Geotechnical and Geological Engineering, 1996, **14** 341–352

TECHNICAL NOTE

Some mechanical properties of reconstituted Boom clay

A. BOUAZZA*, W.F. VAN IMPE† and W. HAEGEMAN†

**Monash University, Department of Civil Engineering, Clayton, Melbourne, Victoria 3168, Australia*
†University of Ghent, Soil Mechanics Laboratory, Technologiepark 9, B-9052 Zwijnaarde, Belgium

Received 27 October 1995
Accepted 22 August 1996

Summary

The mechanical behaviour of reconstituted normally consolidated Boom clay was examined in a series of laboratory triaxial stress path tests. The aim was to establish some basic characteristics of this soil. The compressibility of the reconstituted Boom clay was found to be moderate, corresponding to the soils of the same plasticity. The results indicated also that the destructured Boom clay exhibited a brittle behaviour. The undrained secant stiffness was found to vary with strain level and also to be dependent on the consolidation pressure.

Keywords: Critical state, normally consolidated, reconstituted clay, undrained tests, Boom clay

Introduction

The north-east of Belgium is characterized by a thick deposit of overconsolidated marine clay. The deposit is known as Boom clay and belongs to the Oligocene series (De Beer, 1967). It is also found at great depth under almost the entire onshore and offshore area of The Netherlands (Shokking and Nooy van der Kolff, 1995). The study was undertaken in connection with the building of a tunnel near Antwerp, Belgium (De Beer, 1967) and an investigation into the technical feasibility of constructing a nuclear-waste repository facility in the region of Mol, in the east of Belgium (Mair *et al.*, 1992). Both projects required more knowledge about the properties of Boom clay. Previous laboratory work on intact samples of Boom clay, carried out at high pressures, has shown that they are characterized by a very stiff response and are close to peak resistance after an axial strain of about 2% with strength reducing after large axial straining.

The present note reports some work carried out on reconstituted samples. The aims were to examine some of the basic mechanical properties of the Boom clay. The research consisted mainly of undrained triaxial compression tests on samples reconstituted from a slurry. Similar work to examine the mechanical properties of reconstituted marine clay has been carried out recently and in the past (Rampello and Silvestri, 1993; Rampello *et al.*, 1993; Coop *et al.*, 1995; Guerriero *et al.*, 1995). The present results are part of a larger inter-university research programme into the mechanical properties of Boom clay, conducted in 1993 and 1994.

Samples preparations and testing procedures

Block samples of Boom clay were obtained from the vicinity of Antwerp where it outcrops. The clay was dried in air at room temperature and pulverized to pass a 0.25 mm sieve. The dried soil was mixed with water to form a slurry having a water content (w) of 80% corresponding approximately to $w \approx 1.26\, w_L$, where w_L is the Liquid Limit. The slurry was one-dimensionally compressed to a σ'_v of 100 kPa, a state where the samples could be handled without disturbance. The samples were tested as received from the coordinating laboratory. The triaxial samples were usually extruded, trimmed, and placed in the triaxial cell at a moisture content that varied between 33% and 36%. The basic properties of the samples used for the present work were at a liquid limit (w_L) of 64%, and a plastic limit (w_p) of 27%. The grading was 3% sand, 64% silt, and 33% clay. The calcium carbonate ($CaCO_3$) content of the sample was 4.6%, and its colour was grey. The samples belong to the BK3 subdivision of Boom clay proposed by Schittekat *et al.* (1983) which corresponds to the lower part of the 'grey clay' (Waasland clay) described by Vandenberghe (1978).

The tests were carried out in a stress-path triaxial cell. The samples were 38 mm in diameter and 90 mm long. The stress-path cell had conventional instrumentation consisting of standard pore pressure transducers and an internal load cell. An external displacement transducer was used to measure axial strains. The samples were consolidated isotropically in the apparatus to the required initial states with a back pressure of 100 kPa and sheared undrained following a path of constant mean stress p, and also in a standard manner (σ'_3 = constant). The analysis of the stress–strain behaviour will be in terms of the pair of stress parameters $p' = (\sigma'_a + 2\sigma'_r)/3$ and $q = \sigma'_a - \sigma'_r$ (Wood, 1984), where the subscripts a and r refer to axial and radial directions respectively. The properties of the reconstituted soil will be referred to as intrinsic and denoted by an asterisk (*) (Burland, 1990).

Discussion

Figure 1 shows results from typical one-dimensional compression tests (1DNCL) and isotropical compression tests (INCL) plotted together in the compression plane v:lnp', where v is the specific volume ($v = 1 + e$) and p' is the mean effective stress. The compression curve obtained by isotropic consolidation is a straight line which can be represented by:

$$v = N - \lambda\, ln\, p' \tag{1}$$

where $N = 2.63$ and $\lambda = 0.129$, λ being the slope of the loading line and N the value of v on the isotropic normal compression line at $p' = 1$ kPa.

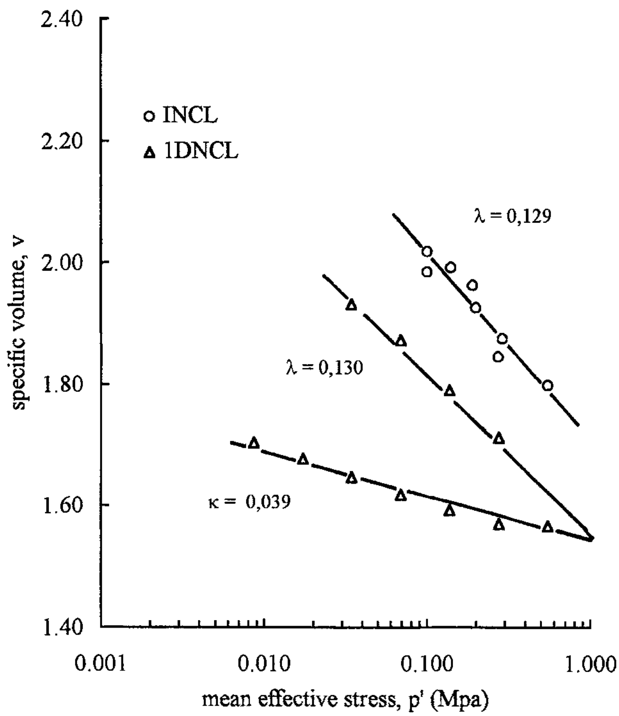

Fig. 1. Isotropic and one-dimensional compression of reconstituted Boom clay

The intrinsic one-dimensional compression and swelling curve have also been approximated by linear representations. The slope λ (= 0.130) of the loading line is almost identical to the slope of the isotropic loading line. This value of λ is close to the value reported for London clay ($\lambda = 0.16$) (Atkinson, 1994) and identical to the value found for the Marino clay of the same origin ($\lambda = 0.128$) (Guerriero *et al.*, 1995). Moreover, the relationship between λ and the plastic index *PI* ($\lambda/PI = 1/275$) is comparable to the ratio reported for London clay ($\lambda/PI = 1/280$) (Atkinson, 1994). It can also be noted that the one-dimensional compression line lies below and parallel to the isotropic loading line.

The unloading line or swell path is approximated by a straight line given by

$$v = v_{\kappa} - \kappa \ ln \ p' \tag{2}$$

where κ, the gradient of the swelling line = 0.039. This gives a value of λ/κ of 3.3 which is in the range of the values for clayey soils (2 to 5) given by Schofield and Wroth (1968).

Figures 2 and 3 show the stress–strain and pore water pressure response of selected undrained triaxial tests, initially isotropically consolidated at confining pressures in the range of 30 to 400 kPa. The stress–strain curves of the reconstituted samples exhibit a brittle behaviour with a distinct undrained peak strength. All the samples initially showed a stiff response. A slow continuous decrease of q is also observed (after peak conditions) with an increase of axial strain. This is coupled with a slight increase in the excess pore water pressure up to peak deviator stress before the end of the test, suggesting they are appreaching the critical state. The behaviour appeared to be contractant right from the

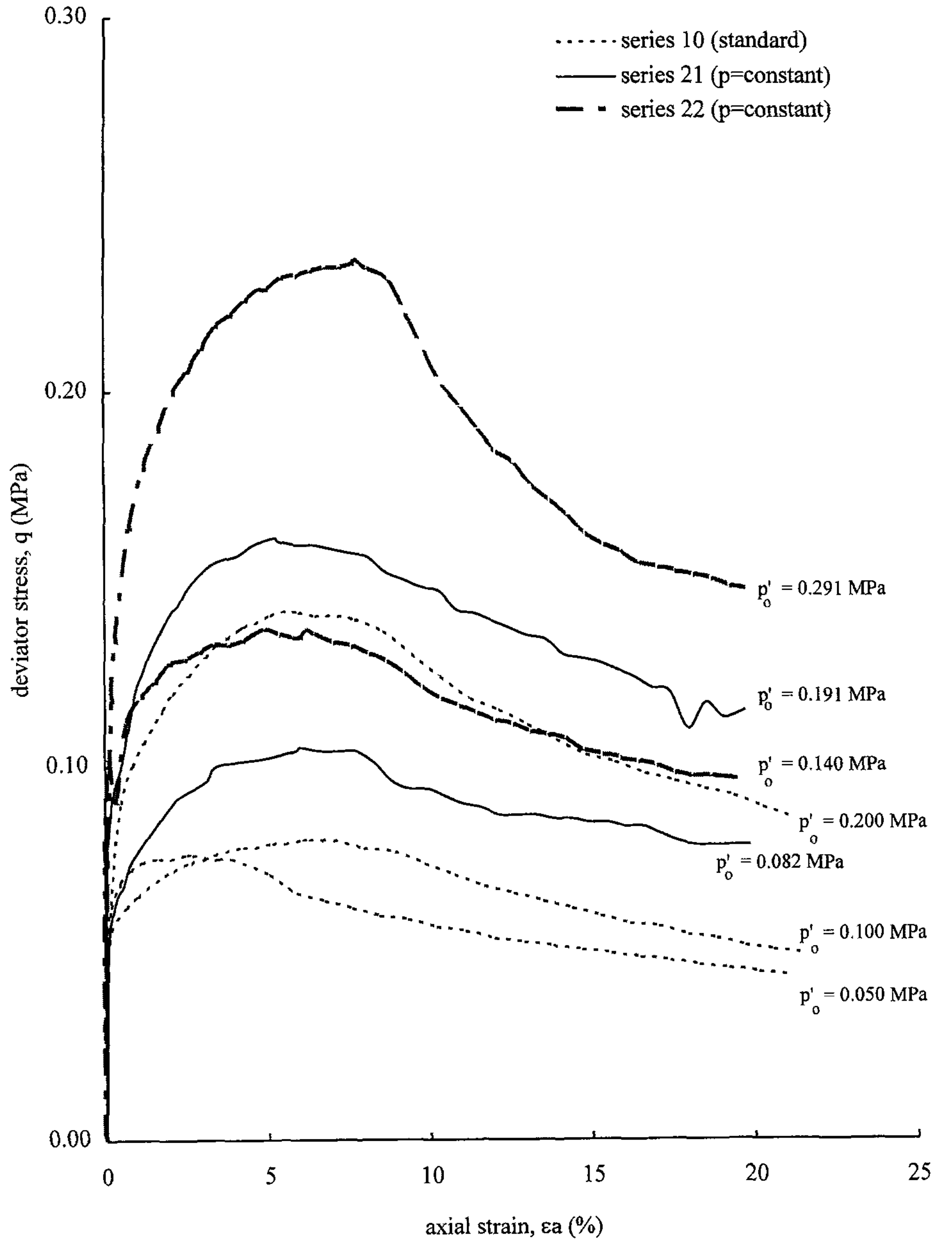

Fig. 2. Typical stress–strain curves observed in the tests on reconstituted isotropically consolidated Boom clay

beginning of the shearing stages of the tests as demonstrated by the positive pore water pressure changes. The brittle shapes of the $q - \varepsilon_a$ curves are not typical of normally consolidated reconstituted samples sheared from isotropic conditions. This aspect is discussed below.

The effective stress paths are plotted in Fig. 4. Taking the curve marked (*a*) as an

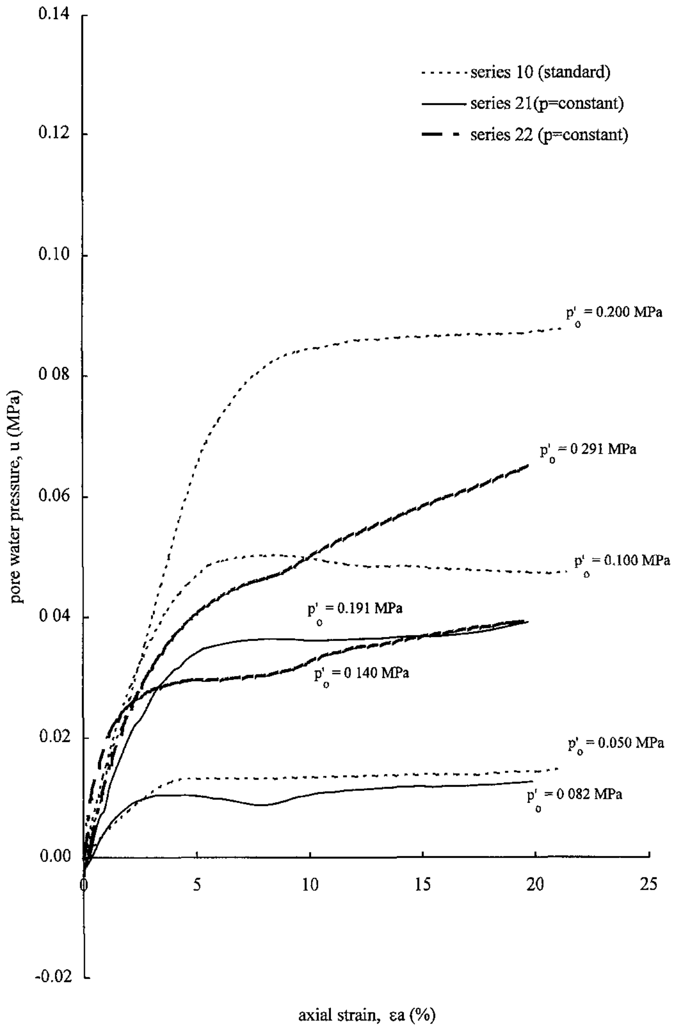

Fig. 3. Typical pore pressure–strain curves observed in the tests on reconstituted isotropically consolidated Boom clay

example, it can be observed that the maximum deviator stress is reached at point B where pore pressure is still rising. The path then curves sharply down to the left to point C to reach the end of the test. It is also worth noticing that in most samples part of the path BC coincides with an increase in the stress ratio (q/p') over a certain range of strain (2 to 4%) after $q_{\max}$ (Fig. 5). Although the test has gone beyond the maximum q value the sample is still experiencing hardening. After that portion of the stress path BC, a drop in the (q/p') ratio is noticed. The observed trend of the state paths is not typical of normally consolidated reconstituted samples sheared undrained from isotropic stress conditions. In-

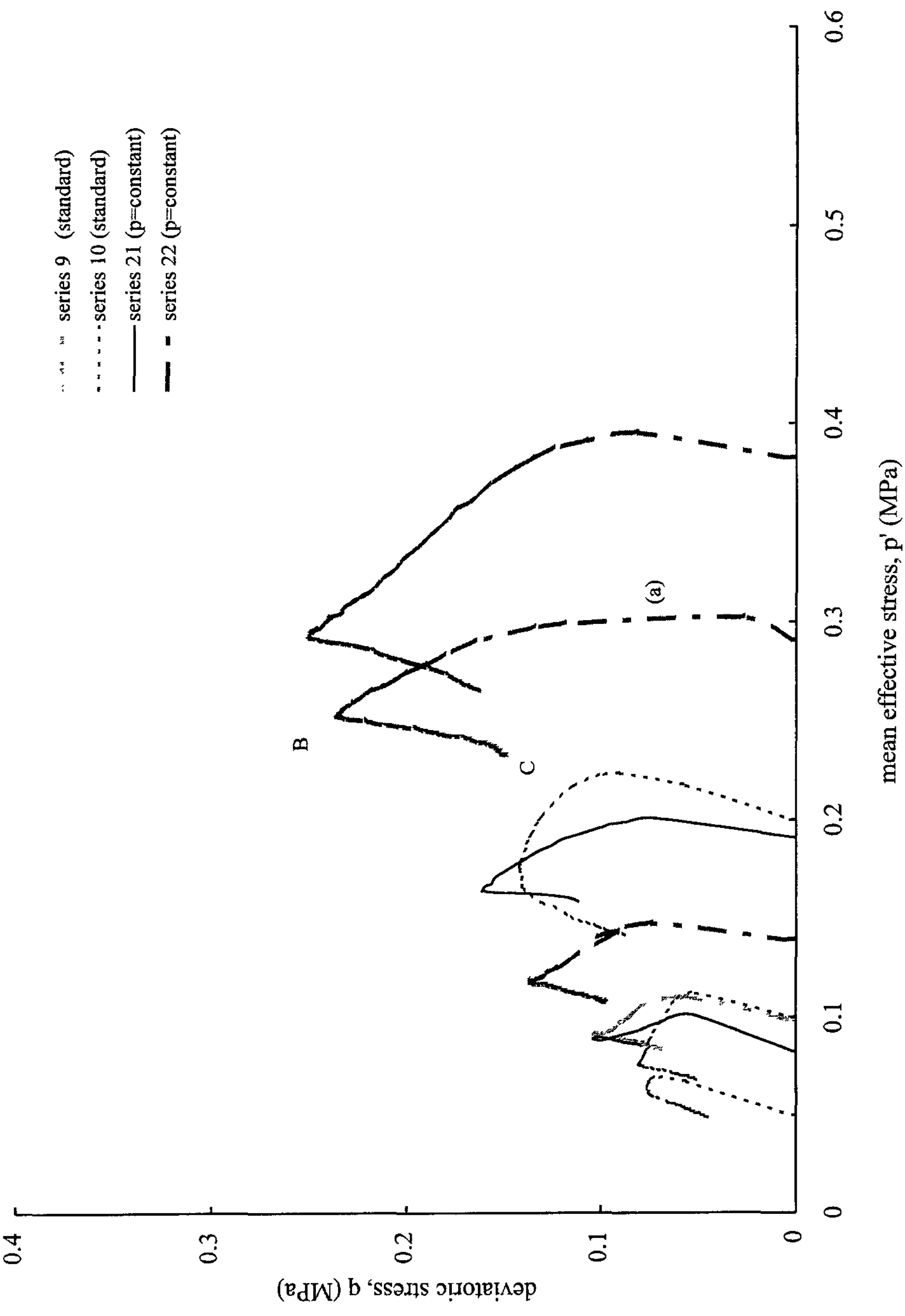

Fig. 4. Stress paths for reconstituted isotropically consolidated Boom clay

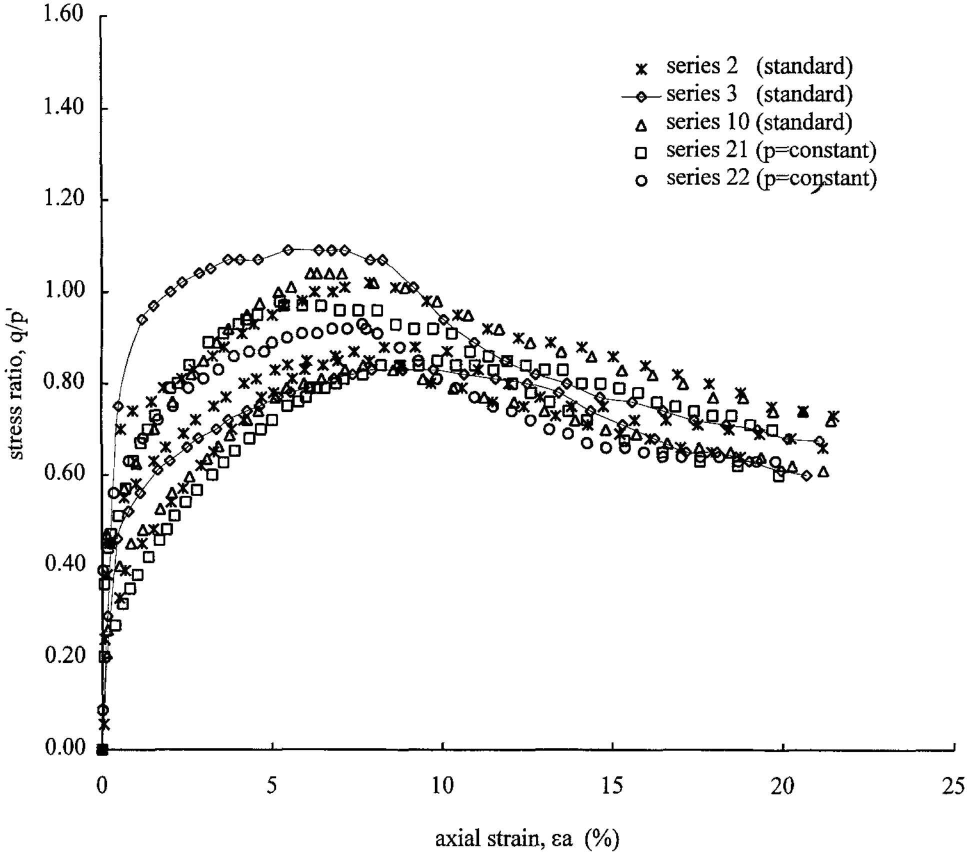

Fig. 5. Typical variation of stress ratio with axial strain for reconstituted isotropically consolidated Boom clay.

deed, evidence (as reported in the literature) from undrained tests on saturated samples of clay is that the effective stress path curves to the left immediately at the start of the test which is not the case in the present tests. Also the sharp change in the stress paths at peak deviator stress indicates more brittle behaviour than would be expected on normally consolidated clay.

End of test points for each sample are plotted in $q : p'$ space in Fig. 6. At medium stress levels, the intrinsic value of the shear angle represented by the end of tests, ϕ'^{*}, is equal to 18.5° corresponding to an $M^{*} = q/p'$ value of 0.71, the intrinsic cohesion intercept, c'^{*}, being around 0.01 MPa. The M^{*} (end of tests) value obtained experimentally for the investigated stress interval is very close to the value given by $M^{*} = \phi'^{*}/25$ (critical state) proposed by Wood (1990).

Figure 7 shows the variation of stress ratio (q/p') with mean effective stress p' at the end of the tests for triaxial compression of reconstituted Boom clay. These results show that the stress ratio at the end of the tests decreases with increasing mean effective stress. This is an unusual result as the critical state parameter M^{*} for normally consolidated soils is regarded as constant. The critical state is defined as the state during which shear stress occurs without further changes in volume or effective stress. Such a condition does not

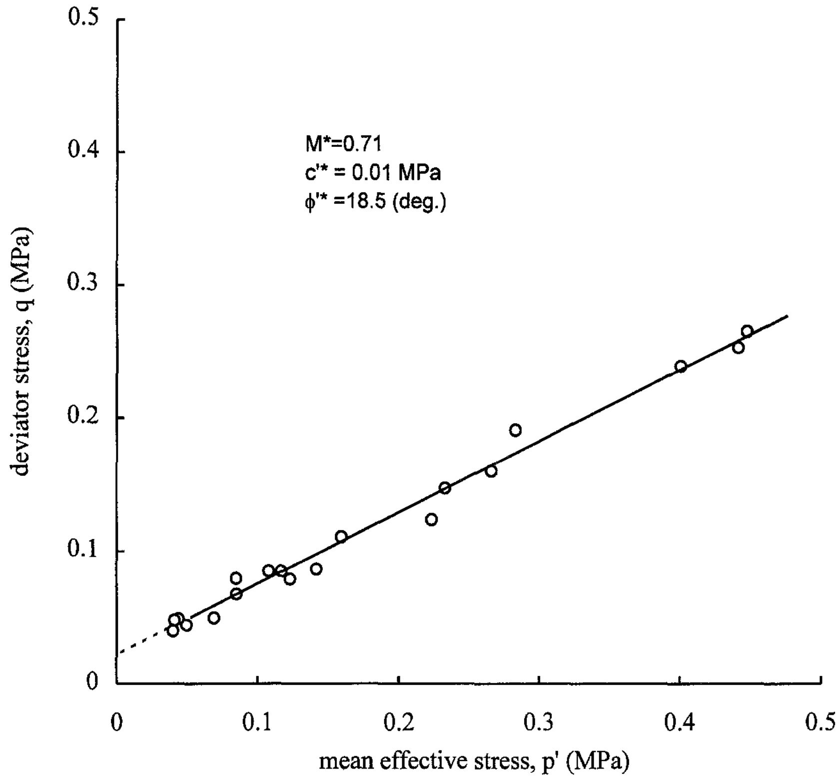

Fig. 6. Variation of end of tests state stress ratio with mean effective stress for reconstituted isotropically consolidated Boom clay

appear to have been achieved for some of the tests reported here as deviator stress and pore water pressures are still changing at the end of the tests. The results obtained on Boom clay by Coop *et al.* (1995) showed that the reduction in M^* was clearly distinguishable only at high values of mean effective stress; in the stress range investigated in the present paper the data reported by Coop *et al.* (1995) are scattered around a constant value of M^*. To eliminate the influence of different void ratios, the stress invariants q and p' were normalized with respect to the equivalent consolidation pressure p'_e (Fig. 8) for isotropically compressed samples following the procedure proposed by Atkinson and Bransby (1978). The normalized stress paths of constant p tests are initially close to vertical whereas those of the standard undrained tests 9 (series 1 and 10) are initially inclined to the vertical. The test series 21 and 22 reach higher (q/p'_e) ratio than the others; these are the tests which reached the highest pressure. In triaxial compression, the undrained response should be independent of the applied total stress path. It is possible that these differences reflect difficulties ensuring that the samples and pore pressure measuring system were fully saturated and sufficiently stiff to give a true 'undrained' response. All the tests shown in Fig. 8 tend to come together at a coordinate of approximately $(p'/p'_e = 0.7,\ q/p'_e\ 0.4)$. These are close to the values proposed by Wood (1990) for critical state line coordinates (e.g. $p'/p'_e = 2^{-\Lambda}$ (= 0.61), and $q/p'_e = M^*\ 2^{-\Lambda}$ (= 0.44)) where $\Lambda = (\lambda - \kappa)/\lambda$.

Figure 9 shows the variation of the undrained secant stiffness (E^u_{sec}), normalized with

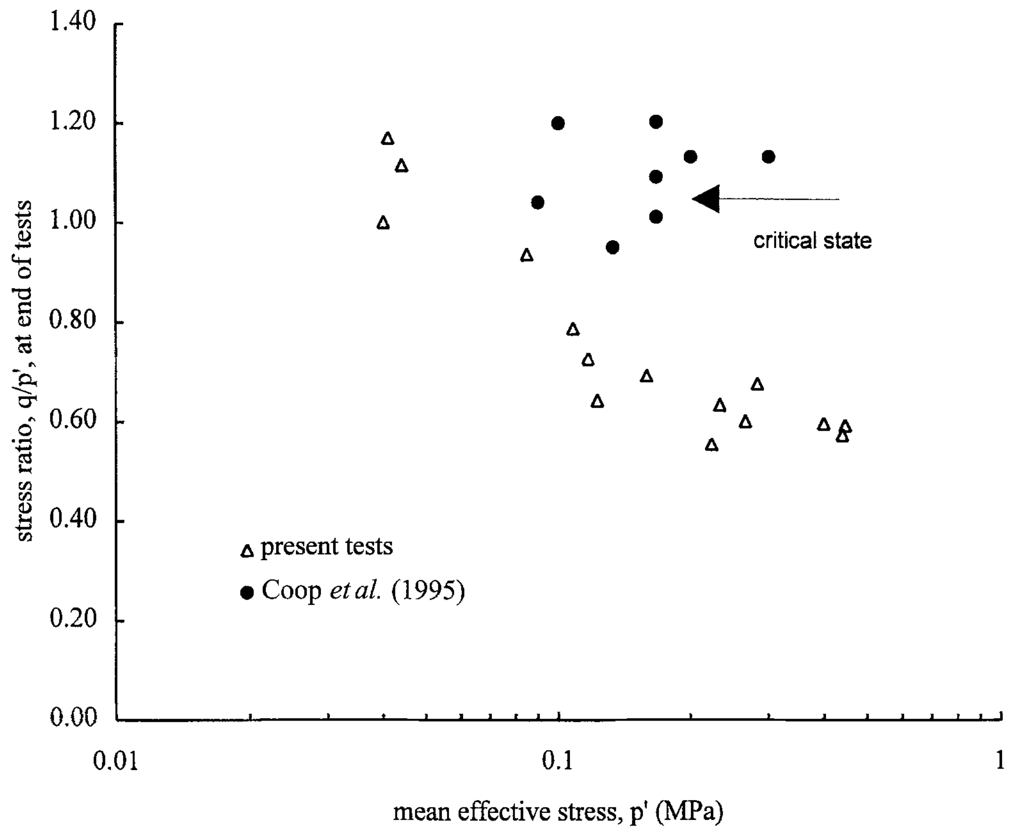

Fig. 7. State of end of tests line for reconstituted isotropically consolidated Boom clay

respect to the (pre-shear) mean effective pressure p'_0, with axial strain ε_a. The logarithmic strain axis is used to allow the very stiff initial response of the soil to be observed. It should be pointed out that in the tests described, the stiffnesses were determined from strain measurements derived from external axial displacement readings which were corrected to allow for compliance of the test system. Nevertheless, Baldi *et al.* (1988) and Atkinson and Sallfors (1991) correctly stress that measurements of axial strain made in the conventional manner may lead to erroneous values of soil stiffnesses especially at small strains. The results show a very clear trend of stiffness variation with strain level, the stiffness dropping to small values at large strains between 1 and 10% corresponding to failure. For small strains between 0.1 and 1%, the secant stiffness decreases rapidly clearly indicating non-linear behaviour. There is also evidence that E^u_{sec} is dependent on the consolidation pressure p'_0. Stiffness was also found to vary with strain levels (Mair *et al.*, 1992) for undisturbed Boom clay samples.

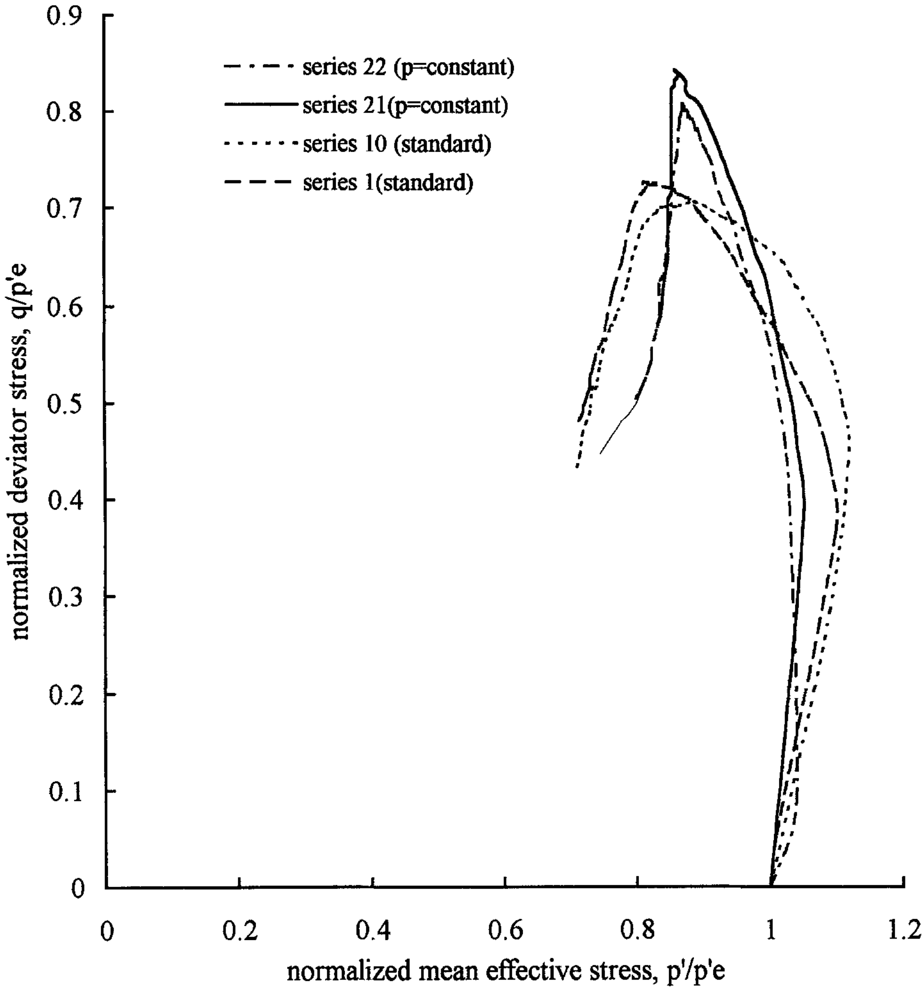

Fig. 8. Normalized stress path for reconstituted isotropically consolidated Boom clay

Conclusions

The present work has established some characteristics of reconstituted Boom clay, isotropically consolidated, sheared in an undrained manner over a range of stresses. The value of the parameter describing the compression behaviour, $\lambda = 0.129$, is similar to that found for soil of the same origin (for example, Marino clay). It indicates a moderate compressibility corresponding to the plasticity of the soil. The gradient of the swelling line $\kappa = 0.039$ is also similar to that found for the Marino clay ($\kappa = 0.042$). The ratio $\lambda/\kappa = 3.3$ is in the range of expected values for clay soils.

The reconstituted material is seen to exhibit brittle behaviour which is not typical of normally consolidated reconstituted samples sheared from isotropic conditions. At moderate stress levels, the state representing the end of tests showed the value of the shear

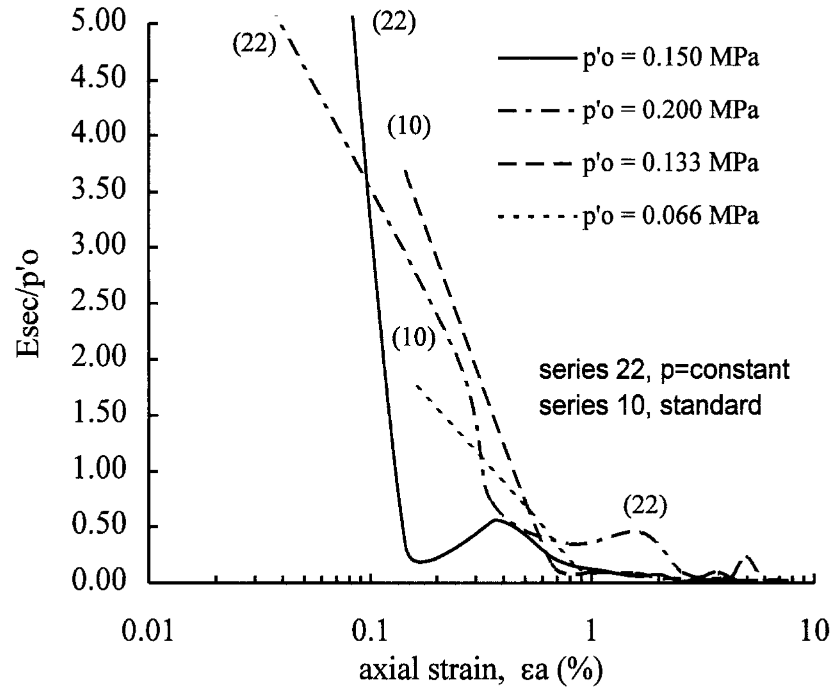

Fig. 9. Undrained secant stiffness normalized by p'_0 as a function of axial strain for reconstituted isotropically consolidated Boom clay

angle ϕ'^* to be 18.5° with an apparent cohesion intercept $c'^* = 0.01$ MPa. The undrained secant stiffness was found to vary with strain levels and also to be dependent on the consolidation pressure.

The present results throw some light on the mechanical properties of reconstituted Boom clay. However, more testing is needed on this material and on intact samples to be able to provide a more complete framework for analysis for all the tests and a theoretical basis for numerical modelling.

Acknowledgements

Financial support for this study, which was part of a larger research programme, was provided by the NFWO. The authors wish to express their gratitude for this support. The support provided to the first author by the NFWO during his stay in 1995 at the Soil Mechanics Laboratory of Ghent University is gratefully acknowledged. The authors wish to thank the anonymous referees for their very useful comments and suggestions.

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