Geotechnical and Geological Engineering 1991, **9**, 1–26

Geotechnical properties of the Corinth Canal marls

A.G. ANAGNOSTOPOULOS, N. KALTEZIOTIS*, G.K. TSIAMBAOS* and M. KAVVADAS

Geotechnical Department, National Technical University of Athens, Greece

**Geotechnical Engineering Section, Central Laboratory of Public Works, Athens, Greece*

Received 21 September 1990

Summary

The Corinth Canal crosses the isthmus of Corinth and is of great importance for Mediterranean navigation as well as the railroad and highway connections between southern and central Greece. In the century-long history of the Canal, the slopes have shown only minor stability problems despite their significant length, very steep inclination and, more importantly, the strong earthquakes that have frequently shaken the Corinth area. This type of unsual behaviour has motivated research into the mechanical behaviour of the bluish grey marl which is the main geological formation in the Canal area.

Geotechnical investigation of the Corinth Canal marl was performed with an extensive laboratory testing programme on high quality undisturbed samples of the intact marl. The material was shown to exhibit brittle behaviour, high stiffness and significant apparent cohesion at low and moderate stress levels. These characteristics indicate that the material possesses significant structural bonding which is believed to be due to cementation between individual particles, caused by the deposition of carbonates at the time of material genesis. With stressing, the material yields due to a gradual bond degradation. The locus of the initial yield points (yield surface) seems to be an ellipse oriented along the isotropic axis.

The testing programme was supplemented with a series of tests on the de-structured marl obtained by thorough remoulding. These tests showed a significant difference in the pre-yield stiffness and the peak strength at low stress levels, but comparable post-rupture shear strengths.

Introduction

Marly formations cover a large part of the Greek territory and present great interest from the geotechnical point of view. They are mainly neogene deposits of lacustrine, fluviolacustrine or marine origin containing 35 to 75% calcium carbonate and a complementary content of clay minerals (Barth *et al.*, 1939; Pettijohn, 1975). Their physical and mechanical characteristics may be quite variable and are controlled by the following factors (Datta *et al.*, 1982; Hawkins *et al.*, 1988):

(1) The mineralogy, texture, content and type of the clay minerals and especially the proportion and distribution of calcium carbonate.

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(2) The clay-sized content as determined by the hydrometer test. This content is often higher than the true clay content determined by the mineralogical analysis due to the presence of micritic material of calcite and quartz in the fine fraction of the marl.
(3) The degree of compaction and cementation caused by the diagenesis of the material after deposition.
(4) The fissuration, weathering and decalcification with time due to the effect of environmental factors.

Marls have developed a structure (Mitchell, 1976) mainly in the form of 'bonding' between individual particles due to cementation through the deposition of carbonates at time of material genesis. This structure is manifested by their high shear strength, in the form of apparent cohesion, and high stiffness at low and moderate stress levels. As a result, their geotechnical properties are not controlled by the initial void ratio and the subsequent stress history, as with typical sedimentary clays, rendering conventional geotechnical investigation and interpretation techniques insufficient (Vaughan *et al.*, 1988).

An extensive deposit of marl occurs in the Corinth area and more specifically in the area where the Corinth Canal has been excavated. This marl exhibits low compressibility and significant shear strength (apparent cohesion) at low and moderate stress levels, which is made evident by the high and steep slopes of the Canal.

The ongoing design of new railroad and highway bridges as well as the potential widening of the Canal for navigation purposes have led to renewed interest and further research of the physical and mechanical characteristics of the Corinth Canal marls.

This article investigates the geotechnical characteristics of the bluish grey marl which constitutes the main geological formation of the Corinth Canal slopes.

History of the Corinth Canal

The Corinth Canal (Fig. 1) crosses the Isthmus of Corinth and is of great importance for Mediterranean navigation as well as the railroad and highway connections between southern and central Greece. It is 6.3 km long with a maximum height of 75 metres above sea level and 10 metres below sea level. The slopes have an average inclination of 4.5 to 1 (about 75 degrees) and show only minor falls, wedge-type dislocations and deformations, which developed mostly after severe earthquakes.

The idea of crossing the Isthmus of Corinth dates back to the 7th century B.C., at the time of Periander of Corinth, one of the seven Sages of ancient Greece. He built a four metre wide stone-paved roadway, called Diolkos, which was used to drag small ships across the Isthmus. In Roman times, Nero started the excavation of the Canal in A.D. 66 using six thousand prisoners; a few months later the project was abandoned mainly due to political instability in the Empire.

After the successful completion of the Suez Canal in 1869, construction of the Corinth Canal was again considered feasible. The excavation works started in 1882 by the French company 'Societé Internationale du Canal maritime de Corinthe' under the direction of General E. Turr an associate of Ferdinand de Lesseps, the man who built the Suez Canal. The original plans proposed a slope inclination of 10:1. Five years after the excavation had started and several accidents had occurred due to slope instabilities, the slope inclination was revised to a more modest 5:1 (E. Fuchs, 1887). After long delays and large cost overruns, the

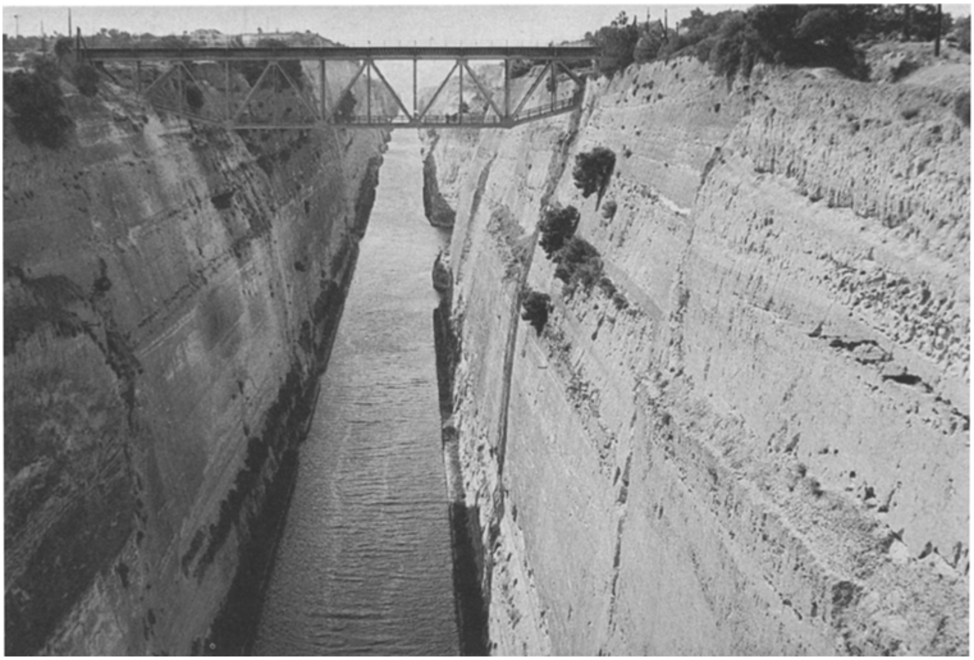

Fig. 1. A general view of the Corinth Canal

Greek company 'Societé du Canal de Corinth' continued the excavation works and the project was completed in 1893.

Between 1893 and the Second World War, fourteen slides occurred which involved about 250 thousand cubic metres of earth material and kept the Canal closed to navigation for a total of about four years. During the Second World War the railway bridge was blown up causing serious stability problems to the neighbouring slopes. After that time only minor slides have occurred, mainly caused by seismic events.

Geology and seismicity of the Canal area

The greater Canal area belongs to the Sub-Pelagonian geotectonic zone and consists of the following formations:

(1) Limestones, sandstones and basic igneous rocks of the Palaeozoic.
(2) Mesozoic limestones, radiolarites, shales and ophiolites.
(3) Plio-pleistocene lacustrine and marine alternations of marls, marly limestones, calcareous sandstones and conglomerates.
(4) Volcanic rocks (dacites).
(5) Pleistocene fluviatile and marine sandstones, conglomerates and marly limestones (calcarenites).
(6) Recent superficial alluvial deposits (mixtures of clays, sands and gravels) of fluvioterrestial origin.

The central part of the Canal, where the highest slopes have been excavated and the new facilities will be constructed, consists mainly of Plio-Pleistocene marls (about 85% of the total mass) with thin interlayers of marly sands and marly limestones. The marls in the upper portion of the slopes are quite homogeneous, whitish yellow to light brown, while those in the middle and lower portion are homogeneous and rarely laminated, yellow grey to bluish grey.

The course of development of these sediments begins with fresh-water deposits, continues with a series of changing salinity with fresh-water interim periods and ends with marine ingressions (Freyberg, 1973). Fresh-water intrusions of sands are associated with climatic variations and periods of heavy rain. The quiet periods of marl sedimentation were marked by brackish (slightly salty) water in less rainy periods. After deposition and in the end of their volume reduction, the marls were subjected to a cementation process due to the presence of a high proportion of fine particles of calcium carbonate. The cementation process has preserved the depositional void ratio (about 0.60) which corresponds to an overburden pressure of about 700 kPa (i.e., to the surcharge of about 70 metres of submerged marl) by locking the individual grains in place. As a result, the excess overburden due to any subsequent recent sediments as well as the material being raised above water, did not cause further consolidation and reduction of the depositional void ratio.

Recent research on the stratigraphy of the Corinth Canal (Collier, 1990) has revealed that, within the central horst block of the Canal, there exists an upwards transition from fresh or brackish water facies (grey to bluish grey marls) to marine sediments (light yellow marls and a series of Pleisto-Holocene beach-to-shoreface sub-sequences) as is shown in a longitudinal section of the Canal (Fig. 2). The dated sub-sequence sediments correspond to the late Pleistocene eustatic highs and the observed facies pattern is correlated to the major late Pleisto-Holocene periodic eustatic transgressions.

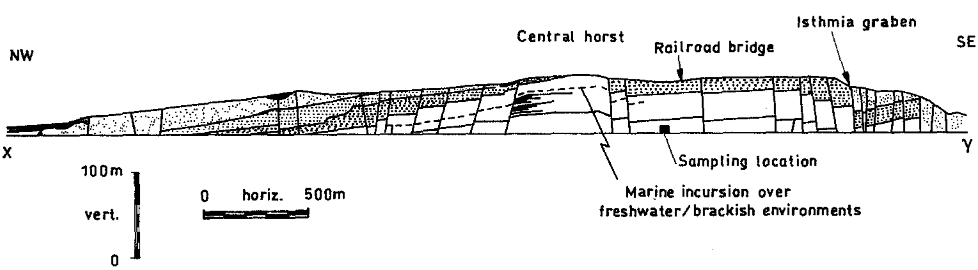

Fig. 2. Geological structure of the NE slope of the Canal

Structurally, the Corinthian Gulf area (formed in the upper Miocene) has suffered three tectonic phases by trench sinking:

(1) An extensional phase during the Plio-Pleistocene resulting in the formation of subvertical faults striking E–W to ENE–WSW.
(2) A compressional phase in lower Pleistocene which resulted in the formation of inverse faults with the same strike as above.
(3) An extensional phase from middle Pleistocene up to date with the reactivation of old normal faults or the formation of new ones with the same strike. This late tectonic evolution is closely related to the general instability of the area and to the strong and frequent earthquakes.

As a result of these tectonic phases, the Isthmus of Corinth was formed by antitilted fault blocks (Freyberg, 1973). The northern system of the block-faulted area has its largest height to the east and dips under sea level to the west, whereas the southern system acts reversely.

The blocks limited between adjacent main faults are crossed by a major system of subvertical joints running parallel to the faults at an angle of 30 to 40 degrees with respect to the Canal axis. According to the results of a recent study (Christoulas *et al.*, 1984), these joints are smooth, undulated to planar with a maximum aperture up to 10 cm, are filled with sandy clay and have an average spacing of about four metres.

Philippson (1890) in a study of Isthmus reports 23 main faults transversing the Canal. Freyberg (1973) in a more detailed mapping of the Canal slopes, presents 45 faults which create a series of tectonic grabens and horsts. Andrikopoulou *et al.*, (1988) in a geotechnical zoning of the Canal recorded 52 main faults. All major faults and their relative movement can be easily observed on both slopes of the Canal (Fig. 3). Some of the recent faults are thought to be active since minor surface ruptures have been observed after strong earthquakes in the last 60 years (Mariolakos and Stiros, 1986). More specifically, several of these joints were extended during the 1981 earthquake and caused some dislocations and limited instability in the upper portion of the slopes in the vicinity of the existing highway bridge abutments.

According to Collier (1990) the Isthmus of Corinth is currently undergoing an uplift phase, since marine deposits are found more than 100 metres above the maximum Quaternary sea level. Uplift of the Isthmus is one of the two tectonic displacement vectors affecting the area. The other is the extensional fault-induced subsidence which may be accompanied by block rotations. Furthermore, episodes of syn-sedimentary normal faulting within the late Quaternary history of the Isthmus have been detected, including the development of the wide Isthmia graben.

Studying shallow earthquakes in the Corinth area, Ritsema (1974) found that the tensional axis T is almost horizontal with a N–S direction, while the axis of compression P is almost vertical. This is in good agreement with neotectonic and seismotectonic observations.

The recent (Pleistocene-Holocene) tectonic evolution of the Corinth Gulf area is directly related to the strong and frequent earthquakes that have shaken the Canal. Recent seismicity studies (Drakopoulos *et al.*, 1978; Papazachos *et al.*, 1981) have concluded that the area is seismologically very active; during the last 130 years it has suffered three destructive earthquakes. The latest such events occurred on the 24th and 25th of February 1981 (magnitudes 6.7 and 6.4 R) with an epicentre located in the Gulf of Corinth, about 20 km from the Canal. The shocks caused localized slope instabilities, some damage to the railroad and highway bridge abutments and the Canal remained closed for three days.

The seismic risk for shallow shocks, expressed as the return period for a surface of one square degree is shown in Table 1 (Galanopoulos, 1968).

Mineralogy and physical characteristics

Mineralogy

The mineralogical composition of the bluish grey Corinth Canal marl was determined by means of chemical analysis, X-ray diffraction and scanning electron microscopy (S.E.M.) in conjunction with the energy dispersive analysis of X-rays (EDAX). Table 2 presents the results of the chemical analysis and shows very large proportions of CaO and SiO_2.

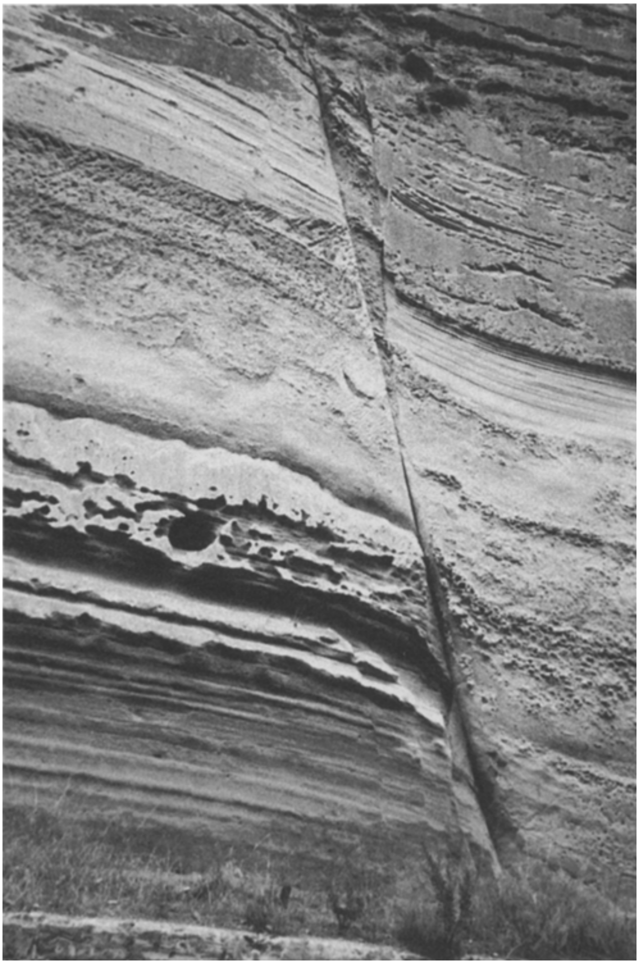

Fig. 3. Close view of a normal fault

Table 1. Return periods of shallow earthquakes in the Corinth area (after Galanopoulos, 1968)

Magnitude (R)	Return period (years)
Over 7.0	135–170
Over 6.5	55–70
Over 6.0	23–28
Over 5.5	9–12

Table 2. Chemical analysis of the Corinth Canal marl

Oxide	Content (%)
SiO_2	17.5
Al_2O_3	1.5
Fe_2O_3	1.1
CaO	42.5
MgO	1.5
K_2O	0.3
Na_2O	0.6
Ignition loss	34.5

The large proportion of calcite restricted the use of the X-ray diffraction method and required the application of the method on the residue after the dissolution of the calcite with hydrochloric acid. Table 3 presents the results of the X-ray diffraction technique.

The clay minerals consist mainly of illite (1.5 to 3.0%), chlorite (1.0 to 7.0%) and mixed constituents of illite-montmorillonite (<1%) giving a total of 2.5 to 8%. The non-clay minerals are mainly calcite (73 to 77%), quartz, feldspars and pyrite. According to Barth *et al.*, (1939) the bluish grey marl is classified on the basis of the calcium carbonate content as a limey marl.

The microstructure of the material was studied under the S.E.M. and using the EDAX technique. The grain size is very small with cryptocrystalline material of calcite as well as mixed material of clay particles and quartz (Fig. 4). The platey clay minerals with a mica structure are very altered, whereas some very well developed small automorphic crystals of quartz and calcite and well preserved diatom shells are present (Fig. 5).

The contacts of the various minerals allow the development of a high percentage of pore space. The micritic material consisting mainly of calcite seems to act as cementation agent although it is very difficult to determine with certainty by mere inspection of the microstructure whether a calcareous sediment such as this marl is cemented or not (Lee *et al.*, 1985).

Physical characteristics

The grain size distribution of the marl was determined on oven-dried samples using sieve and hydrometer analysis. Envelopes of particle size curves are shown in Fig. 6. The sand content (>60 μm) is less than 12%, the silt-sized fraction ranges between 75 and 87% and the clay-sized fraction (<2 μm) between 13 and 24% (Anagnostopoulos *et al.*, 1989). The clay-sized fraction as determined by these sedimentation tests is much higher than the true clay content determined by the mineralogical analysis and can be attributed to the presence of micritic material (<4 μm in size) consisting mainly of calcite as it was detected in microphotos of the S.E.M. This feature has also been noted in other limey marls with low values of clay-sized fraction (Tsiambaos, 1988).

The Atterberg limits of the material were determined on oven-dried samples. Figure 7 shows the data points on the Casagrande Plasticity Chart. According to the USCS this marl can be classified as a low plasticity clay (CL) or silt (ML or CL–ML).

Table 3. Mineralogical composition of the Corinth Canal marl

Mineral	Content (%)
Calcite	73–77
Quartz	13–17
Illite	1.5–3
Chlorite	1–7
Illite-Montmorillonite	<1
Feldspars	2–4
Pyrite	<1

Fig. 4. S.E.M. microphoto showing platey clay minerals with a mica structure

The natural water content of the material tested was 20–21%, i.e., slightly above the plastic limit. The specific gravity is 2.72 Mg m^{-3}, the in-situ void ratio is 0.55–0.60 and the corresponding degree of saturation is 90–95%.

Activity, as defined by Skempton (1953), is the ratio of the plasticity index to the percentage of clay-sized fraction. Figure 8 (Activity chart) shows that the bluish grey Corinth Canal marl is classified as inactive (activity of 0.30 to 0.65) if the clay-sized fraction is considered, whereas, it has normal activity (0.75 to 1.25) when the true clay mineral content is used.

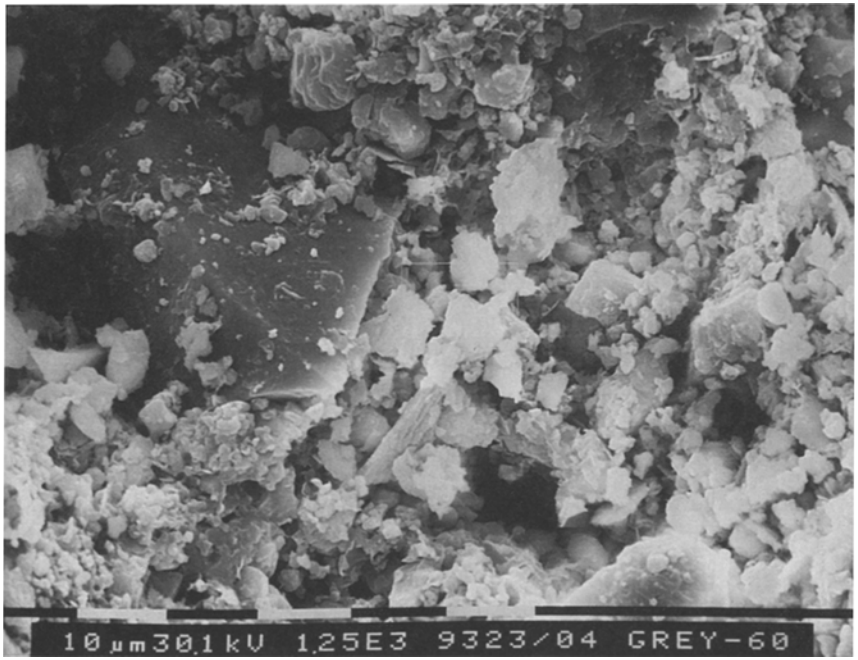

Fig. 5. S.E.M. microphoto showing an automorphic crystal of quartz

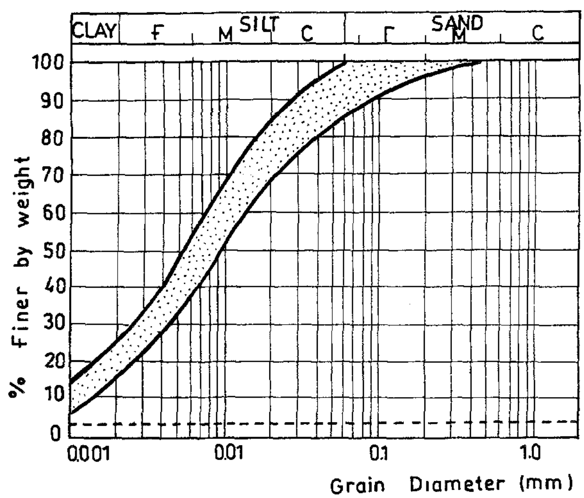

Fig. 6. Grain size distribution of the Corinth Canal marl

Investigation of the geotechnical properties

The effect of bonding in marls

The prediction of deformation and the safety margin against a catastrophic failure is the primary objective of geotechnical engineering. Until recently, geotechnical design was based

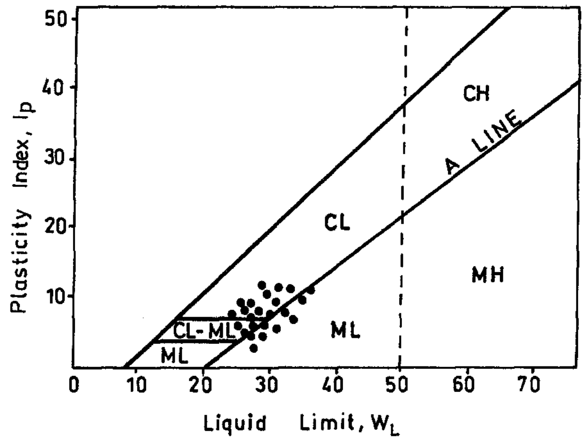

Fig. 7. Casagrande Plasticity Chart of the Corinth Canal marl

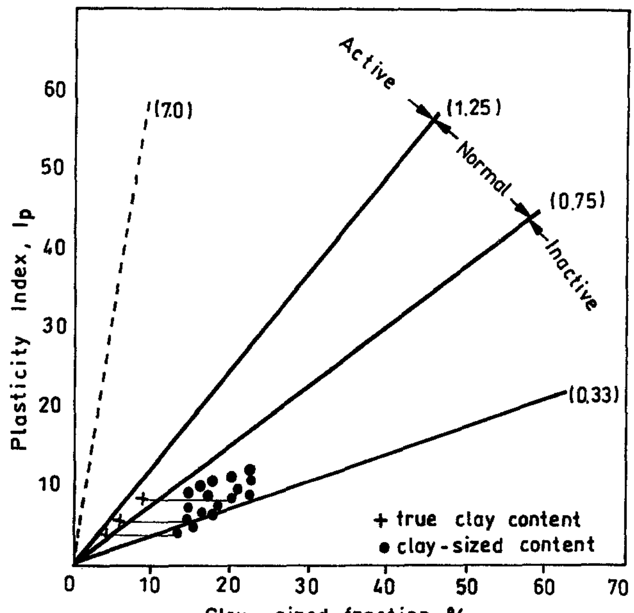

Fig. 8. Clay-sized fraction versus Plasticity Index of the Corinth Canal marl

entirely on past experience and engineering judgement rendering geotechnical engineering an art rather than a science. However, modern design procedures, which rely more heavily on analytical techniques, require the detailed description of the deformation and failure characteristics of the soils involved. Thus, extensive investigation of the geotechnical properties of soil and rock formations is generally justified.

Specifically in the case of marls, conventional geotechnical investigation and interpretation techniques are not sufficient, since material properties are directly linked to geological questions, such as the way in which the marls were originally formed and the geological processes to which they have been subsequently subjected. Marls have developed a structure which involves significant linkage or 'bonding' between individual particles. As a result, the geotechnical properties of the marls are not controlled only by the initial void ratio and the stress history as is the case with typical sedimentary clays.

The presence of bonding in many particulate materials (such as sand-stones and mud-stones) has been extensively studied in rock mechanics. In these materials, bond strength dominates material behaviour at least in the stress range of practical interest. Rather more rare are studies of particulate materials in which the contribution of bonding is comparable to the other factors affecting the material behaviour. Marls, in general, belong to this second category. Bonding in marls has a significant contribution to their stiffness and strength and, at the same time, has gradually faded the material's memory of the initial void ratio and its previous stress history. Finally, it should be mentioned that the effect of bonding on stiffness and strength has recently become apparent in all natural soils of geological age (e.g. Leroueil *et al.*, 1979; Vaughan *et al.*, 1988) with only differences in the relative importance of bonding when compared to the other factors generally recognized by classical soil mechanics (i.e., the void ratio and the stress history).

In the case of the Corinth Canal marls, the presence of a structural component of strength due to bonding is evident by the high and steep slopes of the Canal which have stood up for about one hundred years with minimal stability problems despite the strong earthquakes that have repeatedly shaken the area.

An analysis of the long term global stability of the Canal slopes using homogeneous soil properties in conjunction with the Simplified Bishop method of slices for toe circles (Kavvadas, 1990), shows that for reasonable values of the friction angle (25 to 30 degrees) and static ($\varepsilon = 0$) or seismic loading (using a pseudo-static coefficient, ε, up to 0.20), the required cohesion for stability is at least 250 kPa (Fig. 9). Such high value of the cohesion intercept can only be attributed to significant structural bonding since it is incompatible with the in-situ void ratio of the deposit (0.60) and the corresponding overburden pressure. In fact, as was mentioned in the previous section, the cementation process has preserved the depositional void ratio and any subsequent recent sediments, as well as the surcharging due to the material being raised above water, did not cause further consolidation and reduction of the void ratio (the actual overburden at present is about 1500 kPa and consolidation to this stress level would give a void ratio significantly less that 0.60). Actually, one-dimensional consolidation tests on the fully de-structured (slurry) material presented in Fig. 17 show that the in-situ void ratio 0.60 corresponds to an overburden pressure of about 700 kPa, i.e., to about 70 metres of submerged (at the time of sedimentation) overburden which was approximately the actual overburden at the toe of the Corinth Canal slope. This value of the void ratio and the associated vertical consolidation stress of about 700 kPa is incompatible with the present overburden pressure and the cohesion intercept required for the stability of the slope according to the analyses mentioned above. Furthermore, there seems to be no evident increase in the shear strength of the deposit with depth, as would be anticipated in the case of a typical normally consolidated clay. All the previous arguments lead to the conclusion that the mechanical behaviour of the Corinth Canal marl can be explained if a structural component of strength due to bonding between individual particles is assumed.

The prevailing opinion among geologists today (e.g. Andrikopoulou *et al.*, 1988) is that bonding of the Corinth Canal marl is due to cementation caused by the deposition of carbonates at the time of material genesis. However, the bond strength thus acquired is not sufficiently high to give the marl rock-like properties.

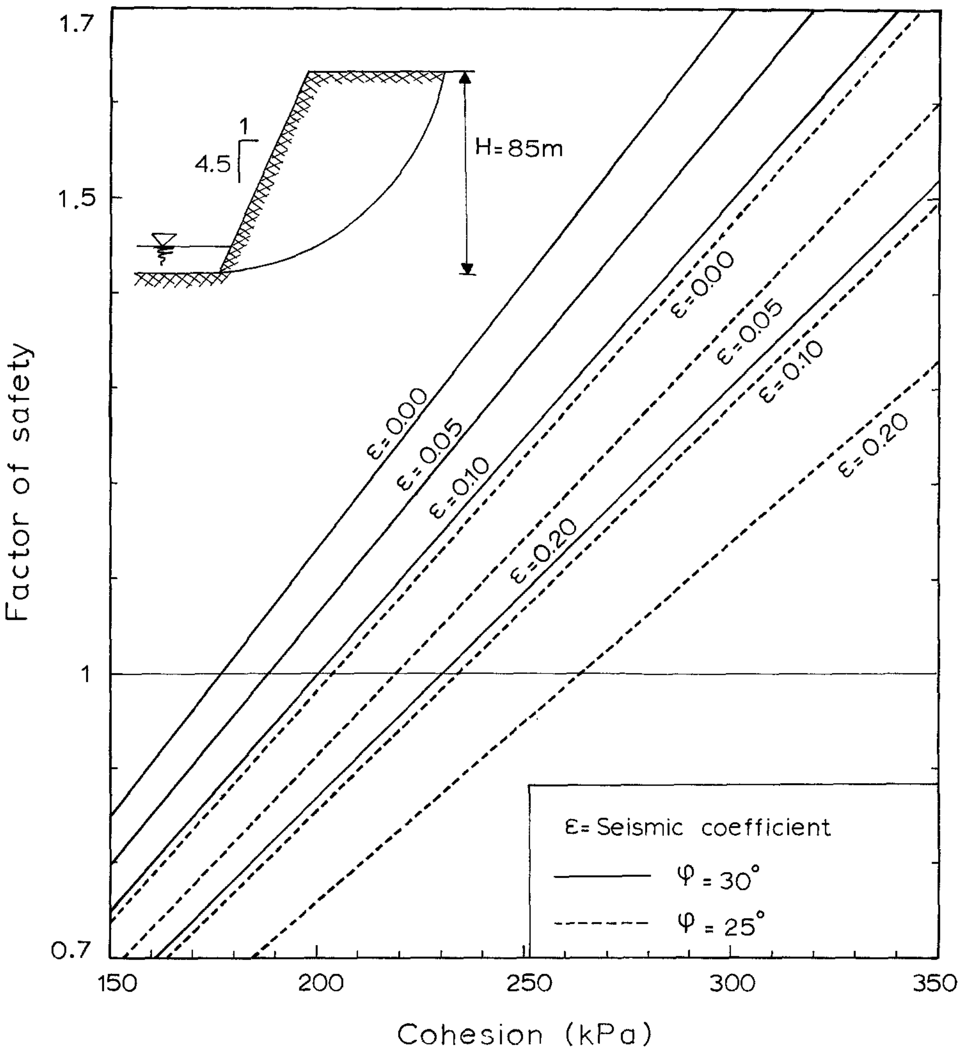

Fig. 9. Slope stability analysis of the Corinth Canal slope. Simplified Bishop method of slices. Toe circles

Bonding is manifested by a high stiffness and significant shear strength at low and moderate stress levels, which is incompatible with the in-situ void ratio. Furthermore, since the bond strength is significantly higher than the anticipated cohesion due to the effect of overburden, the properties of the material become practically independent of the previous stress history.

Bonding may be broken by straining usually associated with stress changes, and, once broken, it is irrecoverable, except by the long-term geological processes which created it. On the contrary, the cohesive component of strength in typical sedimentary clays is due to attractive forces between closely packed particles and as such, it can be recovered if the packing and density is recovered.

In general, the behaviour of the bonded Corinth Canal marl is characterized by an initial stiff response, followed by yielding and significantly reduced stiffness. The initial yield state can be seen in the stress-strain behaviour of the drained and undrained tests as well as in the change of the volumetric and pore pressure response of the drained and undrained tests, respectively. The envelope of the initial yield states in the triaxial plane can be postulated as the yield surface of the material, according to the standard Plasticity Theory convention. The determination and mapping of the initial yield surface is very important in the study of the mechanical behaviour of the Corinth Canal marl and was one of the primary objectives of the laboratory testing programme on the intact material.

Sampling and testing of the 'intact' marl

The influence of bonding on the engineering behaviour of the Corinth Canal marl was investigated by means of an extensive laboratory testing programme on 'intact' samples of the lower bluish grey marl. Early in the study it was realized that in order to investigate the magnitude of the bonding and its effects on the stiffness and shear strength, sample disturbance should be minimized. This is a desirable objective in the geotechnical investigation of all soils but it is especially important in the study of bonded formations. The cutting of a sample (or the cutting of a borehole or a trial pit) involves mechanical disturbance of the cut faces. In brittle and friable bonded soils this effect may result in a pronounced reduction of the low-strain (pre-yield) stiffness and a less pronounced drop of the in-situ shear strength. While improved methods of rotary sampling are in use today, it was thought that in the case of bonded materials satisfactory undisturbed samples can only be obtained by hand trimming block samples from test pits. Furthermore, the in-situ stress relief which is inevitable in any sampling process will cause sample swelling in partly saturated soils. Typically, the strains involved are small, but the soil structure is stiff and even small strains may reduce structural stiffness, yield stress and strength (Marsland, 1971).

In order to minimize disturbance in the present investigation, large block samples (about 40 cm in size) were carefully hand trimmed from the toe of the slope 2 to 3 metres above sea level (Fig. 10). The blocks were then thoroughly sealed with paraffin wax, placed in double plastic bags and carefully transported to the laboratory.

The testing programme on the 'intact' marl consisted of:

(a) conventional triaxial tests on cylindrical specimens (isotropically consolidated and subsequently sheared under drained or undrained conditions),
(b) isotropic consolidation and rebound tests on cylindrical specimens and,
(c) One-dimensional consolidation tests in the conventional oedometer apparatus.

Cylindrical specimens 35 mm in diameter and 70 mm in length were used in the triaxial tests. Specimens were carefully trimmed to size into a thin-wall lubricated tube equipped with a stationary piston, by hand-trimming ahead of the tube as it was slowly advanced by a manual screw-type press. Specimens were then extruded from the tube (by advancing the piston) into a divided sampler having the required length (70 mm) and cut to the correct length against the edges of the sampler. The authors believe that this method of trimming minimizes disturbance to acceptable levels for this type of brittle material.

Each specimen was then placed in the triaxial cell and saturated by back pressuring in steps while maintaining an effective stress in the specimen of about 50 to 80 kPa. This value of the effective stress was selected in order to avoid sample swelling due to the removal of the

Fig. 10. Hand trimming of a block specimen

negative pore pressures in the partly saturated sample. In general, the samples did not show any appreciable swelling potential. This can be explained by the low content of clay particles, the stiff structure and the high initial degree of saturation (about 90%). During the saturation process, the degree of saturation was monitored in terms of the Skempton's B parameter. Saturation was considered sufficient when B values exceeding 0.95 were obtained. Such values of the B parameter are believed to correspond to a degree of saturation over 99% for the stiff 'intact' marl. Typical values of the back pressure in the range of 400 to 500 kPa were used.

After saturation, the samples were isotropically consolidated to the required consolidation stress in steps not exceeding 500 kPa, in 24 hour increments. Consolidation in steps was selected to avoid inhomogeneities in the sample induced by the high pore pressure gradients near the ends. A large set of consolidation stresses in the range of 100 to 6000 kPa was selected, in order to scan a wide domain of effective stress paths and thus investigate the de-structuring of the material with increasing effective stress.

Following the consolidation stage, specimens were sheared under drained and undrained conditions. The shearing rate selected (axial displacement of 0.009 mm min^{-1}, i.e. about 24 hours to failure) was sufficiently small to ensure complete pore pressure dissipation in the drained tests. The same rate was used in the undrained tests to minimize stiffness and strength differences due to variable creep rates between drained and undrained tests.

Consolidated drained and undrained effective stress paths were used to locate and map the initial yield surface. Conventional platten to platten axial strain measurements using LVDT transducers were performed. While more sophisticated strain measurements (such as using transducers attached to the sides of the sample) have a considerable advantage in the investigation of the soil response at very low strain levels, they are not essential in mapping the surface of initial bond yield (e.g. Vaughan, 1985) since the associated axial strains exceed 0.5%.

Figure 11 presents the shear stress and volumetric strain response of selected drained triaxial tests on the intact marl. The samples were initially isotropically consolidated at a wide range of confining pressures in order to study the material behaviour in a large portion of the triaxial plane in the stress space. The results show brittleness and dilation at low confining stresses and more ductile response with significant volume reduction at higher consolidation pressures. This type of behaviour is analogous to the behaviour of typical (unbonded) clays which are dilatant at states denser than critical and contractive when wetter than critical.

In the case of the samples showing dilative behaviour, the maximum dilation rate occurs at axial strain levels slightly higher than those corresponding to the peak shear stress. This type of behaviour does not occur in typical unbonded materials, where maximum dilation and peak strength states coincide, but it has been observed on a weakly bonded artificial soil (Maccarini, 1987).

The change in the response of the intact marl with increasing confining stress cannot be attributed to variations in the initial void ratio, since the decrease in the void ratio of the intact material with increasing confinement is minimal (see Fig. 17). It seems more reasonable to assume that the change in the response of the intact marl with increasing confining stress is due to gradual de-structuring in the form of bond degradation. Bond degradation also affects the stiffness of the shear stress–strain response. The experimental curves show an abrupt decrease in the stiffness at a shear stress level which is assumed to coincide with the initiation of bond degradation and defines the so-called initial yield state. At low confining stresses, yield and failure (defined as the peak shear stress state) nearly coincide, whereas at higher confining stresses yield becomes more muted and occurs well before failure. Post-yield behaviour seems to involve gradual bond degradation without sudden collapse in the marl structure, independently of the confining stress level.

Figure 12 shows the effective stress paths of the complete set of drained triaxial tests performed on the intact marl. The envelope of the initial yield states bounds the region of the pre-yield (bonded) response and defines the initial yield surface of the marl. Increasing isotropic and/or shear stress leads to states outside the initial yield surface which are associated with gradual bond degradation and reduced stiffness. The shape of the initial yield surface shows that initiation of bond degradation can be achieved more easily with the application of a shear stress rather than an isotropic stress of the same magnitude. However, even pure isotropic consolidation (at about 3250 kPa) leads to an initiation of bond degradation.

Similar type of behaviour (though not as pronounced) is exhibited by undrained triaxial tests on the intact marl. Figure 13 shows the shear stress and excess pore water pressure response of selected undrained triaxial tests, initially isotropically consolidated at a wide range of confining pressures. The peak shear stress at low confining stresses is not as pronounced as in the case of the drained tests and is associated with the development of almost nil excess pore pressures. At higher consolidation stresses the response becomes more ductile and significant positive excess pore pressures develop. This type of behaviour is analogous to the behaviour of typical (unbonded) clays. The analogy is further supported by the shape of the effective stress paths (Fig. 14). At very low confining stresses (less than about 400 kPa) total and effective stress paths approximately coincide (slope 3:1 in the p-q space)

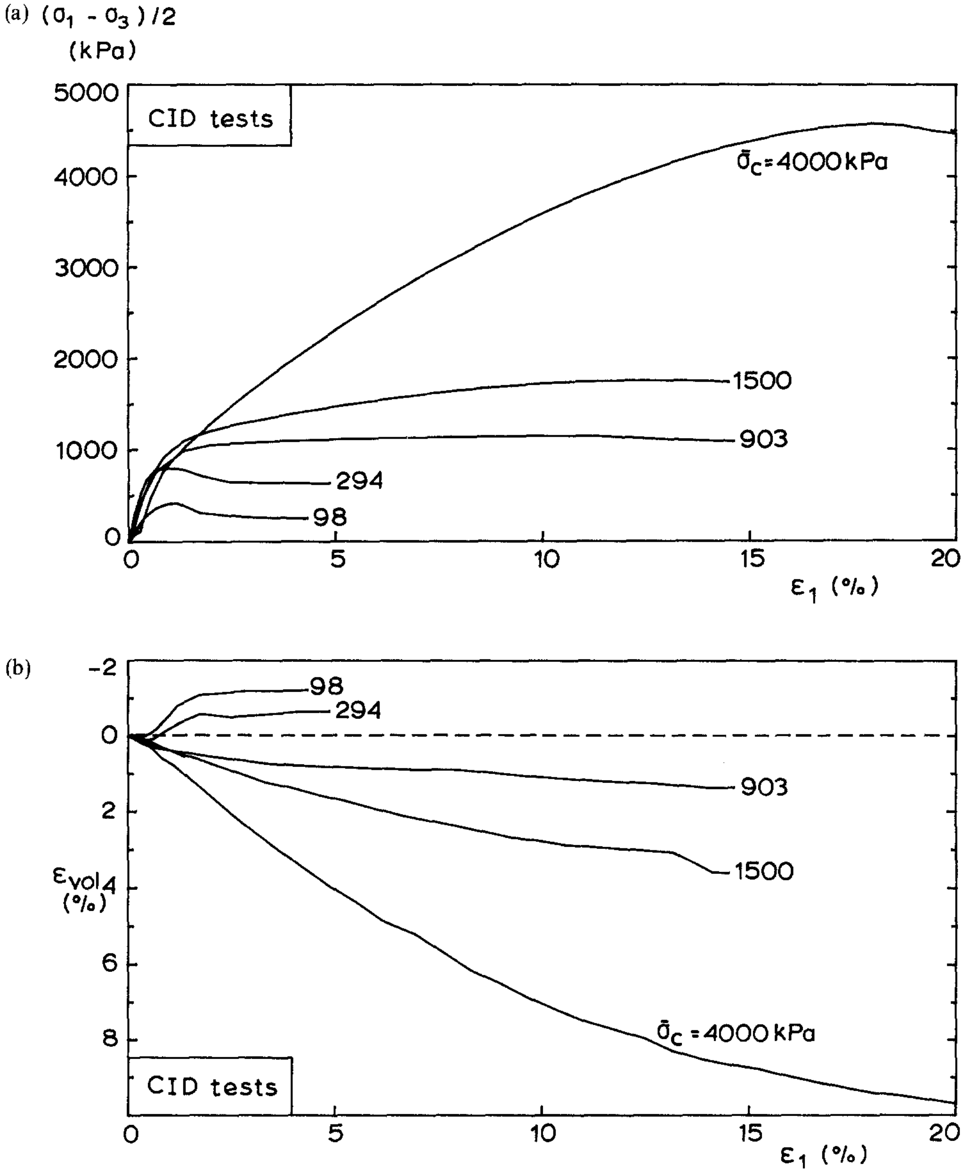

Fig. 11. Drained triaxial tests on the intact marl: (a) shear stress–axial strain curves; (b) volumetric strain–axial strain curves

indicating that almost nil excess pore pressures develop. As a result, the shear induced excess pore pressure component is negative (i.e., there exists a tendency for dilation) and approximately equal in magnitude to the (positive) isotropic component of the excess pore pressure, which is equal to the increase in the mean effective stress during shearing. At higher confining stresses (500 to 3000 kPa) the effective stress path is almost vertical nearly up to failure, indicating that negligible shear induced excess pore pressures develop. Finally, at high confining stresses (exceeding 3000 kPa) positive shear induced pore pressures develop

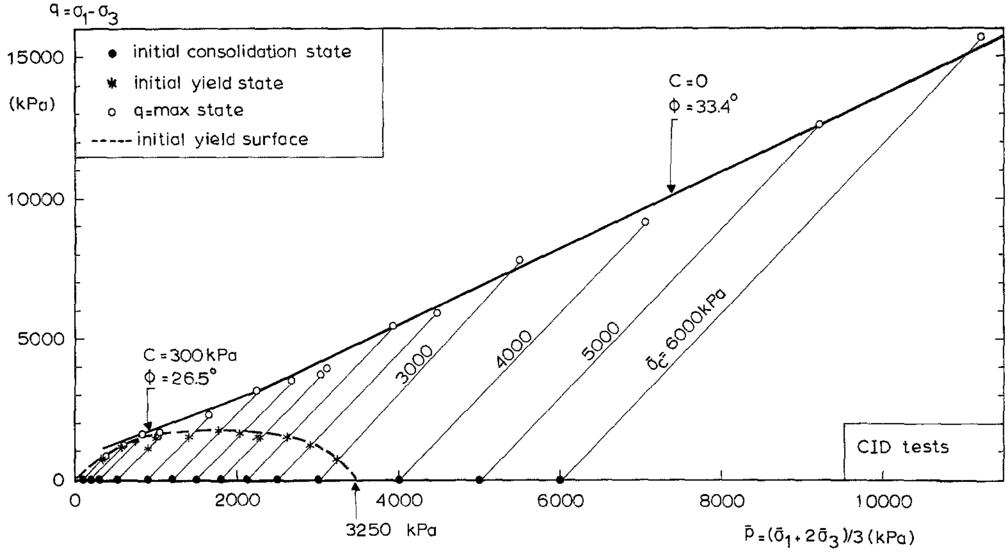

Fig. 12. Drained triaxial tests on the intact marl. Effective stress paths

indicating a tendency for a shear stress induced decrease in the volume of the specimen, i.e., significant bond degradation. The region in the stress space associated with significant bond degradation in the undrained tests, which is identical to the region where positive shear induced pore pressures develop (i.e., where the mean effective stress decreases), lies outside the initial yield surface determined from the drained triaxial tests on the intact marl and thus supports the existence of a unique yield surface for both drained and undrained tests.

The test results available at present justify the hypothesis of a yield surface with an elliptical shape, having its major axis aligned with the isotropic stress axis, which indicates that the Corinth Canal marl possesses isotropic yielding characteristics. This type of behaviour is different than the behaviour of typical (unbonded) natural clays. For such clays there is extensive evidence (e.g. Leroueil *et al.*, 1979) that the yield surface is oriented along the K_0-line, i.e., yielding reflects the preferred direction of the one-dimensionally (K_0) consolidated material. In the case of the Corinth Canal marl, the development of a structure due to calcium carbonate bonds between individual particles seems to have completely faded the material's memory of its stress history preferred directions. In fact, it is believed that the interparticle bonds have developed uniformly in all directions (i.e., isotropically) and thus bond degradation is associated with an isotropic yield surface.

Figure 15 shows the peak shear stress states of both drained and undrained triaxial tests on the intact marl. For mean effective stress levels less than about 2000 kPa, the peak shear stress states can be approximated by a straight line failure envelope with a friction angle of 26.5 degrees and an apparent cohesion intercept of 300 kPa. Such values of the effective shear strength parameters are compatible to those back-figured from the long-term stability analyses of the Corinth Canal slopes (Fig. 9) and cover the stress range of practical interest to engineers.

Failure of the intact Corinth Canal marl in the triaxial tests (both drained and undrained) develops in the same way as in typical (unbonded) clays. At low confining stress levels failure occurs with the development of a narrow shear band in the specimen, further deformations

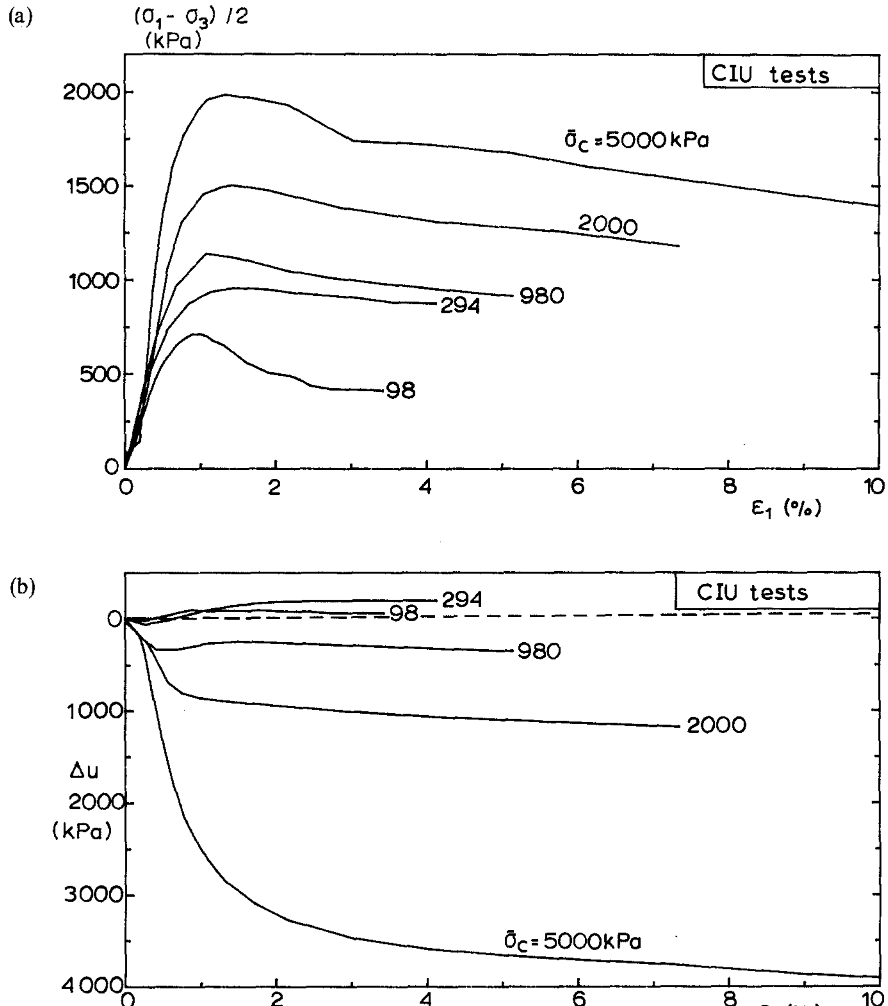

Fig. 13. Undrained triaxial tests on the intact marl: (a) shear stress–axial strain curves; (b) excess pore pressure–axial strain curves

are localized inside the shear band and strain softening occurs. Consequently, the strain and excess pore pressure measurements obtained from boundary measurements are questionable in the strain softening region and thus the critical state cannot be defined with confidence. At higher confining stresses failure develops with approximately uniform straining, the specimen bulges and strain hardening occurs until the residual strength is reached.

The transition between the stiff (bonded) and the softer (de-structured) states can also be seen in the compression curves of the intact marl during isotropic and one-dimensional consolidation. Figures 16 and 17 present such curves in the e-log p space and in the natural scale e-p space. The corresponding curves of the same material in its completely de-structured (slurry) state are also shown. The compression curves of the intact marl exhibit very small compressibility until yielding initiates at about 3250 kPa, whereas the compressibility of the intact material in the post-yield domain is comparable to that of the

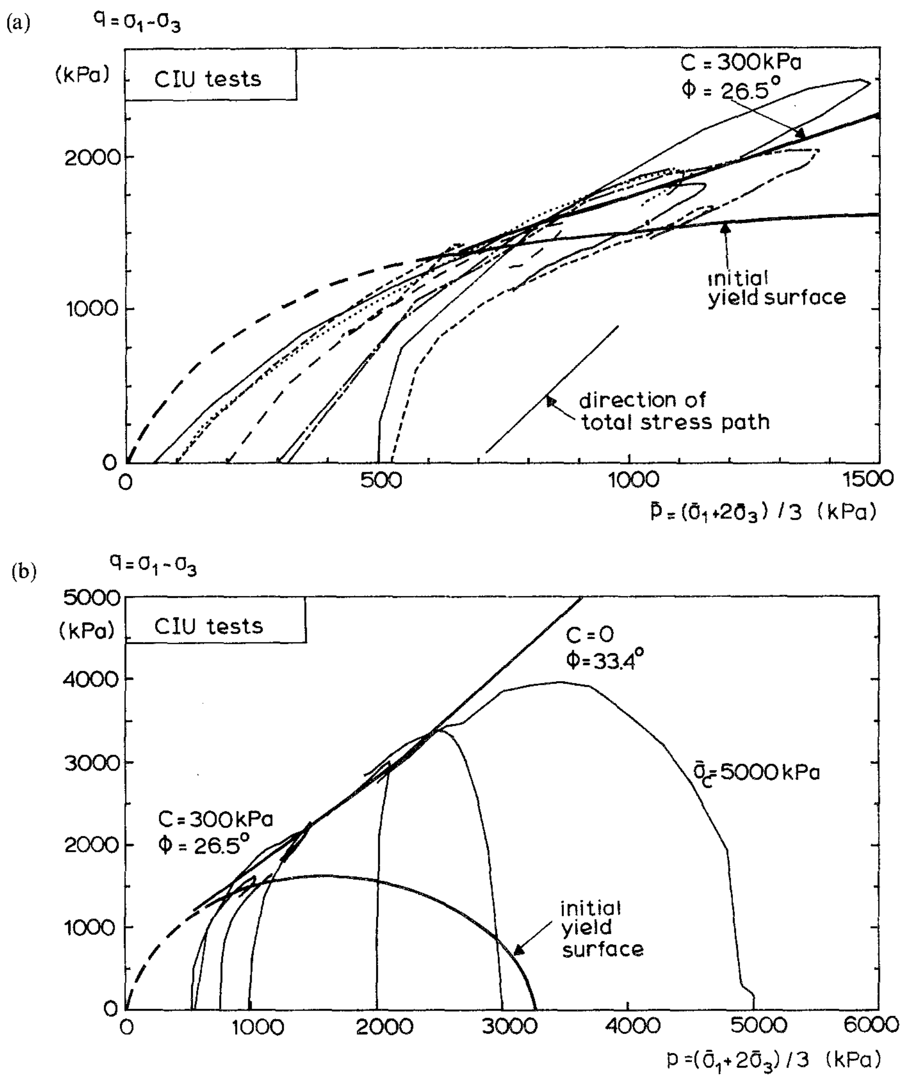

Fig. 14. Undrained triaxial tests on the intact marl. Effective stress paths

slurry. The behaviour of the destructured marl is examined in more detail in the following section.

Testing of the 'de-structured' marl

Interpretation of the behaviour of the 'intact' marl is improved if the behaviour of the same material in its 'de-structured' state is also considered. Leroueil *et al.* (1979) use the term 'de-structured' to describe a soil from which interparticle bonding has been removed. De-structuring of the Corinth Canal marl was achieved by crushing the material, adding distilled water up to a water content about 1.5 times the liquid limit and then remoulding the slurry for one hour in a mechanical mixer. Following mixing, the slurry was placed in a 30 cm

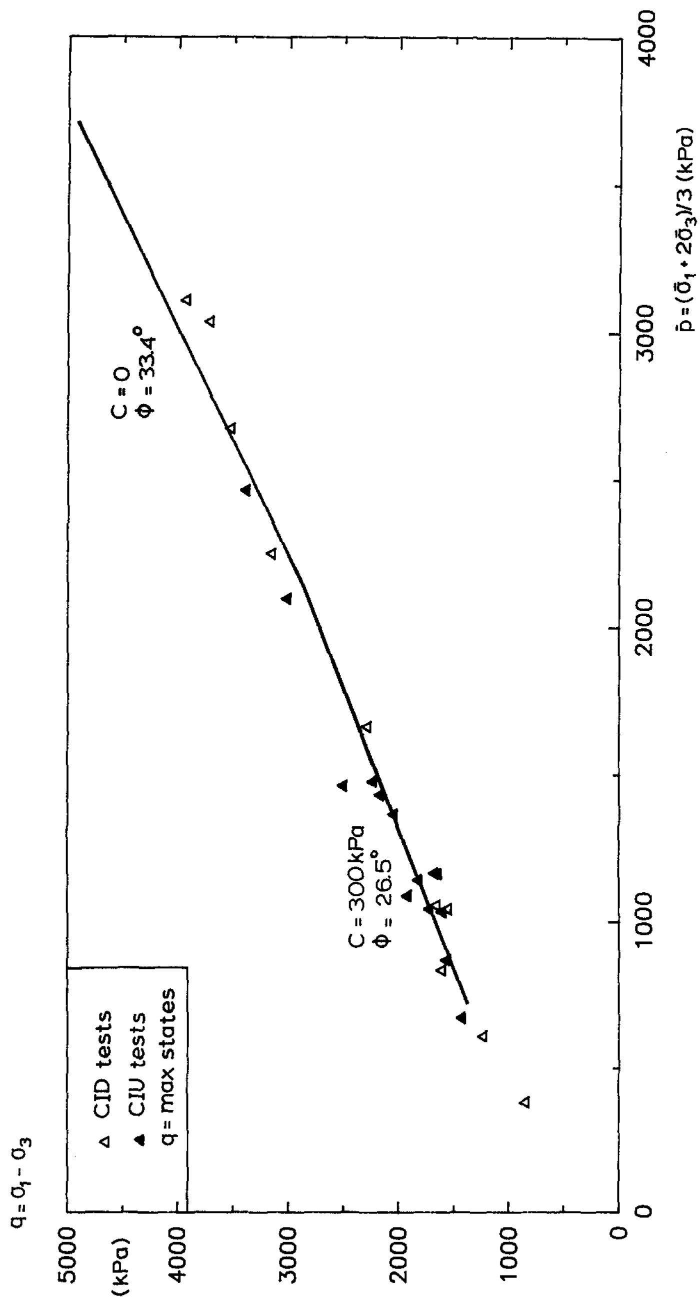

Fig. 15. Peak shear stress state points of drained and undrained tests on intact marl

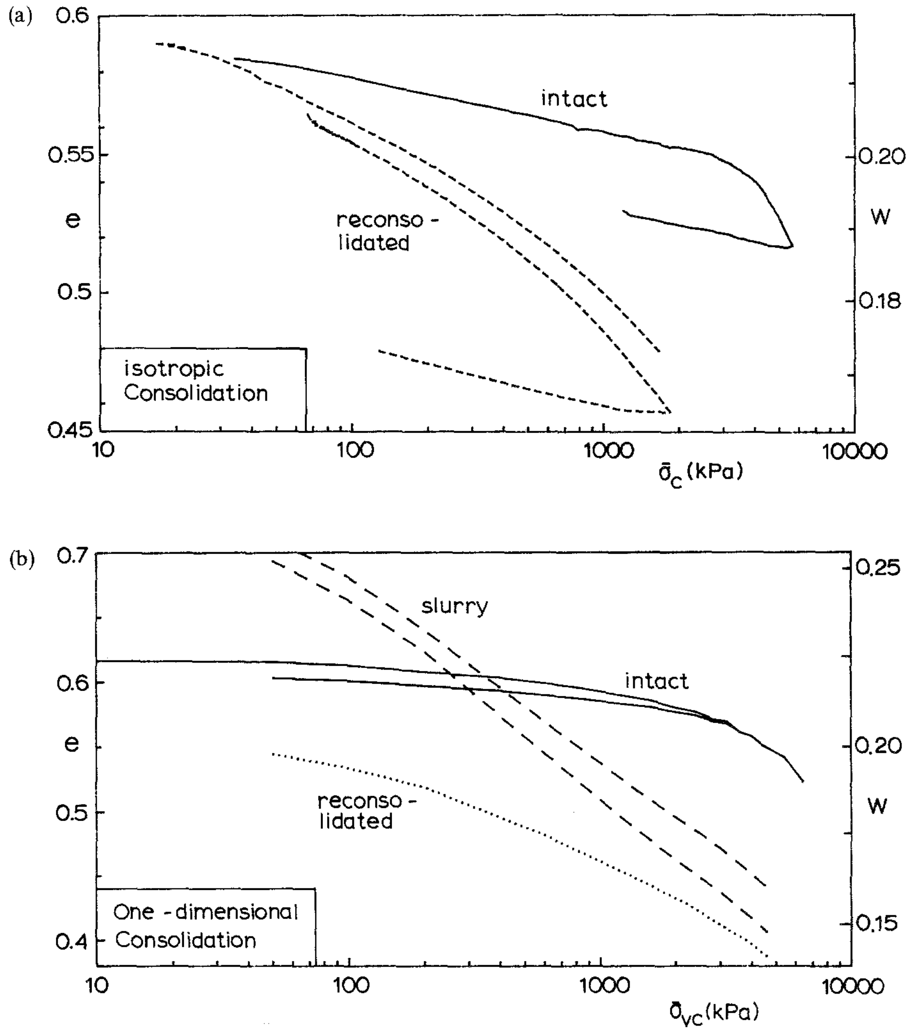

Fig. 16. Logarithmic stress scale of (a) Isotropic consolidation tests on intact and reconsolidated (de-structured) marl (b) One-dimensional consolidation (oedometer) tests on intact, reconsolidated and slurry marl

diameter consolidometer and was slowly compressed up to the in-situ water content (20%). Consolidation took place under constant rate of strain (rate 0.0015 mm min^{-1}) with measurement of the axial load. A vertical pressure of about 700 kPa was required to reconsolidate the slurry to the in-situ void ratio. The applied strain rate was slow enough to induce small excess pore pressure gradients in the specimen and thus achieve a uniform water content. Control tests showed that this method of sample reconstitution is repeatable and can provide uniform water contents in the whole sample volume (19.5 to 20.5%). The height of the specimen after consolidation was about 12 cm. After consolidation the specimen was carefully removed from the cell and hand cut to give ten cylindrical samples for triaxial testing.

The compression curve of the large diameter consolidation test was practically identical to

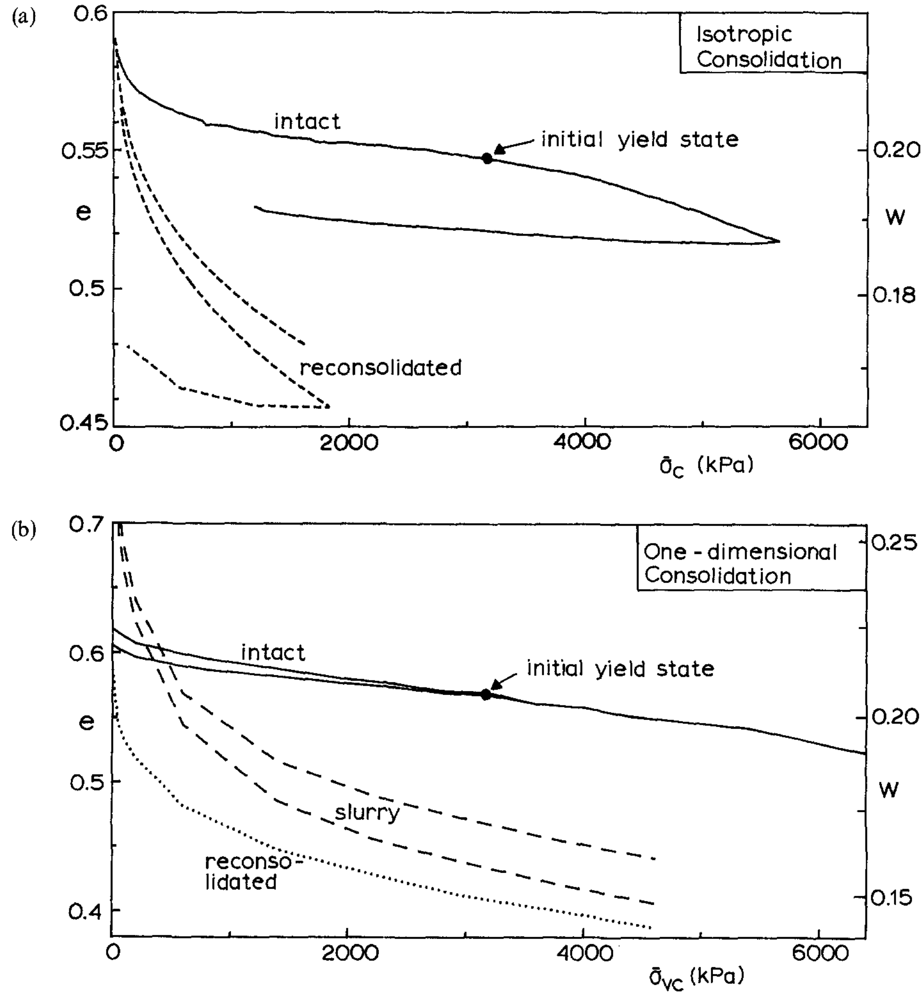

Fig. 17. Linear stress scale of (a) Isotropic consolidation tests on intact and reconsolidated (destructured) marl (b) One-dimensional consolidation (oedometer) tests on intact, reconsolidated and slurry marl

the curves obtained from the conventional oedometer tests on the slurry material (Figs 16 and 17). These figures also show the results of isotropic and one-dimensional consolidation tests on the reconstituted (de-structured and reconsolidated) samples. The behaviour of the reconsolidated samples should be analogous to that of an overconsolidated material rebounded from the virgin compression curve of the slurry, since the compression curve of the reconsolidated material converges on that of the slurry. The reconsolidated material does not exhibit sharp yielding despite its preconsolidation stress of about 700 kPa. This feature indicates that ageing under the preconsolidation pressure is necessary to obtain the typical characteristics of an 'overconsolidated' material (such as a distinct increase in compressibility at the preconsolidation pressure).

The compression curve of the isotropic compression and the oedometer test on the intact marl (Figs 16 and 17) show a clear sharp yield at a mean effective stress of about 3250 kPa.

The initial yield point is evident in both the semi-logarithmic and the linear scale plots and can be determined by using the standard Casagrande construction in the semi-logarithmic plot or, equivalently, by the inflection point of the stress–void ratio curve in linear scale. According to Vaughan (1985), the linear stress scale plots are more suitable than the conventional semi-logarithmic plots for demonstrating and quantifying yield of bonded materials. In the case of the Corinth Canal marl, the semi-logarithmic plots are equally appropriate. The compression curve of the intact marl moves into 'meta-stable' states (Vaughan *et al.*, 1988) in the stress–void ratio space, i.e., it moves in the region which is inaccessible to the fully de-structured material. After initiation of bond degradation, the intact material has approximately the same stiffness as the de-structured (reconsolidated) material. However, the two compression curves do not converge to a unique line since the accumulated pre-yield void ratio difference is preserved at least up to the stress level applied (6000 kPa). This type of behaviour is similar to that of cohesionless soils, where the location of the stress–void ratio curve in the e-p space is controlled by the initial void ratio (i.e., the density), while the stiffness is mainly controlled by the current stress level.

The difference in the compression curves of the 'intact' and 'reconsolidated' (destructured) material is believed to be due to the bonds in the intact marl. It has been postulated (Vaughan *et al.*, 1985) that part of the stress applied to the intact material is carried by the bonds. More specifically, at any void ratio the bond stress is equal to the difference in the stresses corresponding to the intact and reconsolidated curves at the specific void ratio. Since, the compression curves of the intact and reconsolidated marl do not converge to a unique line, part of the applied stress is carried by bonds even in the post-yield domain, which means that complete bond degradation cannot be achieved by the application of consolidation stress paths for the stress levels used in this investigation.

The difference in the behaviour between intact and reconsolidated marl during shearing is shown in Fig. 18, which compares the response of intact and reconsolidated samples for drained triaxial tests on pairs of samples initially isotropically consolidated at 100 and 500 kPa. The intact material shows a clearly defined peak strength at the very low consolidation stress, while the reconsolidated samples show a plastic strain hardening behaviour. Furthermore, both pairs of tests show that the intact sample has a much higher pre-yield stiffness but each pair of tests eventually converges to the same value of the post-rupture shear strength. The stress–strain curves of the reconsolidated samples do not exhibit brittle behaviour (i.e. a distinct yield state) as would be anticipated for a typical overconsolidated material (the OCR of the reconsolidated samples is $700/100 = 7$). The same feature was also present in the stress–strain curves of the oedometer tests on the reconsolidated marl (Fig. 16 and 17) and is due to the fact that significant ageing under the preconsolidation pressure is required in order to obtain the typical characteristics of an 'overconsolidated' material.

Conclusions

In the century-long history of the Corinth Canal, the slopes have shown only minor stability problems despite their significant length, very steep inclination and, more importantly, the strong earthquakes that have frequently shaken the Corinth area. This type of unusual behaviour has motivated research into the mechanical behaviour of the bluish grey marl which is the main geological formation in the Canal area.

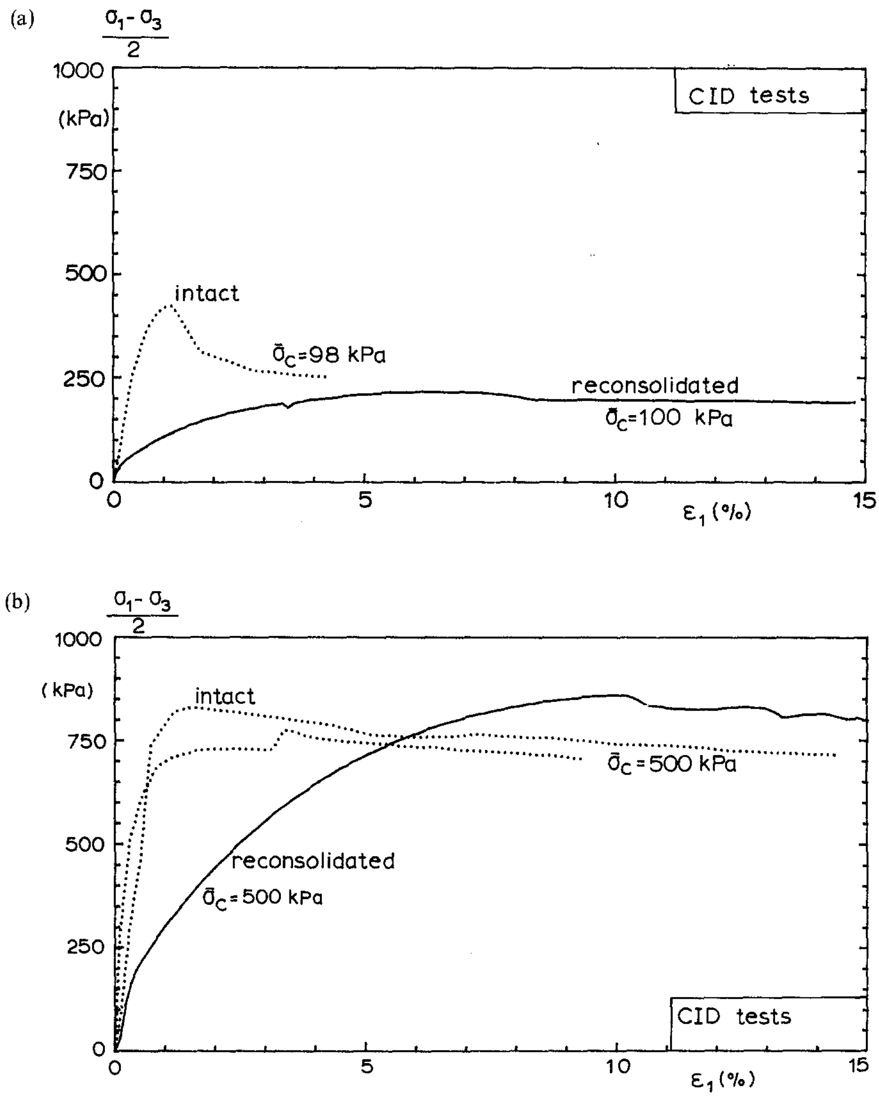

Fig. 18. Drained triaxial tests on the intact and reconsolidated marl (consolidation stress 100 and 500 kPa)

The central part of the Canal, where the highest slopes have been excavated, consists of Plio-Pleistocene lacustrine and marine marls with thin interlayers of marly sands and marly limestones. After deposition and in the end of their volume reduction, the marls were subjected to a cementation process due to the presence of a high proportion of fine particles of calcium carbonate. The cementation process has preserved the depositional void ratio (about 0.60) which corresponds to an overburden pressure of about 700 kPa (i.e., to the surcharge of about 70 metres of submerged marl) by locking the individual grains in place. As a result, the excess overburden due to any subsequent recent sediments as well as the material being raised above water, did not cause further consolidation and reduction of the depositional void ratio.

Geotechnical investigation of the mechanical properties of the marl and, more specifically, the study of the effect of bonding on the stiffness and strength, was performed with an extensive laboratory testing programme on carefully trimmed undisturbed samples of the intact material. The testing programme was supplemented with series of tests on the de-structured marl obtained by thorough remoulding. The test results show that:

(1) The material exhibits brittle behaviour, high stiffness and significant apparent cohesion (about 300 kPa) at low and moderate stress levels which verify the existence of structural bonding. This bonding is believed to be due to cementation between individual particles, caused by the deposition of carbonates at the time of material genesis. The measured value of the cohesion is compatible with the cohesion required for the stability of the Canal slopes.

(2) With stressing, the material yields due to a gradual degradation (destruction) of the bonds between the particles. The locus of the initial yield points (yield surface) seems to be an ellipse in the triaxial plane oriented along the isotropic axis.

(3) A comparison of the behaviour of the intact and reconstituted samples consolidated at the same stress shows:
   (a) a significant difference in the pre-yield stiffness,
   (b) a significant difference in the peak shear strength at low stress levels, and
   (c) similar post-rupture shear strength.

Acknowledgements

The laboratory testing programme for this work was performed at the Soil Mechanics Laboratory of the National Technical University of Athens, the Central Laboratory of Public Works, Athens, Greece and at the University of Durham, UK (in a student exchange scheme funded by the EC ERASMUS programme).

The assistance provided in the experimental work by the technicians Mr G. Pyrgiotis, Mr Ch. Tsaligopoulos and the students Ms M. Moula, Mr N. Vaptismas, Mr P. Pournaras and Mr P. Karanikolas is gratefully acknowledged.

During the study, the authors have benefited from discussions with Professor J.B. Burland and Dr L. Bressani of Imperial College, and Dr D. Toll of the University of Durham.

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