10. Compression behaviour of soils

10-1. Compression in a broader sense

Compression of soil comprises the volume reduction due to compression of constituent materials (soil particles, pore water, pore air, etc.) and consolidation (emigration of pore water). In this section, we consider only that due to consolidation. The underlying assumption is that the soil in consideration is well saturated to have very low undrained compressibility.

It is important to remember that, in soil mechanics at least, the term “compression” is often used in a broad sense, applied to any stress path which includes increases in $p'$ (see the diagram). The relevance of different compression modes to practice is well illustrated in the example of an embankment construction.

Along the centre line:

- Compression with large lateral strain allowed
  - $K_0$-compression
  - $D = 6.9$

- Compression with small lateral strain allowed
  - $D = 13.0$

In this section, we limit our focus on $K_0$- and isotropic compression, as these are conventionally simulated in laboratory as ‘consolidation tests’. The main objective of the section is to understand relationships between the compressive stress and soils’ volume changes and how they are affected by various factors.
10-2. Test apparatus

(i) $K_0$-consolidation: Oedometer

An oedometer consists of a confining ring housing a tested soil sample and a top cap with a porous stone through which pore water can escape. The vertical load may be applied by dead weights, pneumatic pressure, motor, etc.

As we have studied in Week 1, only the vertical stress is known in an oedometer. For the horizontal directions, we only know that the normal strains are zero. The horizontal stresses are expressed as $K_0 \sigma_v'$. Knowing a $K_0$ value is a big issue in soil mechanics.

There are two ways of applying vertical loads:

(a) (Conventional) step loading
This is the simpler, classical, standard method. Vertical loads are increased in steps by, for example, adding dead weights.

(b) CRS (Constant Rate of Strain) loading
The vertical strain is increased at a constant rate by means of a powerful motor. By measuring pore water pressure at the same time, the $e - \log p'$ relationship is obtained continuously.
(ii) Isotropic consolidation

Although isotropic consolidation is notionally simpler than $K_r$-consolidation, performing tests no easier. It requires high pressure, and factors such as membrane penetration need to be considered.

In this week’s lecture, the compression curves shown are all obtained for $K_r$-conditions, unless specified otherwise.

10-3. Sedimentation and normal consolidation of clayey soils

(i) Sedimentation Compression Curve

If soil is deposited uniformly in layers, it undergoes $K_r$-consolidation due to overburdens from subsequently deposited soil. So if the soil is normally consolidated (no removal of overlying layers in the past), a relationship between in-situ $e$ and in-situ $\log p'$ over a soil layer should be identical to the $e - \log p'$ relationship obtained from oedometer tests of slurry in laboratory. But usually it isn’t. The in-situ relationship, the Sedimentation Compression Curve (SCC: Skempton, 1970) comes above the laboratory $K_r$-Normal Compression Line ($K_r$-NCL) (Burland, 1990; Chandler, 2010). Let us have some more look.

Sedimentation Compression Curves of natural clayey soils (Skempton, 1970; reproduced from Chandler, 2010)
(ii) Intrinsic behaviour and void index

Intrinsic behaviour and void index are concepts put forward by Burland (1990).

Intrinsic behaviour:

Behaviour of reconstituted soil, consolidated from slurry prepared at least from the water content of 1.0~1.5 times the liquid limit ($w_L$), without ever being air- or oven-dried.

Examples of *Intrinsic Compression Lines (ICL)*
(Burland, 1990; reproduced from Chandler, 2010)

In the above diagram, the ICLs for soils with higher $w_L$ comes above those for lower $w_L$.
Because all the ICLs are essentially just a line, they can be bundled as a single relationship by normalisation using the *void index*, $I_v$.

$$I_v = \frac{e - e_{100}^*}{e_{100}^* - e_{1000}^*}$$

Intrinsic behaviour: the e – log $p'$ and $I_v$ – log $p'$ plots.

Soil A and Soil B with $I_v = 0$ and $I_v = -1$. The void index, $I_v$, is a normalised measure of the void ratio in the soil.
By re-plotting the Sedimentation Compression Curves in the $I_v - \log \sigma'_v$ form, the relative positions of the SCL (Sedimentation Compression Line: A line showing an average of SCCs of many soils) with regard to the ICL are confirmed to be higher for many different soils.

What if soils are sedimented in a column, in laboratory, the artificial SCC is still different from the natural SCC. The difference in time scales seems to matter, leading to development of natural structure in soil.

(Reproduced from Chandler, 2010)

(Locat & Lefebvre, 1982; Reproduced from Leroueil & Vaughan, 1990)

(Stallebrass et al., 2007; Reproduced from Chandler, 2010)
10-4. Normal compression of sandy soils

The compression characteristics of sandy soils are less intensively studied than those of clayey soils. For one thing, the amount of volumetric strain in sand involved in usual engineering scales is not as large as that in clay. However, significant compression is expected locally where large stress concentration is likely occur; for example, at pile tips.

An important difference in compression characteristics between clay and sand is that, for sand, the starting point of the compression curves are not unique. Without loading, you can prepare dense and loose specimens. This is not true (at least in realistic situations) with clay.

Note the following:
- Dense sand behaves as if over-consolidated.
- The apparent yield stress is very high, even if sand was not actually pre-loaded.

(Dogs Bay Sand (carbonate) (Coop, 1990)

Ham River Sand (quartz) (Coop & Lee, 1992)
10-5. Over-consolidation

Over-consolidation allows soils denser states for a given effective stress level. We have studied that over-consolidation leads to ‘apparent’ cohesion in soils, as represented by the Hvorslev surface.

In $K_0$-conditions, over-consolidation (or unloading) usually leads to a non-linear effective stress path. As a result, $K_0$-over-consolidated soil has high $K_0$ values, sometimes in excess of 1.0.
In passing, the equations proposed by Mayne and Kulhawy (1982) for expressing the unloading and recharging $K_0$ values are introduced. These are empirical equations, but sometimes useful in interpreting in-situ data or planning laboratory tests.

\[
\sigma'_v - \sigma'_h
\]

$K_0$-normal compression:

\[
K_{0,NC} = 1 - \sin\phi'
\]

(Jaky, 1944)

$K_0$-unloading:

\[
K_0 = K_{0,NC} OCR^{\sin\phi'}
\]

(Mayne and Kulhawy, 1982)

$K_0$-reloading:

\[
K_0 = K_{0,NC} \left( \frac{OCR}{OCR_{\text{max}}} \right)^{1-\sin\phi'} + \frac{3}{4} \left( 1 - \frac{OCR}{OCR_{\text{max}}} \right)
\]

(Mayne and Kulhawy, 1982)

Note that $OCR$ refers to the over-consolidation ratio, defined as

\[
OCR = \frac{\sigma'_{cv}}{\sigma'_v}
\]

Maximum effective vertical consolidation stress in the past

Current effective vertical consolidation stress
Example: London Clay

The London Clay is a very heavily over-consolidated clay. Note the $K_0$ values estimated in-situ are generally in excess of 1.0. Compare them with those for the (probably) normally-consolidated Bothkennar Clay. Because over-consolidation leads to smaller void ratio, $e$, the water content, $w$, is also very small. The water content can be as small as the plastic limit, $w_L$, and partly as a result, cracks called ‘fissures’ can develop.

$K_0$ profiles of Bothkennar Clay (Nash et al., 1992)

$K_0$ profiles of London Clay at Heathrow Terminal 5 construction site (Hight et al., 2007)

Fissures in London Clay (Hight et al., 2007)

Friends and I at Heathrow T5 site (when I was 24 years old!)
10-6. Development of natural structure

(i) Clays

There is a feature in compression characteristics that is very commonly seen in natural clays, whether normally consolidated or over-consolidated; the effective yield stress $\sigma_{vy}'$ is larger than the effective maximum consolidation stress $\sigma_{vc}'$ expected from the SSC or geological evidences.

This feature is believed to derive from what is called a ‘natural structure’ in soils. Mitchell (1976) defined ‘structure’ as

Structure = Fabric + Bonding

Once loaded beyond the yield stress, the natural structure is destroyed and the compression curve approaches the ICL.

It is debatable if this effect has really anything to do with actual bonding (i.e. cementation). The true mechanism of natural structure is not fully understood, so the vague word ‘structure’ has come to be used to describe anything that is not seen in laboratory-prepared samples.

Now, let us take time to think what practical implications this feature has.

Example: Boom Clay (Chandler, 2010)
(ii) Sands

A similar feature of natural structure is also seen in sands. Coop and his co-workers have performed many high-pressure compression tests to see the effect in uncemented, weakly cemented and strongly cemented natural sands (including sandstones).

Cemented carbonate sand (Coop & Atkinson, 1993)

Strongly cemented sand (Cuccovillo & Coop, 1997)  
Strongly cemented sand (Cuccovillo & Coop, 1999)
References