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# Consolidation Anisotropy of Some Natural Soft Soils

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## Abstract

The one-dimensional deformation behavior of eight homogeneous, natural silts and clays was studied using incremental-load oedometer tests. Similar levels of compressibility occurred for loading parallel to, and orthogonal to the cross-anisotropic plane. As expected, the horizontal ($c_{vh}$) and vertical ($c_{vv}$) coefficient of primary consolidation values reduced with increasing applied stress. The level of consolidation anisotropy was assessed in terms of the ratio $k_r$ (= $c_{vh}/c_{vv}$), with 1.0< $k_r$<2.1 for the test soils. A review of the literature and test data indicated that for many homogeneous clays, 1.2<$k_r$<2.5, with a mean $k_r$ value of typically 1.6. The $k_r$ values, which generally reduced slightly with increasing applied stress, were related to the soil fabric and plasticity characteristics. In practice, better estimates of the level of consolidation-anisotropy can be made for similar soils based on the soil descriptions, classification and the $k_r$ data presented which will lead to more reliable predictions of field consolidation rates.

***Keywords:*** *consolidation, anisotropy, soft soils, soil fabric.*

## 1 Introduction

Much of geotechnical design is driven by serviceability limit state conditions so that the ability to accurately predict amounts and rates of ground settlements under applied loads is particularly important. For example, the residual settlements of the highway embankment on soft ground (Figure 1) must be acceptable before pavement construction can commence. In general, amounts of primary consolidation settlements for inorganic deposits can be predicted fairly accurately using Terzaghi's one-dimensional consolidation theory and the pertinent consolidation properties, measured over the appropriate stress range, for the different compressible strata. However, field settlement rates are generally more difficult to predict. Natural soft soil deposits are laid down by a sedimentation process, with the preferred horizontal alignment of the plate-like mineral particles giving rise to an inherent cross-anisotropic soil fabric. Hence, the ground permeability is generally greater for horizontal than for vertical seepage conditions.

The level of permeability-anisotropy can be quantified by the ratio $k_r$, of the horizontal-to-vertical coefficient of primary consolidation values. Closely spaced vertical drains can be inserted in the soft ground (Figure 1), the effect of which is to accelerate the rate of primary consolidation settlement by shortening the drainage paths and taking full advantage of the soil anisotropy. The drain spacing is one of the most important design considerations, and is controlled to a large extent by the ground permeability, in particular for horizontal drainage conditions. However, the primary consolidation properties for design are often measured for vertical drainage conditions only, generally using oedometer tests on representative undisturbed test specimens. Consequently, in practice, design settlement predictions are often conservative, with field settlements occurring more rapidly due to preferred horizontal drainage. Other factors, including the ground stratification, which cannot be fully represented in the laboratory specimens, also contribute to the field settlement rates being underestimated.

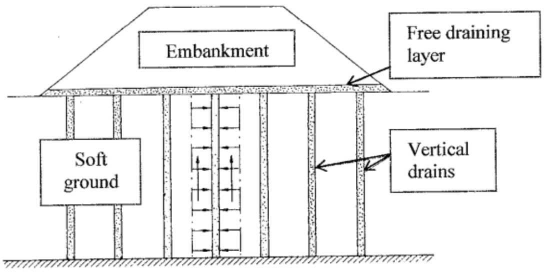



Figure 1. Embankment construction on soft ground.

The objective of the present study was to measure the level of consolidation-anisotropy present in a range of natural soft soils with different inherent fabrics. This database can be used in practice for similar soft soil deposits to more accurately predict the coefficient of primary consolidation values for horizontal drainage conditions ($c_{vh}$) from the valves measured for vertical drainage conditions ($c_{vv}$), which will lead to more accurate designs.

## 2 Test Soils

High-quality samples were taken from three soft-soil sites around Ireland (Figure 2) using a 100 mm diameter, thin-walled tube sampler, the standard sampler used in Irish practice. Samples were taken from fully saturated, homogeneous alluvial deposits at depths of between 0.7 and 5.0 m below ground level. The consolidation behavior of eight different soils, labeled [1] to [8], was studied.

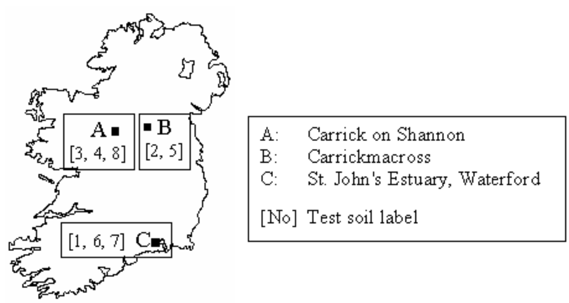



Figure 2. Locations of soft-soil sites.

The soil descriptions to BS5930 (1999), soil properties and *insitu* properties are listed in Table 1. The soils, which included silts and clays of low to extremely high plasticity, were classified using the standard plasticity chart (Figure 3). Soils [4, 7, 8] were slightly organic, with loss in mass on ignition values of approximately 4 to 6 %, measured at 440 $^{o}$C (Table 1), which gave rise to their higher plasticity characteristics.

Table 1. Description and insitu properties of test soils.

| Soil label | Depth (m) | Soil description and classification (BS5930,1999) | $w_l$ (%) | $w_p$ (%) | $I_p$ | $G_s$ | LOI (%) | Insitu properties $w_o$ (%) | $e_o$ | $\gamma$ (kN/m$^3$) | $\gamma_d$ (kN/m$^3$) | $\sigma_{vo}'$ (kPa) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| [1] | 5.0 | Very soft, mottled grey-brown, thinly laminated, clayey SILT (MH) | 64 | 44 | 20 | 2.67 | 4.1 | 40 | 1.53 | 14.5 | 10.4 | 26 |
| [2] | 2.5 | Very soft, greyey brown, medium laminated CLAY (CI) | 42 | 18 | 24 | 2.72 | - | 45 | 1.27 | 17.0 | 11.7 | 20 |
| [3] | 5.0 | Soft to firm, brown, thinly laminated CLAY with fine sand partings (CVS) | 90 | 35 | 55 | 2.70 | - | 123 | 3.93 | 12.0 | 5.4 | 12 |
| [4] | 3.5 | Soft, dark grey, fibrous organic CLAY (CEO) | 143 | 52 | 91 | 2.51 | - | 134 | 3.55 | 12.6 | 5.4 | 11 |
| [5] | 2.0 | Very soft, laminated, calcareous SILT (Marl) (MH) | 68 | 52 | 16 | 2.31 | - | 88 | 2.08 | 13.8 | 7.4 | 9 |
| [6] | 1.5 | Very soft, mottled grey-brown, thinly laminated, SILT with some shell fragments (MH) | 58 | 31 | 27 | 2.56 | 3.6 | 53 | 1.33 | 16.5 | 10.8 | 12 |
| [7] | 0.7 | Very soft, mottled grey-brown, thinly laminated, SILT with some wood fibres (MV) | 74 | 37 | 37 | 2.43 | 6.1 | 68 | 1.56 | 15.7 | 9.6 | 6 |
| [8] | 2.1 | Soft, dark grey, SILT with occassional root fibres (MEO) | 91 | 42 | 49 | 2.45 | - | 72 | 1.83 | 14.6 | 8.5 | 12 |

It is difficult to quantify the soil fabric using routine laboratory tests. Often the geotechnical designer has to estimate the actual level of consolidation-anisotropy based solely on the reported soil descriptions and engineering judgment. Hence, the soil fabric, if evident, was described from a visual inspection only of the extruded tube samples. Features such as laminations, fine fissures filled with silt or sand, organic inclusions and fine root-holes were carefully recorded.

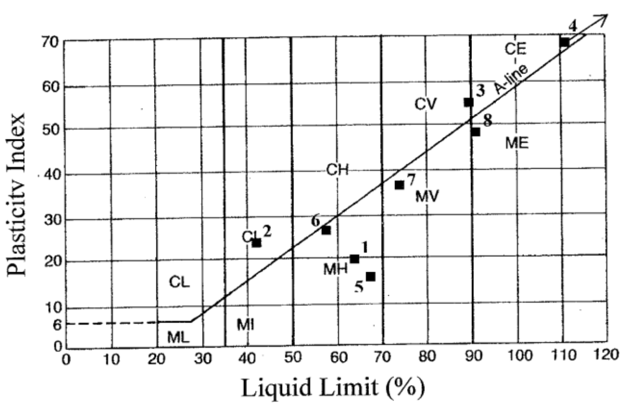



Figure 3. Test soils plotted on plasticity chart (BS5930, 1999).

## 3 Consolidation Tests

Standard oedometer consolidation tests were conducted on undisturbed specimens, 76.2 mm diameter by 19.0 mm high. The oedometer apparatus facilitated one-dimensional consolidation of the specimens under a condition of zero lateral strain. Hence, pairs of identical test specimens were carefully prepared for each soil from the 100 mm diameter tube samples such that the axis of one specimen was parallel to, and the axis of the second specimen orthogonal to the cross-anisotropic plane (Figure 4). The values of the coefficient of primary consolidation for horizontal ($c_{vh}$) and vertical ($c_{vv}$) drainage conditions, respectively, were determined in this manner. The specimen pairs were taken from the same location and depth in the ground, and hence the same *insitu* vertical effective stress ($\sigma_{vo}$') given in Table 1.

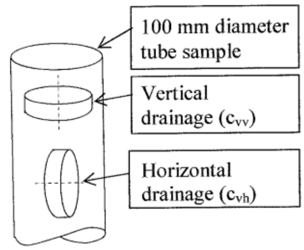



Figure 4. Orientation of specimen pairs for oedometer tests.

Each oedometer test comprised five incremental-load stages, each of which was of 24 hours duration. The tests commenced with a low applied stress of 12.5 kPa since the soils were of soft consistency. The applied stress was increased at the end of each load stage using the standard load increment ratio of unity until a final stress of 200 kPa had been applied. This load regime covers the stress range experienced by the ground due to the construction of an embankment of up to 10 m in height.

## 4 Experimental Results

### 4.1 Compressibility

The vertical compressibility of the soils, expressed in terms of void ratio-logarithm applied stress plots (e-log $\sigma$), is shown in Figure 5. The compression index values, calculated from the slope of the e-log $\sigma$ curves over the linear range, are given in Table 2. As expected, the more plastic soils existed at higher void ratios and had higher compression indices. Each pair of specimens had practically the same initial void ratio which confirmed that the specimen pairs were initially identical. Furthermore, similar levels of compressibility for horizontal and vertical drainage conditions indicate that, overall, the compressibility was largely independent of the level of consolidation-anisotropy.

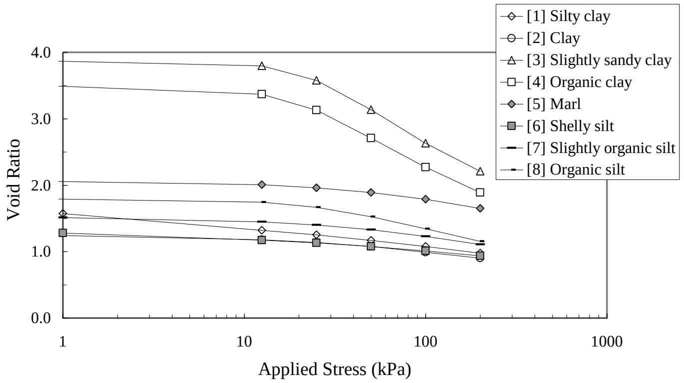



Figure 5. Void ratio-logarithm applied stress plots, compressibility.

Table 2. Compression index ($C_c$) values of test soils.

| **Label** | [1] | [2] | [3] | [4] | [5] | [6] | [7] | [8] |
|---|---|---|---|---|---|---|---|---|
| **Compression index** | 0.34 | 0.29 | 1.4 | 1.4 | 0.47 | 0.30 | 0.39 | 0.63 |

## 4.2 Consolidation Rate

Typical strain versus logarithm-time consolidation data are shown in Figure 6. The specimen strains at the start of each load stage were matched in plotting this data to facilitate more direct comparison of the *S*-shaped consolidation curves for the different drainage directions. The strains occurred more rapidly for drainage parallel to the cross-anisotropic fabric. The strains in the specimen pairs were broadly similar at the end of each load stage. Slightly larger strains occurred for the more plastic soils when the load direction was orthogonal to the cross-anisotropic plane, although for practical purposes, this difference was insignificant.

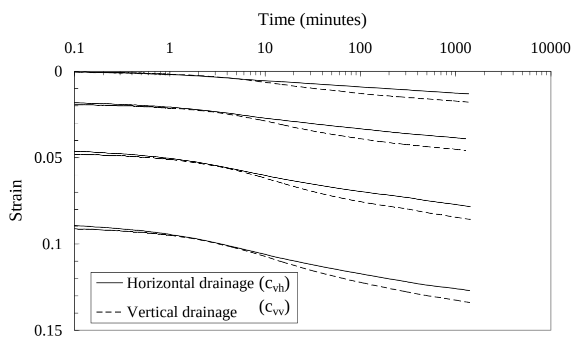



Figure 6. Oedometer strain-logarithm time plots for test soil [7].

The coefficient of primary consolidation values ($c_{vv}$ and $c_{vh}$) of the soils were calculated for each stress increment (Table 3) using Terzaghi's one-dimensional consolidation theory, and applying the logarithm-of-time curve fitting method (Casagrande and Fadum, 1940) to the settlement-time data. The time necessary to achieve a given degree of consolidation for each load stage was more clearly identified using the logarithm-of-time, rather than the square-root-time (Taylor, 1942), curve fitting method. Su's Maximum-Slope curve fitting method (Das, 1997), in which the data is also plotted on a logarithm timescale, was used when the consolidation curves did not match the conventional *S*-shape curve. The values of the consolidation parameters for the first load stage were treated with caution in the analysis since the compression-time plots indicated that the test specimens had been slightly disturbed.

## 4.3 Consolidation Anisotropy

As expected, the $c_{vh}$ and $c_{vv}$ values decreased with increasing applied stress (Table 3). As the soils compressed, the void ratio decreased creating a new stress-induced fabric. The levels of consolidation-anisotropy, given in terms of the $k_r$ ratios (Table 3), can be reconciled with the soil descriptions, and are generally a function of the soil fabric and plasticity. A $k_r$ value of one indicates isotropic consolidation properties, for example soils [5, Marl] and [6, shelly silt]. Higher $k_r$ values were associated with more marked soil fabrics, for example laminated clay [2], and higher plasticity soils [4, 7]. In general, the levels of consolidation-anisotropy measured for the soils appeared to be stress-level dependant, with the $k_r$ ratios generally reducing marginally with increasing applied stress.

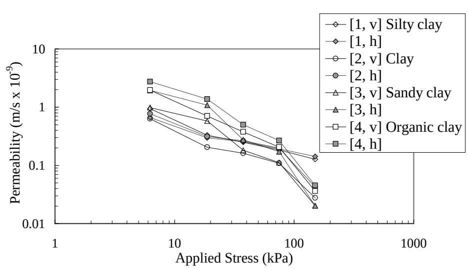
The laboratory coefficient of permeability ($k_{vv}$ and $k_{vh}$) values at 21 $^oC$ were calculated using Eq. (1), and plotted against logarithm applied stress (Figure 7). On a log-log plot, the soil permeability decreased essentially linearly with increasing applied stress. Significant reductions in the coefficient of permeability values for soils [3] and [4, 8] were most likely due to closure of the fine sand-filled fissures and root holes, respectively, which formed part of the *insitu* fabric.

$$k_{vv} = m_{vv}\, c_{vv} \qquad (1)$$

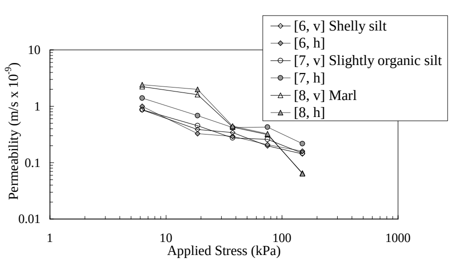
where $m_{vv}$ = Coefficient of volume change,
and $c_{vv}$ = Coefficient of primary consolidation,
both laboratory measured values for sample loading orthogonal to the cross-anisotropic plane.

Table 3. Coefficient of primary consolidation values for different drainage directions.

| **Label** | $c_{vv}$ ($m^2$/year) | $c_{vh}$ | $k_r$ | $c_{vv}$ ($m^2$/year) | $c_{vh}$ | $k_r$ | $c_{vv}$ ($m^2$/year) | $c_{vh}$ | $k_r$ | $c_{vv}$ ($m^2$/year) | $c_{vh}$ | $k_r$ | $c_{vv}$ ($m^2$/year) | $c_{vh}$ | $k_r$ | **Mean $k_r$** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| [1] Silty clay | 0.4 | 0.4 | 1.1 | 0.5 | 0.6 | 1.2 | 0.5 | 0.7 | 1.3 | 0.7 | 0.9 | 1.3 | 0.8 | 1.1 | 1.3 | 1.2 |
| [2] Clay | 0.4 | 0.9 | 2.0 | 0.3 | 0.7 | 2.0 | 0.4 | 0.7 | 1.9 | 0.4 | 0.8 | 1.7 | 0.2 | 0.3 | 1.5 | 1.8 |
| [3] Sandy clay | 1.5 | 3.1 | 2.0 | 0.6 | 1.0 | 1.7 | 0.1 | 0.2 | 1.6 | 0.1 | 0.2 | 1.6 | 0.1 | 0.1 | 1.2 | 1.5 |
| [4] Organic clay | 2.0 | 4.6 | 2.3 | 0.5 | 1.0 | 1.9 | 0.3 | 0.4 | 1.2 | 0.3 | 0.3 | 1.2 | 0.1 | 0.1 | 1.2 | 1.6 |
| [5] Marl | 370 | 370 | 1.0 | 180 | 180 | 1.0 | 90 | 90 | 1.0 | 100 | 90 | 0.9 | 33 | 33 | 1.0 | 1.0 |
| [6] Shelly silt | 0.8 | 0.9 | 1.0 | 0.8 | 0.8 | 0.9 | 1.0 | 0.9 | 0.9 | 1.0 | 1.1 | 1.0 | 1.2 | 1.4 | 1.1 | 1.0 |
| [7] Organic silt | 1.3 | 3.1 | 2.4 | 1.0 | 2.0 | 2.0 | 0.8 | 1.4 | 1.9 | 1.0 | 1.9 | 1.9 | 0.9 | 1.5 | 1.7 | 2.0 |
| [8] Organic silt | 2.9 | 2.8 | 1.0 | 2.9 | 3.0 | 1.0 | 0.7 | 0.8 | 1.2 | 0.7 | 0.8 | 1.1 | 0.3 | 0.3 | 1.1 | 1.1 |
| **Stress range (kPa)** | 0 to 12.5 | | | 12.5 to 25 | | | 25 to 50 | | | 50 to 100 | | | 100 to 200 | | | |




(a) Test soils [1] to [4].




(b) Test soils [6] to [8].

Figure 7. Calculated permeability ($k_{vv}$ and $k_{vh}$) versus logarithm applied stress:
v = vertical drainage, h = horizontal drainage.

## 4.4 Secondary Compression

The effect of the inherent soil fabric on the long-term deformational response of the soils, which is quantified in terms of the coefficient of secondary compression ($C_{sv}$ and $C_{sh}$) values, was also studied. The $C_{sv}$ and $C_{sh}$ values given in Table 4 were calculated as the specimen strains, based on the original specimen height of 19.0 mm, that occurred over one log cycle of time for the linear final part of the compression versus logarithm-of-time curves.

Table 4. Secondary compression values for soils under different loading conditions.

| Label | $C_{sv}$ | $C_{sh}$ | h : v ratio | $C_{sv}$ | $C_{sh}$ | h : v ratio | $C_{sv}$ | $C_{sh}$ | h : v ratio | $C_{sv}$ | $C_{sh}$ | h : v ratio | $C_{sv}$ | $C_{sh}$ | h : v ratio | Mean ratio |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| [1] Silty clay | 0.005 | 0.004 | 0.8 | 0.004 | 0.004 | 0.9 | 0.004 | 0.004 | 1.0 | 0.004 | 0.004 | 0.9 | 0.005 | 0.006 | 1.1 | 0.9 |
| [2] Clay | 0.003 | 0.002 | 0.9 | 0.003 | 0.003 | 0.9 | 0.003 | 0.003 | 0.9 | 0.003 | 0.003 | 1.0 | 0.003 | 0.003 | 1.1 | 0.9 |
| [3] Sandy clay | 0.003 | 0.004 | 1.3 | 0.012 | 0.014 | 1.2 | 0.018 | 0.017 | 1.0 | 0.017 | 0.016 | 1.0 | 0.013 | 0.012 | 0.9 | 1.1 |
| [4] Organic clay | 0.005 | 0.004 | 0.8 | 0.011 | 0.011 | 1.0 | 0.016 | 0.020 | 1.2 | 0.015 | 0.016 | 1.1 | 0.012 | 0.015 | 1.2 | 1.1 |
| [5] Marl | 0.002 | 0.002 | 0.9 | 0.002 | 0.002 | 1.1 | 0.003 | 0.003 | 1.0 | 0.003 | 0.003 | 0.9 | 0.003 | 0.004 | 1.1 | 1.0 |
| [6] Shelly silt | 0.003 | 0.003 | 1.2 | 0.003 | 0.003 | 1.0 | 0.004 | 0.004 | 1.0 | 0.004 | 0.004 | 1.0 | 0.004 | 0.005 | 1.2 | 1.1 |
| [7] Organic silt | 0.003 | 0.003 | 0.9 | 0.004 | 0.003 | 0.8 | 0.005 | 0.004 | 0.8 | 0.008 | 0.008 | 1.0 | 0.009 | 0.007 | 0.8 | 0.9 |
| [8] Organic silt | 0.003 | 0.003 | 1.1 | 0.010 | 0.011 | 1.1 | 0.010 | 0.011 | 1.1 | 0.011 | 0.012 | 1.1 | 0.010 | 0.012 | 1.2 | 1.1 |
| **Stress range (kPa)** | 0 to 12.5 | | | 12.5 to 25 | | | 25 to 50 | | | 50 to 100 | | | 100 to 200 | | | 1.0 |

As expected, the amounts of secondary compression were greater for the higher plasticity soils [3, 4, 8]. The coefficient of secondary compression values appear to be largely independent of the applied stress level, considering that final specimen strains of between 15 and 38 % had occurred. Minor discrepancies were due to the difficulty in interpreting the strain-time settlement data using the curve fitting methods. Moreover, for a given soil, the $C_{sv}$ and $C_{sh}$ values corresponding to a particular stress increment were approximately equal indicating that secondary compression was largely independent of the level of consolidation-anisotropy.

## 5 Discussion

Typical $k_r$ ratios associated with the fabric of homogeneous soft soil deposits reported in the literature are listed in Table 5. $k_r$ ratios of approximately two (Morgenstern and Tchalenko, 1967), and 1.3–1.7 (Olsen, 1962) were measured for kaolinite for applied stresses of up to 400 kPa, and 400 kPa to 25.6 MPa, respectively. Olsen (1962) also reported that $0.9< k_r<4.0$ for illite and Boston blue clay over the stress range 400 kPa to 25.6 MPa.

Table 5. Anisotropy consolidation ratios ($k_r$) reported for different soils.

| Soil type | $k_r$ ratio | Reference |
|---|---|---|
| Peaty silt | 1.2 – 1.7 | Tsien (1955) |
| Plastic marine clay | 1.2 | Lumb and Holt (1968) |
| Soft clay | 1.5 | Basett and Brodie (1961) |
| Soft marine clay | 1.05 | Subbaraju (1973) |
| Boston blue clay | 0.7 – 3.3 | Haley and Aldrich (1969) |
| Singapore marine clay | 2.0 – 3.0 | Chu et al. (2002) |
| Bothkennar soft clay | 1.0 – 1.3 | Little et al. (1992) |

In general, Tables 3 and 5 indicate that for most natural homogeneous clays, $1.2<k_r<2.5$, with a mean $k_r$ value of approximately 1.6. Since soils with similar general stress history and fabric have similar permeability-applied pressure relationships, better estimates of the level of consolidation-anisotropy can be made for other soils taking into consideration the soil description, classification and *insitu* state of vertical effective stress. The soil description must include that of the soil fabric, observed from a visual inspection. More accurate predictions of the rates of primary consolidation can then be made in cases where the consolidation properties have been measured in the laboratory for vertical drainage conditions only.

In general, the level of consolidation-anisotropy of the test soils reduced only slightly over the applied stress range 12.5 to 200 kPa, even though final specimen strains of between 15 and 38 % had occurred. Leroueil et al. (1990) also found that for natural clays the permeability-anisotropy did not change significantly for specimen strains of up to 25 %.

## 6 Summary and Conclusions

The one-dimensional deformation behavior of eight homogeneous, alluvial silts and clays was studied using incremental-load oedometer tests on identical pairs of undisturbed specimens, prepared such that the axis of one specimen was parallel to, and the axis of the second specimen orthogonal to the inherent cross-anisotropic plane. Some of the soils were slightly organic, with loss in mass on ignition values of between 4 and 6 %.

As expected, the more plastic soils had higher initial void ratio, compression index and secondary compression values. Their compressibility was also marginally greater when the load direction was orthogonal to the cross-anisotropic plane, although overall, similar levels of compressibility occurred for the two loading conditions. The coefficient of secondary compression values, $C_{sv}$ and $C_{sh}$, were approximately equal for a given soil, and were generally independent of the applied stress level.

As expected, the horizontal ($c_{vh}$) and vertical ($c_{vv}$) coefficient of primary consolidation values reduced as the void ratio decreased with increasing applied stress. The level of consolidation anisotropy was assessed in terms of the ratio $k_r$ (= $c_{vh}/c_{vv}$), with $1.0< k_r<2.1$. A review of the literature and test data indicated that for many homogeneous clays, $1.2< k_r<2.5$, with a mean $k_r$ value of typically 1.6. The $k_r$ values were related to the soil fabric and plasticity characteristics, for example $k_r = 1.0$ (isotropic) for the marl and shelly silt samples. Soils of higher plasticity and with more defined fabrics, for example the laminated silty clays tested, had $1.6< k_r<2.1$. In general, the level of consolidation-anisotropy reduced slightly with increasing applied stress. The laboratory permeability values also decreased linearly with increasing applied stress, when plotted on a log-log scale. Larger reductions in permeability with increasing applied stress occurred for soils in which fine sand-filled fissures and root holes were present.

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