad (> 7) \text{ m.} \]

\[ L_1 = \text{pile length for longer than 7 m.} \]

Pile Group Reduction
- Feld Rule
- Friction Area Ratio

Lateral Loaded pile
- Brom’s Theory

Pile Foundation

- Site Information
  1. Space
  2. 4. Soil Profile
  3. Noise
  5. Soil Prop
  3. Vibration

- Support Structure Data
  1. Function Requirement
  2. Load

- Evaluation of Pile Types
  1. Short pile
  2. Precast P/C Pile
  3. Bored pile

- Performance Analysis of Pile
  1. Pile Capacity
  2. Settlement
  3. Lateral Stability

- Feasibility Analysis
  1. Construction Problems
  2. Cost

- Select other pile type

- Detailed Design
- Drawings
- Specification

Monitor During Construction and Operation
Foundation Information
1. Soil Profile
2. Soil Properties
3. Ground Water Level

Embankment Information
1. Cross-Section
2. Material Properties
3. Condition of Analysis

Selection of Failure Mode
1. Circular
2. Wedge
3. Compination
4. Irregular

Methods for Analysis
1. Simple Method of Slices
2. Bishop's Method
3. Friction Circle (graphic)
4. FEM

Pore Pressure Data

Calculate min F.S.
F.S. > Flow F.S.

Design Criteria

Movement and Pore Pressure Monitoring

Modified Section or Corrective Method

Embankment Information
1. Cross-Section
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Design Criteria

Movement and Pore Pressure Monitoring

Modified Section or Corrective Method

Design Adjustment

Project: FIFTH BWS. IMPROVEMENT PROJECT
Subject: DESIGN FLOW CHART

Comparison
- Time
- Construction Sequence
- Construction Method
- Cost

Trial Section and Outfall Preliminary Bunk

Detail Design

Construction

Monitoring

As Predicted

Differ From Prediction

Continue Until Completion

Design Adjustment

Project: FIFTH BWS. IMPROVEMENT PROJECT
Subject: DESIGN FLOW CHART

Comparison
- Time
- Construction Sequence
- Construction Method
- Cost

Trial Section and Outfall Preliminary Bunk

Detail Design

Construction

Monitoring

As Predicted

Differ From Prediction

Continue Until Completion

Design Adjustment
Project: FIFTH BWS IMPROVEMENT PROJECT
Subject: FLOW CHART OF THE DESIGN OF SLUDGE LAGOON

INVESTIGATION
- Soil Investigation
- Topographic Survey
- Other Information Collection

SOIL MODELS

SOIL PROPERTY EVALUATION

PROBLEMS IDENTIFICATION
- Settlement Rate
- Total Settlement
- Stability Bearing

DESIGN CRITERIA
- Load Strength Density
- Rate of Construction
- Construction Sequence
- Allowable Settlement
- Waiting Period

ANALYSIS W/O IMPROVEMENT (PREDICTION)
ANALYSIS W IMPROVEMENT (PREDICTION)

Geotechnical Engineering involvement

<table>
<thead>
<tr>
<th>Engineering Structures</th>
<th>Settlement</th>
<th>Stability</th>
<th>Bearing</th>
<th>Load</th>
<th>Rate of Construction</th>
<th>Construction Sequence</th>
<th>Allowable Settlement</th>
<th>Waiting Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. EARTH AND ROCK FILLED DAMS</td>
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<tr>
<td>1.1 Dam Embankment</td>
<td>✓</td>
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<tr>
<td>1.2 Dam Foundation</td>
<td></td>
<td>✓</td>
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<td>1.3 Dam Abutments</td>
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<td>1.4 Appurtenant Structures</td>
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<td>2. CANALS AND PIPELINES</td>
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<td>2.1 Conveyance Structures</td>
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<td>2.2 Intake Structure</td>
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<td>2.3 Storage or Surge Tanks</td>
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<td>2.4 Receiving Structure</td>
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<td>3. DEEP EXCAVATION</td>
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<tr>
<td>3.1 Free Slope</td>
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<td>3.2 Cantilever Sheet piling</td>
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<td>3.3 Braced Excavation</td>
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<td>3.4 Anchored Retaining Wall</td>
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</table>
# Geotechnical Engineering involvement

## Engineering Structures

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<tr>
<th>Geotechnical Engineering involvement</th>
<th>Related Geotechnical Engineering Topics</th>
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</thead>
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<td>Engineering Structures</td>
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<td>4. Building and Bridge Foundations</td>
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<tr>
<td>4.1 Shallow footing</td>
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<td>4.2 Pile Foundation</td>
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<tr>
<td>4.3 Caisson or Deep massive footing</td>
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<td>4.4 Micro pile or Root pile</td>
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<tr>
<td>5. Embankments, highway and runway</td>
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<tr>
<td>5.1 Highway, Railway</td>
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<td>5.2 Runway, Taxiway and Apron</td>
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<td>5.3 Land Reclamation (Coastal)</td>
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<td>6. Tunneling</td>
<td></td>
</tr>
<tr>
<td>6.1 Rock Tunneling</td>
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<td>6.2 Underground Opening</td>
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<td>6.3 Soil Tunneling</td>
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<tr>
<td>6.4 Cut and Cover</td>
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</table>

## Engineering Structures

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<tr>
<td>7. Soil and Rock Slopes</td>
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<td>7.1 Natural Slope</td>
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<td>7.2 Cut Slope</td>
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<td>7.3 Fill Slope</td>
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<td>8. Offshore or Near shore Structures</td>
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<td>8.1 Oil Drilling Platform</td>
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<td>8.2 Jetty and quay wall</td>
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<td>8.3 Dry Dock</td>
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</tbody>
</table>
ตารางลำดับโครงสร้างและปัญหาทางวิศวกรรมปฐพีที่ต้องพิจารณาในการออกแบบ

<table>
<thead>
<tr>
<th>ลักษณะโครงสร้าง</th>
<th>Consolidation</th>
<th>Pile Capacity</th>
<th>Bearing Capacity</th>
<th>Stability</th>
<th>Lateral Earth Pressure</th>
<th>Seepage &amp; Drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>ตูโบกต์</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>คลองส่งน้ำแบบก้างคอนกรีต</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>คลองส่งน้ำแบบตามแนวราบทางคลองเดิมหรือใหม่</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
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</tr>
<tr>
<td>หันกันน้ำจากนอกและภายในพื้นที่</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
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</tr>
<tr>
<td>การยุบของน้ำเป็นพื้นที่กันดิน</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>ตูโบก (Siphon) และสะพาน</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>อุโมงค์ (Lateral)</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
</tbody>
</table>

〇 Primary Problem
□ Secondary Problem

ตารางรายละเอียดการเจาะสำรวจทางวิศวกรรมปฐพี

<table>
<thead>
<tr>
<th>ลักษณะโครงสร้าง</th>
<th>ความลึกเจาะ</th>
<th>Field Vane Shear</th>
<th>Basic Properties (Seive Analysis, Atterberg's Limit, Wn, etc.)</th>
<th>Unconfined Comp. Test</th>
<th>Triaxial on Test</th>
<th>Consolidation on Test</th>
<th>Lateral Load Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>ตูโบกต์</td>
<td>50 ม.</td>
<td>-</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>คลองส่งน้ำแบบก้างคอนกรีต</td>
<td>30 ม.</td>
<td>15 ม.</td>
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<td>✓</td>
<td>✓</td>
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<tr>
<td>คลองส่งน้ำแบบตามแนวราบทางคลองเดิมหรือใหม่</td>
<td>20 ม.</td>
<td>15 ม.</td>
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<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>หันกันน้ำจากนอกและภายในพื้นที่กันดิน</td>
<td>20 ม.</td>
<td>15 ม.</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>การยุบของน้ำเป็นพื้นที่กันดิน</td>
<td>20 ม.</td>
<td>15 ม.</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>ตูโบก (Siphon) และสะพาน</td>
<td>30 ม.</td>
<td>-</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>อุโมงค์ (Lateral)</td>
<td>30 ม.</td>
<td>-</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>