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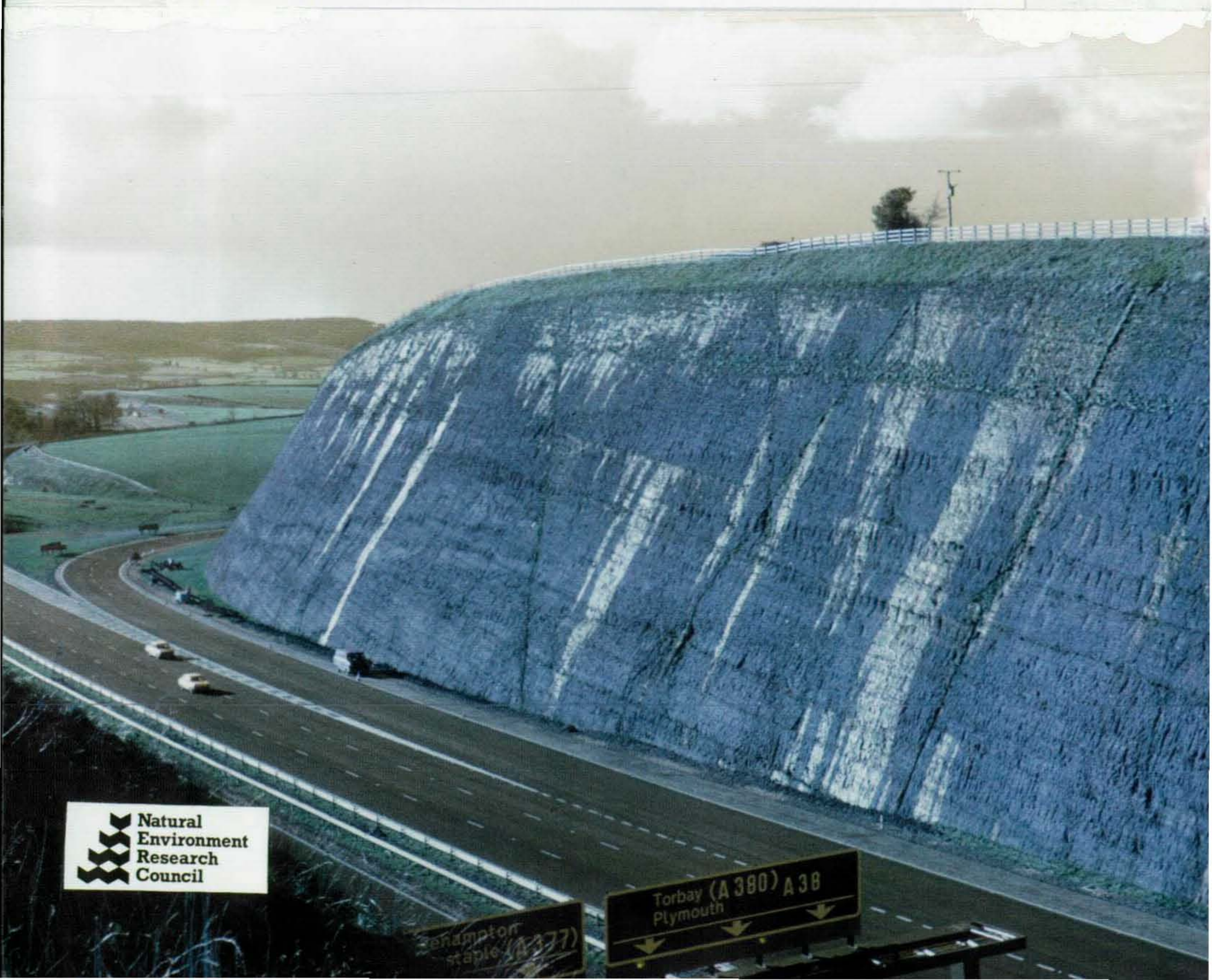
THE ENGINEERING GEOLOGY OF THE EXETER DISTRICT 1:50 000 GEOLOGY SHEET 325

A. Forster



Natural Environment Research Council

BRITISH GEOLOGICAL SURVEY

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Maps and diagrams used in this report are based on Ordnance Survey mapping.

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Cover photograph: Heavitree Breccia in cuttings [SX 932 881] for the M5 Motorway near Exminster, Devon (photograph courtesy of Dr John C. Davey).

THE ENGINEERING GEOLOGY OF THE EXETER DISTRICT

INTRODUCTION

This description of the engineering geology of the district around Exeter is based on the lithostratigraphical units (Table 1) identified during the remapping of the district (Fig.1 1:50 000 geological map sheet 325) by the field geologists of the British Geological Survey (Bristow et al 1985) and a geotechnical database compiled by the Engineering Geology Group of the BGS.

The geotechnical database was compiled from reports of site investigations for trunk roads, motorways and other construction projects carried out in or near the district which are held by the BGS in the National Geoscience Data Collection. Additional information was obtained from scientific papers published on the geology, engineering geology and the earth surface processes which have affected the area.

The geotechnical samples and their geotechnical parameters were obtained for the requirements of particular construction projects, and the sampling and geotechnical testing done on the samples were governed by the technical requirements of those projects. Therefore, the data which were abstracted from site investigation reports and entered into the geotechnical database do not necessarily represent all the lithostratigraphical units of the area. Also the values shown may not be representative of the variation in properties within a lithostratigraphical unit. However, there is sufficient information to give a guide to the likely behaviour of most of the lithostratigraphical units which have been mapped.

The geotechnical data were abstracted from good quality site investigation reports and entered onto data sheets. The data were entered into a computer file by double key entry to minimise keying-in errors, and the resulting file was transferred to a database system where it was checked for missing or erroneous values. The data were split into subsets on the basis of lithostratigraphical unit and the subsets transferred to a statistical package for analysis.

Table 1 The lithostratigraphic units of the Exeter district

QUATERNARY	
	Head
	River Terrace Deposits
	Alluvium
	Marine deposits
PERMO-TRIASSIC (NEW RED SANDSTONE)	
Successions outside the Crediton Trough	
	Otter Sandstone
	Budleigh Salterton Pebble Beds
	Aylesbeare Mudstone Group
	Clyst St Lawrence Formation
	Dawlish Sandstone
	Monkerton Formation
	Heavitree Breccia
	Alphington Breccia
	Whipton Formation
Succession inside the Crediton Trough	
	Dawlish Sandstone
	Yellowford Formation
	Shute Sandstone
	Newton St Cyres Breccia
	Crediton Breccia
	Creedy Park Sandstone
	Thorverton Sandstone
	Knowle Sandstone
	Bow Breccia
	Higher Comberoy Formation
	Cadbury Breccia
	Upton Formation
	Exeter Volcanic Rocks
CARBONIFEROUS	
Upper	Bude Formation
	Crackington Formation
	Ashton Shale
Lower	Teign Chert
	Combe Shale
UPPER DEVONIAN to LOWER CARBONIFEROUS	
	Trusham Shale and Hyner Shale (undifferentiated)
IGNEOUS ROCKS (except for Exeter Volcanic Rocks)	
	Lavas - Spilites, Lamprophyres.
	Tuffs
	Minor Intrusions - Dolerites.
	Dartmoor Granite.

DATA ANALYSIS

The data were analysed using a number of procedures including summary statistics, histograms and X-Y plotting routines. The results of this analysis enabled the lithostratigraphical units to be described in terms of their geotechnical properties and likely behaviour in engineering operations.

The use of statistics in analysing and summarising geotechnical databases has been examined by Hallam (1990), and his recommendations have been followed in this report. The statistical methodology requires the use of a robust non parametric approach. Thus for each parameter the mean, maximum, minimum and standard deviation are avoided because they are only representative for data which are normally distributed about the mean value. They are also sensitive to extreme data values at the ends of the data distribution and to the effects of spurious values. A more representative summary is given by the use of percentile values. In this report, 2.5, 10, 25, 50 (median), 75, 90, and 97.5 percentile values may be quoted depending on the number of values in the data set. This approach has the advantage of giving the most likely value to be found for a geotechnical parameter (the median) and to demonstrate how the other values in the data set are distributed about that value. The percentile values are insensitive to spurious values which fall on the extremes of the distribution. The summarised results of the data analysis are given in Tables 2-20 .

ENGINEERING GEOLOGY OF BEDROCK MATERIALS

Trusham Shale and Hyner Shale (Undifferentiated)

The Hyner and Trusham shales form minor outcrops in the south west of the district. They comprise silty shale metamorphosed to hornfels within the metamorphic aureole of the Dartmoor Granite. No geotechnical details were available.

Combe Shale

The Combe Shale is a black SHALE with white silty laminations. The upper and middle parts of the shale have been baked by dolerite intrusions resulting in its conversion to a hard splintery rock (adinole and hornfels) for a few metres width next to the intrusion. Exceptionally up to 15 m may be affected as at [SX 8571 8596] south of Harehill Plantation, Doddiscombsleigh. No geotechnical details were available.

The dolerites occur as sills mostly less than 50 m thick and comprise massive, hard, grey, DOLERITE which weathers spheroidally; no geotechnical details were available.

Teign Chert

The Teign Chert comprises mainly black, bluish black, or grey, very thin to medium bedded, CHERT with very thin to thin inter-beds of grey to greyish green SHALE. The upper part ('Posidonia beds') is predominately siliceous SHALE with CHERT and scarce, thin LIMESTONE beds.

Individual chert beds range from a few millimetres thickness up to 0.3 m and the shale inter-beds from a few millimetres to about 80 mm. Several thin tuff beds are present, mainly in the middle part and range in thickness from a few millimetres to about 40 m. No geotechnical data were available for the Teign Chert, but it is likely to be abrasive to digging equipment and cause a high rate of wear.

Crackington Formation

The Crackington Formation is a folded, grey, weak to very weak, shaley heavily overconsolidated MUDSTONE with interbeds of moderately strong to strong sandstone in a ratio of between 3:1 and 4:1. It weathers to a stiff to hard fissured CLAY with sandstone fragments. It is commonly covered by Head and may be weathered to a considerable depth. In a borehole at Exwick Barton [SX 9003 9457], the Crackington Formation was weathered to a depth of 11.4 m.

The standard penetration test data show the consistency of the Crackington Formation (weathered material) to range from soft to hard, but to be generally stiff. Median moisture content is 23% with a range of 15% to 35%at the 10/90 percentiles level. The plasticity chart shows it to be an inorganic clay of low to intermediate plasticity with material occasionally in the high or very high plasticity classification (Fig. 2).

The geotechnical properties and natural slope instability of the Crackington Formation have been studied in detail by Grainger (Grainger and George, 1978; Grainger and Witte, 1981; Grainger, 1983; Grainger, 1984; Grainger and Harris, 1986; Grainger, 1985). He showed that many slopes exhibit topographic features due to active or recent shallow translations, slides, earthflows or creep. Individual slips are of the order of 10 - 20 m wide, 15 - 40 m long and 2 - 4 m thick, although many movements are formed by the joining together of number of individual slips. The instability is largely confined to the periglacially remoulded material in the near surface zone. Some parts of the outcrop are more prone to instability than others, and this has been attributed to differences in the clay mineralogy of the parent rock. Grainger (1984) recognised four clay mineral assemblages:

1. Illite.
2. Illite plus 5-50% kaolinite.
3. Illite plus minor amounts of kaolinite and chlorite.
4. Illite plus 5-50% chlorite.

The illite and illite/kaolinite assemblages are more readily broken down than the others and give rise to a weathered material of low shear strength. This assemblage is prominent in the dark pyritic shales with thin infrequent sandstones (Ashton Shale

Member) which occur at the base of the Crackington Formation.

Grainger and Harris (1986) identified four landscape types developed on the Crackington Formation:

1. Upland flats and very low angled slopes, deeply weathered and disturbed by periglacial action. At their margins even very shallow slopes exhibit slow earthflows.

2. Convex to straight upper valley sides at angles up to 35°, thin superficial material on sandstones and individual mudrocks showing hill creep.

3. Concave lower valley sides. Weathered bedrock and accumulations of soliflucted material and slipped Head from upslope with relict shear surfaces. Landslips are prone to reactivation by construction work. Natural activity is by shallow translational slide and flow triggered by groundwater concentrations.

4. Stream valley bottoms where local undercutting of soil and weathered rock causes rotational sliding.

Grainger and Harris (1986) quoted a range for the residual angle of friction, ϕ'_r, for landslide material of 15° to 25°. These values of residual internal friction and the Skempton DeLory equation (Skempton 1957) give a maximum stable slope angle, with the water table at the surface, in the range 7.5° to 13°. However, Grainger and Harris (1986) pointed out that a perched water table, held up by impermeable remoulded material, may develop in the near surface zone. Substantial amounts of water may also be present as an underflow below the impermeable zone and may create artesian pressures below the lower part of hillslopes to the detriment of slope stability. This would enable movement to take place at angles less than those predicted by the Skempton, DeLory equation. Clay gouge on faulted surfaces may act as aquicludes, resulting in localised concentrations of groundwater which cause slope instability.

The overall stability of artificial faces in the Crackington Formation is difficult to predict due to its highly folded, fractured nature. Each face must be considered individually and its stability assessed in the light of the interaction between the cut face and the pattern and frequency of discontinuities. Design slope angles which have been used range from 1:1.5 (37.5°) in sound rock to 1:2.5 (24°) in weathered and highly broken rock. Local instabilities will be controlled by the local dip of the bedding in relation to the slope face, water inflow, faulting and folding. In some cases support by an engineered structure may be necessary.

Excavation may be accomplished by digging in the weathered material and ripping in the less weathered shale and sandstone. Where more massive sandstones are locally present blasting may be required. The excavated material is generally suitable as fill at an optimum moisture content of about 17.5%, giving a maximum dry density of about 1.79 Mg/m^3.

Although in places pyritic, the Crackington Formation generally conforms to class 1 conditions for sulphate attack on buried concrete (Anon., 1981a); pH is slightly acidic. Therefore it is unlikely that special concrete mixes will be required for concrete structures below ground level. A summary of geotechnical properties is given in Table 2.

Bude Formation

The Bude Formation consists of two main lithological groups: thinly bedded SILTSTONE and MUDSTONE with interbeds of fine grained SANDSTONE; and thick blocky SILTSTONE and massive SANDSTONE in beds up to 7 m thick. On weathering the Bude Formation does not weaken as much as the Crackington Formation and the increased proportion of sandy material renders the resulting clays less plastic. Sherrell (1971) quoted a range for the residual angle of friction for remoulded weathered mudstone of the Bude Formation, at the Nag's Head Landslip, Cullompton of 23° - 26°. This landslip in a road cutting was the result of the interaction of geological structure and groundwater. The excavation of the cutting was achieved by motor scrapers in the weaker rock, but required blasting and mechanical face shovels to excavate the massive sandstones. A similar figure for residual angle of friction of 25° is quoted by Crofts and Berle (1972) for a shaley clay of undifferentiated Culm which had slipped at Tiverton, Devon. Natural slopes rarely exceed 14° and most slopes appear stable. However, landslips have taken place on a few steeper slopes of 16°-20°.

Crediton Trough Successions

Upton Formation

The Upton Formation is an orange brown to purplish brown, slightly gravelly CLAY and clayey GRAVEL. The gravel clasts are angular to sub rounded sandstone fragments and the clay is composed of sand grade shale fragments weathered to a clay with a remnant platy texture. The formation may be a Permian mudflow deposit derived from a Crackington Formation source (Edwards, 1988). No geotechnical data were available but it is likely that its geotechnical behaviour is similar to that of weathered Crackington Formation clays.

Cadbury Breccia

The Cadbury Breccia is a brown to reddish brown, sandstone BRECCIA with a clayey, sandy SILT matrix. The maximum clast size is about 0.35 m diameter near the base of the deposit, decreasing to about 0.07 m at the top. A spring line may be present at the base of the breccia where it meets less permeable rocks of the Bude or Crackington formations. Natural slopes in the Cadbury Breccia are generally less than 17° and appear to be stable. Near-vertical cuttings may remain stable for many years; for example the 3 m high cutting at [SS 9041 0309] near Raddon Hill Farm, Thorverton. No geotechnical data were available for the Cadbury Breccia.

Higher Comberoy Formation

The Higher Comberoy Formation comprises a lower unit of reddish brown clayey SILT, silty CLAY and fine-grained SAND, a middle unit of reddish brown very fine grained BRECCIA and an upper unit of clayey, very fine grained SAND. No geotechnical data were available. The Higher Comberoy Formation may be frost susceptible.

Bow Breccia

The Bow Breccia is a BRECCIA with a silty SAND or sandy SILT matrix. No geotechnical data were available.

Knowle Sandstone

The Knowle Sandstone is a reddish brown, well cemented, medium- to coarse-grained SANDSTONE with minor beds of BRECCIA present in some parts. No geotechnical data were available.

Thorverton Sandstone

The Thorverton Sandstone is predominately a reddish brown, fine- to very fine grained, silty SANDSTONE with some thin beds of CLAY and BRECCIA. The sandstone is generally weakly cemented and weathers to sand, although some well cemented beds are locally present. Natural slopes reach maximum angles of 12°. In the better cemented material, near vertical road cuttings up to 3 m high, have stood for at least 100 years [SS 9092 0255]. No geotechnical data were available. The Thorverton Sandstone may be frost susceptible.

Creedy Park Sandstone

The Creedy Park Sandstone is a reddish brown, clayey, silty, fine grained SANDSTONE with thin interbeds of siltstone, mudstone and breccia. Standard penetration test data show the weathered sandstone to be medium dense near the surface increasing to dense or very dense at a depth of 5 m.

Point load index tests normal to the bedding indicate a value for unconfined compressive strength of about 1 MPa at about 7 m depth. This classifies the Creedy Park Sandstone as a very weak rock. A summary of geotechnical data is given in Table 3.

Crediton Breccia

The Crediton Breccia is a brownish red or reddish brown, weakly to moderately cemented BRECCIA with clasts of Culm sandstone, slate, vein quartz and igneous rocks up to 60 mm diameter, in a clayey SILT matrix. Beds of red CLAY are common throughout.

Penetration test data generally show medium to very dense material even near to the surface. Many values fall beyond the range of the standard test, but extrapolated values indicate an increase in strength with depth to 7 m depth.

Moisture content ranges from 10% to 13%. Plasticity data for the clayey silt matrix indicates a clayey silt of intermediate plasticity. Unconfined compressive strength, based on point load tests, ranges from about 0.5 MPa to 16 MPa (exceptionally 40 MPa) with a median value of 4 MPa. Point load tests normal to bedding, median value 5.4 MPa, are much higher than those parallel to bedding, median value 1.5 MPa. The Crediton Breccia is classified as a very weak rock. The Crediton Breccia forms natural slopes up to 7° which appear stable. A summary of geotechnical data is given in Table 4.

Crediton Breccia (Yendacott Breccia)

That part of the Crediton Breccia formally called the Yendacott Breccia is a reddish brown, weakly cemented, clayey, fine grained BRECCIA with thin (10 mm) beds of reddish brown silty CLAY. The clasts are up to 70 mm diameter and are set in a gravelly, sandy, silty clay matrix. Natural slopes do not exceed 16° and appear stable but some steeper artificial cuttings are unstable. The Yendacott Breccia passes laterally, with increasing clay content and decreasing grain size, into the Yellowford Formation.

Geotechnical data for the depth range 0 to 4.6 m from near Crediton indicate generally medium dense material with a mean SPT value of 28, mean bulk density of 1.96 Mg/m^3, intermediate plasticity and a mean moisture content of 13%.A summary of geotechnical data is given in Table 5.

Newton St Cyres Breccia

The Newton St Cyres Breccia is a brownish red, locally well cemented, generally fine- to medium-grained BRECCIA with a matrix of silty clayey SAND; clasts rarely exceed 50 mm. Weakly cemented medium- to fine-grained silty, clayey SAND is present locally.

Standard penetration test data indicate increasing density from 0 to 5 m depth and range from 10 (medium dense) to 60 (very dense). In less weathered material, below 5 m depth, unconfined compressive strength values, based on point load index tests, show strength increases from about 2 MPa at 9 m to 10 MPa at 13 m depth. Point load strength normal to bedding , median value 3.2 MPa, is greater than point load strength parallel to the bedding, median value 0.9 MPa. The Newton St Cyres Breccia is classified as a weak to moderately weak rock.

Chemical tests indicate neutral pH and class 1 conditions (Anon., 1981a) for sulphate attack on buried concrete.

Natural slopes up to 16° on Newton St Cyres Breccia appear stable and it forms stable, near vertical, slopes in cuttings such as the railway cutting [SX 8588 9906] and the road cutting [SX 8790 9804] on the A377 near Newton St Cyres. A summary of geotechnical data is given in Table 6.

Shute Sandstone

The Shute Sandstone comprises reddish brown, weakly cemented, very silty, clayey, very fine-grained SANDSTONE and sandy SILTSTONE which weather to silty clayey SAND and sandy SILT. Some lenses of breccia and sandy silty clay are present. Natural slopes appear to be stable and reach a maximum of 13° where they are capped by River Terrace Deposits. However, a cutting for farm access near Shute Cross [SS 893 001] has required battering to ensure stability. No geotechnical data were available.

Yellowford Formation

The Yellowford Formation consists of silty, sandy, MUDSTONE, clayey, sandy SILTSTONE, and silty, clayey fine grained SANDSTONE. Beds of fine - grained BRECCIA are present locally. The formation weathers to CLAY, SILT, fine-grained SAND and sandy silty CLAY. Natural slopes do not usually exceed 4°. No geotechnical information were available.

<u>Successions outside the Crediton Trough</u>

Whipton Formation

The Whipton Formation is for the most part silty, fine-grained SANDSTONE but comprises a wide variety of lithologies, ranging from breccia to mudstone. It is therefore not possible to give a general description of its engineering properties.

The geotechnical database contains information on 39 samples, which are described mainly as silty clayey SAND with some clayey SILT and MUDSTONE.

Penetration test data show the samples to be loose near the surface and to increase in relative density with depth, becoming very dense below about 5 m. The more cohesive material is of stiff to hard consistency and plasticity data show it to be a silty inorganic clay of low to intermediate plasticity with a shrinkage limit of 11%.

The Whipton Formation is usually poorly cemented and forms natural slopes of less than 4°, except where active stream erosion leads to localised near vertical slopes which are often subject to instability, such as at the near vertical stream section south of Poltimore Bridge [SX 9417 9342]. A railway cutting in the Whipton Formation near St James's Park Halt [SX 9264 9348], which was cut at an angle of 30°, has remained stable for many years but is not typical of the formation as a whole as the cut is in a relatively well cemented breccia. Stable artificial slopes of 40° to 50° in weathered Whipton Formation are reported to the west of St Thomas in Dunsford Road [SX 904 9914], Hamber Lane [SX 903 911] and Little Johns Cross Hill [SX 902 911]. The long term stability of cut slopes will depend on the lithology concerned, the position of the water table relative to the face, and the degree of cementation of the material. The stability of poorly cemented material is difficult to calculate because neither soil mechanics nor rock mechanics methods are totally applicable. However, it appears that in the poorly cemented sandstone, slopes of up to 50° may be stable in the long term if they are above the water table and are protected from erosion by surface water flows. Uncemented or badly weathered material and situations where a perched water table form a seepage line on the face, will require slopes to be designed to lower angles and some provision should made to drain the face. A summary of geotechnical data is given in Table 7.

Alphington Breccia

The Alphington Breccia is a low strength, weakly cemented, shale-rich BRECCIA which weathers to a soft to stiff, sandy CLAY.

Natural slopes are usually below 8°, but a few slopes up to 15° occur. No natural instability has been observed. Artificial cuts are usually made at angles of about 25° (1:2). In Exeter, where land values are higher and land take has to be kept to a minimum, steeper angles have been used with supporting structures to maintain stability. However, the near vertical river cliff at the Quay [SX 9144

9266] stands well without support and shows only slight signs of erosion. Clast size is usually less than 40 mm diameter but may range up to 100 mm diameter and boulders of quartz porphyry may occur locally making excavation by digging more difficult. No geotechnical data on this formation are included in the database.

Heavitree Breccia

The Heavitree Breccia is a moderately weak to moderately strong (5 MPa - 50 MPa) BRECCIA composed of gravel and small boulders in a matrix of poorly sorted sandy clay or clayey sand. The breccia weathers to a medium dense to dense gravelly clayey SAND or soft to stiff gravelly sandy CLAY. Standard penetration test values show a general increase with depth (Fig. 3) and range from 0 - 50 for weathered material to much greater than 50 for unweathered breccia. Material may show significant weathering to a depth of eight or nine metres from the ground surface. Moisture content ranges from 2% to 22% near to the surface but the range decreases with depth to about 10% to 15% below 3 m depth.

Plasticity data for the weathered matrix indicate that it is a sandy or silty inorganic clay of low plasticity.

Bulk and dry density values, median values 2.13 Mg/m^3 and 1.91 Mg/m^3, show a wide scatter and no distinct relationship with depth. The wide scatter may be due to the difficulty of sampling coarse material in an undisturbed state.

Particle size analysis data (excluding particles larger than gravel fraction) confirms the coarse nature of the breccia and the poorly sorted nature of the matrix.

The undrained cohesion of the weathered matrix ranges from 5 kPa - 105 kPa (soft - stiff) over a depth range of 0 to 8 m and shows a good correlation of increase in strength with increasing depth (Fig.4). The strength of the unweathered breccia is estimated at between 5 and 50 MPa depending on the degree of cementation. Point load strengths have been recorded of 88 MPa (median axial value) and 0.48 MPa (median diametral value) which correspond to unconfined compressive strengths of 22 and 11 MPa.

Rock quality designation values range from very poor to good, mainly poor to good (Fig 5) and show an increase in rock quality with depth (Fig.6) reflecting the effect of weathering on strength. In the Exminster area, open joints up to 10 cm in width have been recorded in a quarry at Shillingford Wood [SX 9065 8746] (Bristow, 1984)

Compressibility is likely to be low and consolidation rapid.

Excavation methods in the Heavitree Breccia will depend on the degree of cementation and weathering, but it should be possible to excavate fresh material with heavy ripping plant, although occasionally blasting may be necessary.

Natural slopes are typically up to 30° for scarp slopes and 5° to 10° for dip slopes. Artificially cut vertical faces up to 35 m high have remained stable for many years but, less steep faces may suffer erosion by the action of surface water run off. Cuttings may be designed to slopes of 2:1 (65°) with protection against erosion by runoff or seepage from perched water tables in the cut face. A 3 to 4 m wide strip at the foot of steep cut faces may be used, with a ditch and bank or a catch fence to stop cobbles that may fall from the face. The breccia is locally sufficiently well cemented for use as a building stone, and has been widely used in the Exeter area.

Chemical tests indicate near-neutral pH and class 1 conditions (Anon., 1981a) for sulphate attack on buried concrete. Therefore no special concrete mixes are likely to be needed for concrete structures below the water table. A summary of geotechnical data is given in Table 8.

Monkerton Formation

The Monkerton Formation is generally a weak silty or clayey fine grained SANDSTONE but beds of medium- or coarse-grained SANDSTONE, sandy silty, heavily overconsolidated MUDSTONE,SILTSTONE and thin BRECCIA are locally present. It weathers to a medium dense to very dense silty clayey SAND or firm to very stiff sandy or silty CLAY.

The few geotechnical test data available for the clayey and silty lithologies indicate that they are silty or sandy inorganic clays of low to intermediate plasticity with a moisture content of about 22%. Bulk density and dry density are about 2.04 Mg/m_3 and 1.66 Mg/m^3 respectively. Low compressibility is likely with rapid consolidation. Natural slopes are usually less than 7°. Near neutral pH is expected and class 1 conditions (Anon, 1981a) for sulphate attack on buried concrete are likely to apply. Therefore special concrete mixes for structures below the water table should not be necessary. A summary of geotechnical data is given in Table 9.

Dawlish Sandstone Formation

The Dawlish Sandstone is a weakly cemented fine to medium grained SANDSTONE weathering to a medium dense to dense becoming very dense with depth SAND. Silty and clayey SANDSTONE and SAND are also present and distinct beds of MUDSTONE occur, including some thick mappable units (Poltimore Mudstone, Bussell's Mudstone).

Standard penetration test values, mainly extrapolated values, show a general increase in N value with depth (Fig. 7) but with a broad scatter. This confirms observations that the transition between sand and weakly cemented sandstone is gradual and difficult to determine. Most N values are in excess of 50 indicating very dense relative density (median value 136).

The median value of moisture content is 12% and ranges from about 4% to 26% near to the surface to 4% to 20% at a depth of 8 m. Although a general trend of declining moisture content with depth is discernable on the plot, it is impossible to identify a transition from weathered to unweathered material.

Bulk density data indicate a range of values near to the surface of 1.50 Mg/m^3 to 1.95 Mg/m^3 but at a depth of 4 m and below, this range decreases to 2.05 Mg/m^3 to 2.25 Mg/m^3. This would indicate a change from weathered to unweathered material at a depth of about 4 m (Fig. 8).

Plasticity data for the clayey and silty parts of the unit show a wide spread from inorganic sandy silts of low plasticity to inorganic clays of high plasticity

Particle size distribution data show that most samples of Dawlish Sandstone are fine-grained, sometimes medium-grained. They range from well to poorly sorted with an average sorting value on the moderately to poorly sorted boundary. Locally the sand may become coarse and pebbly. The sand grains are generally sub-rounded to round.

The weakly cemented nature of the Dawlish Sandstone makes the preparation of cylindrical samples for strength testing difficult. This problem of preparation tends to bias the strength testing towards the stronger material. The median undrained cohesion values for sandy or silty clay lithologies is 88 kPa (77 kPa and 149 kPa for 25 and 75 percentiles). The undrained angle of internal friction is 0. The peak friction angle for a rounded well graded soil is between 34° (loose) and 40° (dense) which is close to the two values of 30° and 32° (Cu = 0) for the Dawlish Sandstone in the database.

The Dawlish Sandstone is generally of very low compressibility with rapid consolidation; the weathered upper zones show more compressibility than the unweathered material. The maximum dry density obtained in compaction tests is 1.97 Mg/m^3 at an optimum moisture content of 10%.

Natural slopes in the Dawlish Sandstone do not exceed 13°, but many artificial faces which have been cut at an angle at or near vertical, remain stable for many years, for example the 15 m high face at Bishops Court Quarry [SX 964 914]. Local erosion and instability take place where water drains over the face or perched water tables cause seepage on the face. Perched water tables may be due to thin silt or clay beds within the sandstone. Recent road cuttings have been made at slopes of between 30° and 40° (1:1.5) with a cover of grass or other vegetation as a protection against erosion by surface run-off. Steeply dipping open joints up to 10 cm wide were seen during the construction of the M5 motorway.

Excavation is possible by digging in the weathered zone, but ripping with a light tyred tractor may be necessary in the denser, more cemented material, at depth. Where the silt or clay fraction is high and drainage is impeded, trafficability may be poor in wet weather. Where excavations in weathered sandstone are below the water table, running sand conditions may be met and require appropriate counter measures such as close boarding or dewatering.The silty and fine-grained sand lithologies may be prone to frost heave and affect shallow (<0.5 m) foundations such as concrete paths.

Chemical tests indicate near to neutral conditions and class 1 conditions (Anon, 1981a) for sulphate attack on buried concrete. Special concrete mixes are therefore not likely to be required for structures below the water table. A summary of geotechnical data is given in Table 10.

Clyst St Lawrence Formation

The Clyst St Lawrence Formation comprises reddish brown, SAND, silty SAND, silty, clayey SAND, sandy SILT, clayey SILT and sandy, clayey SILT becoming weakly cemented with depth. Natural slopes do not generally exceed 8° and appear to be stable. The fine sand and silt lithologies may be frost susceptible. No geotechnical data were available.

Aylesbeare Mudstone Group

The Aylesbeare Mudstone consists of reddish brown, slightly calcareous, SILTSTONE and heavily overconsolidated silty MUDSTONE, with some silty SANDSTONE. A few lenticular beds of fine- to medium-grained SANDSTONE occur within the lower part of the group (Exmouth Mudstone and Sandstone) The mudstones and siltstones weather to firm to very stiff CLAY, SILT and silty CLAY. The outcrop of the Aylesbeare Mudstone is covered by a thin layer of round pebbles and cobbles derived from the Budleigh Salterton Pebble Beds.

The median moisture content of the Aylesbeare Mudstone is 16%, but the plot of moisture content against depth (Fig.9) shows a near surface range of about 15% to 30% and a general decrease in moisture content to a depth of 5 m where moisture content ranges from about 10% to 30%. This indicates that weathering and stress relief extend to a depth of at least 5 m.

Field observation shows weathered mudstone in the top 3 - 4 m to be common, but weathered material has been observed at depths of 9 m and in exceptional circumstances weathered mudstone has been found at a depth of 20 m (for example in borehole [SY 0655 9711] near Larkbeare).

Plasticity data (Fig. 10) show the Aylesbeare Mudstone to be mainly an inorganic clay of low or intermediate plasticity with a small but significant spread into the high plasticity field. Bulk density falls within the range 2.00 Mg/m_3 to 2.25 Mg/m^3 (median value 2.14 Mg/m^3) and it appears to be relatively unaffected by depth. Undrained cohesion values show a wide spread from about 10 kPa to over 300 kPa. The widest spread is seen in the near surface zone and only a diffuse general increase in strength with depth is seen (Fig. 11). Limited consolidation data indicate low compressibility which takes place at a slow rate.

Chemical test information in the database indicates that slightly acid conditions prevail with a median pH value of 6.3. Values for sulphate content indicate class 1 conditions (Anon., 1981a) for sulphate attack on buried concrete. However, reference has been made, in some site investigations, to the possibility of high sulphate values of class 2 and 3 conditions being present.

Natural slopes are generally less that 13° and appear to be stable. e Steeper slopes in stream cut ravines, disused marl pits (25°) and in road cutting (30°-45°) appear to be stable, although the presence of perched water tables in silty or sandy beds may impair stability. Effective stress test results indicate a maximum slope angle for long term stability for shallow slides of about 22° for a minimum stability case with zero cohesion and the water table at the surface. A maximum angle of slope for cuttings of 1:2.5 (24°) has been recommended. Particular care should be taken where weathered material or water bearing sands are present.

Excavations of the weathered material should be possible by digging but ripping may be necessary in the harder mudstones. Some inflow of water into excavations through water-bearing sands may take place.

The silty and fine-sandy parts of the Aylesbeare Mudstone may be susceptible to heave under the action of frost in the near-surface zone. A summary of geotechnical data is given in Table 11.

Budleigh Salterton Pebble Beds

The Budleigh Salterton Pebble Beds are brown or red-brown medium dense to very dense silty sandy GRAVEL with well rounded, pebbles, cobbles and boulders mainly of quartzite. Beds of cross-bedded silty SAND and pebbly SAND, commonly lenticular, occur within the gravels.

Geotechnical data in the database are mainly cone penetration test data which may be inaccurate due to the possibility of the cone tip impacting on large cobbles giving anomalously high N values. The median value for N is 44 (dense) with 25 and 75 percentile values of 32 (dense) and 66 (very dense) respectively. No relationship between CPT and depth is evident (Fig. 12). Particle size distribution data indicate the presence of up to about 10% silt and clay, but over 50% are gravel size or greater. Plasticity data for the finer material which forms the matrix between the pebbles indicate an inorganic silty sandy CLAY of low to intermediate plasticity.

The Budleigh Salterton Pebble Beds do not form natural slopes greater than 12°, except where cut by rivers, when slopes up to 25° may form. Artificial slopes, such as quarry faces at Blackhill Quarry [SY 031 852], may stand near to vertical at a height of 10 m. Weathering causes a scree accumulation below such faces, with an overhang at the top formed of material held together by roots. The erosion of steep cut slopes may be aided by perched water tables in the Pebble Beds due to thin beds of silty sand acting as aquicludes.

Excavations in the Pebble Beds should be possible by heavy excavating equipment, although where they are worked for aggregate techniques intermediate between hard rock quarrying and sand and gravel extraction are used. Excavations below the water table would suffer rapid inflow of water and, in the sandier lithologies, running sand conditions.

A spring line may occur at the base of the Pebble Beds at the junction with the Aylesbeare Mudstone. This may cause slope instability on susceptible mudstone slopes below the Pebble Beds.

The Pebble Beds are suitable for use as aggregate. The crushed quartzite has good wear and polishing resistance and the absence of colloidal silica makes it suitable for use as aggregate in concrete. They may be used as backfill in trenches and excavations below the water table, but will need the addition of cohesive soil for use in embankments.

Sulphate or acid attack on buried concrete or services is unlikely in the Budleigh Salterton Pebble Beds. A summary of geotechnical data is given in Table 12.

Otter Sandstone

The Otter Sandstone has a relatively small outcrop in the sheet area and no geotechnical data were incorporated in the geotechnical database. The Otter Sandstone is generally a very dense reddish or yellowish brown, silty, fine- to medium-grained SAND, locally sufficiently well cemented to be termed SANDSTONE. Thin beds of MUDSTONE and CONGLOMERATE may be present at some levels.

Although weakly cemented, the Otter Sandstone forms steep stable natural slopes of up to 45°. Vertical faces up to 12 m high have been cut by the River Otter, but their long term stability is unknown. Road cuttings, up to 15 m deep, have been successfully excavated at angles of 30° to 40°. Older road cuts with near-vertical appear stable up to 3 m high, but above 3 m, up to 8 m the faces show isolated instability due to loose blocks and overhangs, the result of weathering and erosion.

The finer grained sands and silty lithologies of the Otter Sandstone may be susceptible to frost heave and affect shallow founded structures such as paths and light roads.

Excavations should be possible by digging, and only in rare well cemented areas may more vigorous methods such as ripping be required. Excavations below the water table may encounter running sand conditions and require close boarded support or other preventive measures. The thin mudstone and conglomerate beds may act as minor aquicludes and may give rise to perched water tables detrimental to slope or face stability in slopes and cuttings. No geotechnical data were available.

Exeter Volcanic Rocks

Volcanic rocks associated with the New Red Sandstone (the 'Exeter Volcanic Rocks') comprise basalts and lamprophyres which form relatively small outcrops in the district, but their strength relative to the surrounding rocks may make them significant factors in engineering works.

The lavas were affected by weathering soon after their eruption and, in the case of those which form outcrops, again in the recent past. Where they are unweathered they are commonly highly fractured, vesicular and altered by hydrothermal processes. However, in some outcrops, the unweathered rock may be strong to very strong with medium to wide discontinuity spacing , for example in School Wood Quarry [SX 8749 8730] Dunchideock. The highly weathered material is commonly an angular basalt gravel in a firm to very stiff silty clay matrix which may extend to a depth of 2 m, passing down into less altered rock.

Standard penetration test values in the weathered material range from 25 to 91. Moisture content and plasticity data indicate that the clay is an inorganic clay of intermediate plasticity with a moisture content of about 12%.

Diametral point load tests on unweathered but altered basalt core from a borehole gave a median point load strength (normalised to 50 mm diameter) of 1.53 MPa, which corresponds to an unconfined compressive strength of 36.75 MPa. The 10 and 90 percentile values of point load strength are 1.20 and 2.15 MPa respectively, corresponding to unconfined compressive strengths of 28.4 MPa and 50.9 MPa. This classifies the unweathered basalt as a moderately strong rock. The Rock Quality Designation value (RQD) for this borehole was 45% which classifies it as "poor" in terms of rock quality. Unweathered, unaltered basalt from other localities in the district, such as School Wood Quarry [SX 8761 8721], is expected to be stronger and fall in the range strong to very strong (50 MPa to 200 MPa).

In many areas, the highly weathered or fractured nature of the lavas makes excavations possible with heavy digging equipment, but ripping will be required in the harder less fractured parts. Pneumatic rock breakers or blasting will be required in areas where the basalt is unweathered and the discontinuities are medium to widely spaced and tight.

The subsurface extent of the lava is difficult to predict from their surface outcrop; there may also be considerable variation in strength and discontinuity spacing within the lava rock mass. Therefore it is important that, where basalt is present or is suspected, a site investigation is made to determine its sub-surface nature and extent at an early stage of a construction project. Geophysical methods such as magnetic anomaly surveys may be able to assist in this objective. Some indication of potential areas of concealed lava is given in Cornwell et al. (1990). A summary of geotechnical data is given in Table 13.

Granite

Granite forms a relatively small outcrop in the south-west corner of the district. No geotechnical information were available, but data for the granites of Dartmoor and South-West England are available in the literature. Uniaxial compressive strength values for fresh granite from various quarries in Cornwall and Devon (Watson, 1911; Anon., 1972) are in the range 100 MPa to 200 MPa, which classifies fresh granite as a very strong rock. However, weathering processes which have acted in the past have, in many areas, produced a deeply weathered profile of all weathering grades from fresh granite to residual soil. This has reduce the strength of the granite in the weathered zone depending on weathering grade (Fookes et al., 1971; Dearman et al., 1976). The weathering profile is of varying, but possibly considerable, depth and may include core stones of relatively fresh granite in a matrix of coarse granular material.

Foundation conditions for most structures will depend on the state of weathering. Fresh granite (grade 1) will offer excellent bearing capacity and even moderately weathered material (grade III) should offer reasonable bearing capacity. Highly weathered (grade IV) to residual soils (grade VI) offer less bearing capacity and may be subject to differential bearing capacity due to the presence of core stones or uneven fresh bedrock topography. Heavier structures most take into account the fracturing and alteration of the granite below the weathered zone (Dearman and Baynes, 1978; Knill, 1972).

The method required to excavate granite is dependent on the weathering grade: fresh or slightly weathered grades require blasting, while other grades require progressively less vigorous means; residual soil may be dug by hand. The stability of excavations will be controlled by the jointing in fresh and less weathered material and high steep faces may be stable. Highly weathered material is unlikely to be stable in high steep faces.

ENGINEERING GEOLOGY OF SUPERFICIAL MATERIALS

Head

Head is a term used to denote material which has moved down slope by the processes of solifluction, soil creep and hillwash. It is therefore in a more or less remoulded condition and composed of a variable mixture of the bedrock and superficial lithologies upslope of its position. It is inherently inhomogeneous with regard to its lithology and its geotechnical properties.

Three categories of Head have been distinguished in the Exeter district:

1. Older Head
2. Valley Head
3. Blanket Head and Regolith

Older Head covers upland areas east of the River Clyst. The small amount of geotechnical data available indicate it to be a sandy silty CLAY, locally gravelly. Moisture content ranges from 9% to 24% with a median bulk density of 2.11 Mg/m^3. Plasticity data fall mainly in the intermediate class with a few values just into the low and high plasticity classes. Particle size data indicate some samples of a non cohesive nature. Strength data indicate a stiff to very stiff consistency.

Valley Head occupies valley sides and bottoms and is derived from local material. The wide range in composition is demonstrated by the sample lithology histogram (Fig. 18). This shows the variable nature of the Head with compositions ranging from silty or clayey SAND to sandy or silty CLAY. Standard penetration test values range from 4 to 91 with a median value of 21 indicating loose to very dense, generally medium dense, relative density for the sandy lithologies. Moisture content ranges from 8% to 31% (median value 14%) and plasticity data indicates an inorganic sandy clay or clay of low to intermediate plasticity with a few samples showing high plasticity.

Undrained cohesion data for the cohesive lithologies range from very soft to very stiff consistency with a median value of 72 kPa (firm). Values for undrained angle of internal friction range from 0° to 30°, with a median value of 7°. Consolidation data indicate generally low compressibility with a slow rate of consolidation.

The wide range in value shown by the geotechnical properties of Valley Head and their lack of correlation with depth confirm the inhomogeneous remoulded nature of the material.

The terms Blanket Head and Regolith are used to describe material where it was not possible to distinguish transported material from weathered in situ material during the geological mapping. It is most widely developed on the outcrop of the Bude and Crackington formations. Insufficient geotechnical data were available to comment on this material. It is likely that much of the data has been assigned to the bedrock lithology as weathered material in site investigations.

In terms of their engineering behaviour, the three types of Head material may be considered together.

The behaviour of Head is not accurately predictable, although it is related to its source material. It is generally of low strength and may be excavated with digging machinery. It will require support in excavations and although commonly thin it may be up to 6 m thick at the foot of slopes. It may contain perched water tables or water bearing sands which will flow into excavations causing collapse.

The solifluction process which was active in the formation of Head deposits may have left relict shear surfaces within the Head which are potential surfaces on which movement may take place if the toe of the slope is excavated, the slope is loaded, water introduced into the slope or the drainage of water from the slope impeded. Engineering works on slopes covered by thick Head should include a slope stability investigation at the site investigation stage. Lateral change in lithology and hence geotechnical properties may give rise to uneven settlement of a structure and care must be taken to determine the bearing capacity when constructing on Head deposits. Where deposits are not excessively thick it may be desirable to remove them and place the foundation on bedrock. Summaries of geotechnical properties are given in tables 14,15 and 16.

Buried Channels.

The lower sea levels during past ice ages resulted in down cutting by the river Exe to a level below that of its present path. Two phases of down cutting have been identified in the Exe estuary as a whole, the younger being shallower than the older (Durrance, 1969). These channels have since been infilled by superficial deposits, and represent potential problems to the construction of river crossings or construction on the alluvial sediments. The construction of the M5 crossing between Topsham and Exminster showed the presence of a single phase of buried channel formation, with an average depth for the deepest part of 11.4 m below Ordnance Datum. The buried channel is filled by gravel overlain by silty clay and contains buried terrace levels at -7.8 m and 11.4 m below Ordnance Datum (Durrance, 1974).

River Terrace Deposits

The composition of the River Terrace Deposits and therefore their geotechnical properties varies according to the rock types present in the source area. This is demonstrated by the wide range of lithological descriptions in the database. The River Terrace Deposits are mainly gravels, sandy gravels and gravelly sands, but there are significant amounts of finer deposits such as silty or sandy clay, and sandy or gravelly silt (Fig. 18).

Standard penetration test data indicate a generally medium dense to dense relative density. Plasticity data on the finer grained samples indicate inorganic sandy clay of low plasticity. Particle size analysis confirms the sample description of a basically coarse grained non cohesive material with some clay and silt.

Chemical tests indicate near to neutral pH and class 1 or 2 conditions (Anon., 1981a) for sulphate attack on buried concrete; special concrete mixes are not usually needed for buried structures.

Compressibility is expected to range from very low for the gravels to medium for the silts or clays and to take place rapidly on the gravels and at a medium rate on the silts and clays. The only exception is if organic clay or peat are present, in which case high compressibility will prevail; consolidation will be fast for peat but slow for organic clay.

Excavations should be possible with normal digging equipment, but support may be required in the less dense material. Excavations below the water table may meet very high rates of water inflow and running sand.

The River Terrace Deposits should give good foundation conditions but, the presence of pockets of material of contrasting properties (clays, silts organic material) should be investigated and avoided, removed or otherwise dealt with to avoid differential settlement. A summary of geotechnical properties is given in Table 17.

Alluvium (excluding gravels)

Alluvium in the Exeter area is mainly a brown or reddish brown soft to stiff, normally consolidated, silty or sandy CLAY. Lithological descriptions also indicate the presence of some silt, sand, gravel and organic material including peat.

Standard penetration test values give a median N value of 23 and range from 10 to 43 but show little correlation with depth. Moisture content gives a median value of 25% and a depth plot indicates high moisture contents (up to 75%) in the near surface zone but also some low values (8%) in the same zone. The range decreases generally with depth.

Plasticity data show a wide range of values from sandy inorganic clay of low plasticity to inorganic clay of very high plasticity. The bulk of the data fall in the low to intermediate plasticity categories (Fig. 13). Median values for bulk and dry density are 1.95 Mg/m^3 and 1.61 Mg/m^3 respectively.

Particle size data quoted for the sands show them to be silty, clayey sands with a little gravel.

Values for undrained cohesion show a steady decrease from ground surface to 1 metre depth (Fig. 14) and indicate the presence, in some areas, of a desiccated crust about 1 metre thick; undrained cohesion values of up to 460 kPa have been recorded near to the surface. Consolidation data indicate that the Alluvium has medium to high compressibility with a medium to slow rate of consolidation.

Chemical tests show slightly acidic pH and generally class 1 conditions (Anon 1981a) for sulphate attack with occasionally class 2 or 3 conditions present. The median organic content for Alluvium described as "organic" was 5% and ranged from 2% - 17%.

The median values for peak shear strength for undisturbed and remoulded samples are 29 kPa and 7 kPa with sensitivities generally in the range 2 - 4 (medium sensitivity) but Alluvium of higher sensitivity is also present.

Alluvium is easily excavated by normal digging methods, but excavations are likely to require support and precautions against running sand may be needed. Excessive water inflow may be met if underlying water-bearing gravels are penetrated. Very weak soft alluvium may be met below a stronger desiccated crust. The lower material would offer poor trafficability if the desiccated crust were removed or disrupted.

The Alluvium may contain minor ribbon-shaped or lenticular bodies of peat or sand which could cause differential settlement of structures founded across their edge. A summary of geotechnical properties is given in Table 18.

Sub-Alluvial Gravel

The Alluvium of the Exeter area is commonly underlain by up to 6 m of GRAVEL. Standard penetration test data indicate that the gravel is of medium dense to very dense relative density and show no relationship between depth and N value. Particle size analyses show the gravel to be generally a sandy, rounded to sub-angular gravel with little clay or silt (Fig. 17)

Chemical test data indicate slightly acid groundwater, of pH 6.3, and class 1 conditions (Anon., 1981a) for sulphate attack on buried concrete. Special concrete mixes for structures below ground are therefore unlikely to be necessary.

The Gravel should be excavatable with normal digging equipment but excavations may need support to maintain side stability and some provision should be made for dealing with high water groundwater flows into excavations below the water table. The Gravel should provide good bearing capacity where it outcrops or where foundations need to be carried down through soft alluvium to a stronger stratum. A summary of geotechnical properties is given in Table 19.

Marine Deposits

The Marine Deposits of the Exeter area generally consist of very soft to firm, normally consolidated silty clay or organic silty clay with a small proportion of sandy lithologies.

Standard penetration test values are generally low with a median value of 10 indicating soft (cohesive) or loose (non cohesive) conditions.

The median moisture content is 36% with a wide range in value from 20% to 170% down to 2 m depth; below that depth moisture content ranges from 20% to 100%. Bulk density has a median value of 1.82 Mg/m^3 and shows a wide range of values from 1.25 Mg/m^3 to 2.06 Mg/m^3. The wide range in moisture content and bulk density reflect the range of lithologies present, in particular those with a high organic (peat) content give rise to material of very low density and high moisture content.

Plasticity data (Fig. 15) indicate that the Marine Deposits are mainly inorganic clay of intermediate to high plasticity but there are also minor amounts of inorganic sandy clay of low plasticity, organic or silty clays of high plasticity and some clays, silts and organic material of extremely high plasticity. Undrained cohesion values generally fall in the range 0 to 60 kPa (very soft to firm) but higher values up to 100 kPa are found in the top metre indicating that a desiccated crust is present in some areas (Fig. 16).

Compressibility will be generally high with consolidation taking place at a medium to slow rate. Chemical tests indicate near neutral pH and class 1 conditions (Anon., 1981a) for sulphate attack on buried concrete, although if saline groundwater is present, sulphate and chloride attack on buried concrete may be possible.

The Marine Deposits are easily excavated with normal digging machinery but their extremely weak nature will require the sides of the excavations to be stabilised by trench supports, close boarding, bentonite mud or other means. Running sand or excessive inflow of water through sand or gravel lenses may be encountered.

The movement of heavy plant may be difficult where a desiccated crust has been disrupted, removed or is absent. The movement of light plant may also be difficult on the softer deposits.

The extremely weak and compressible nature of the Marine Deposits will require structures founded on this material to have foundations designed to take this into account, such as rafts to spread the load, or piles to carry the load down through the Marine Deposits to stronger material.

It is possible that isolated lenses or pockets of material of strongly contrasting properties to the surrounding material may be present within the Marine Deposits. Materials such as peat or sand, relics of ancient channel infill, may be surrounded by soft silty clay. If the foundations of a structure were placed across such deposits it could give severe differential settlement causing damage to the building, service connections or connecting roads.A summary of geotechnical properties is given in Table 20.

ENGINEERING GEOLOGICAL CLASSIFICATION OF GEOLOGICAL UNITS

The geological deposits of the Exeter area encompass a wide range of material types and strengths. They may be conveniently considered in the following groups in which material of similar properties and engineering geological behaviour are placed together.

1. Strong rocks.

Rocks which, when fresh, may be expected to have unconfined compressive strengths of at least 50 MPa.

2. Interbedded strong rocks and mudstones.

Strong rocks with interbedded mudstones which are much weaker and weather to clay.

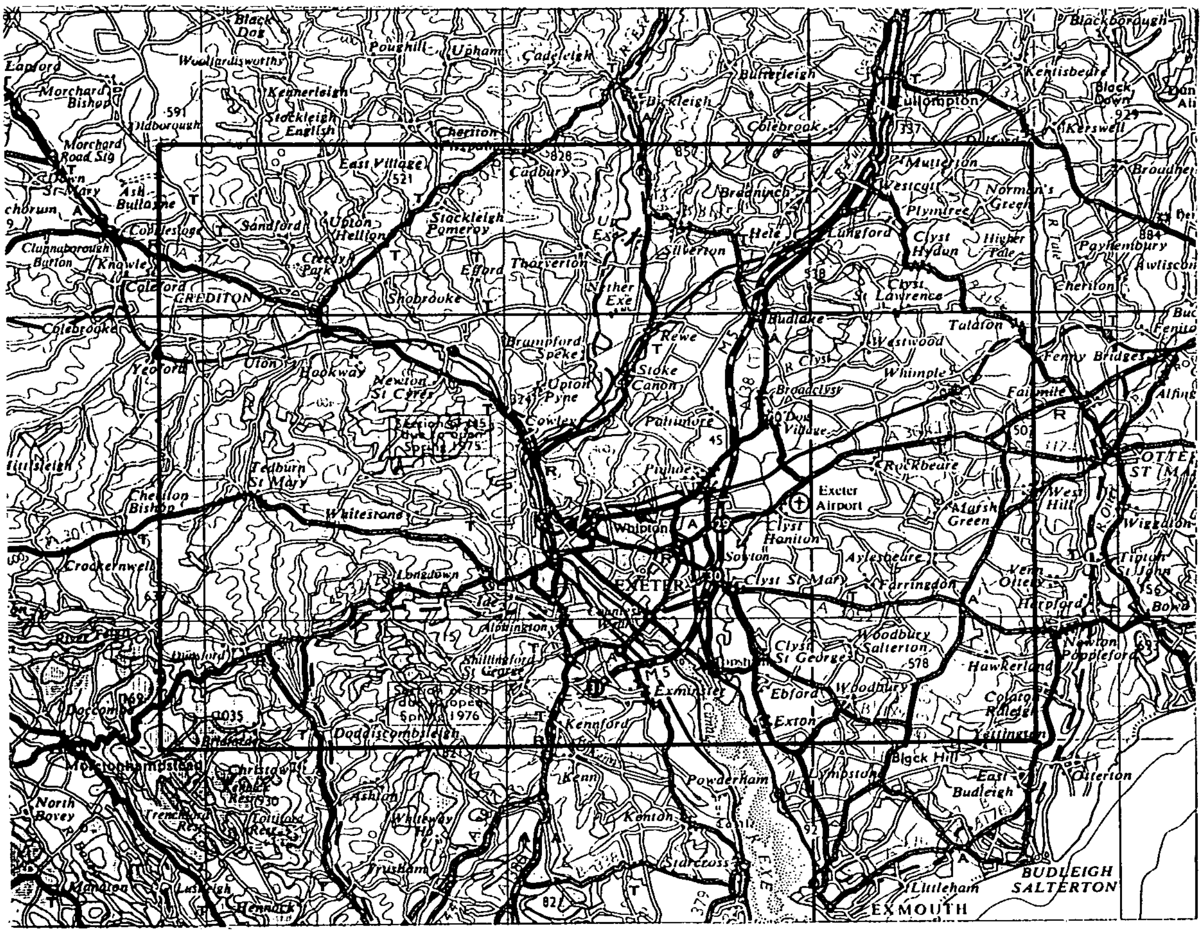
3. Mudstones, shales and clays.

Materials which are predominately argillaceous mudrocks weathering to clay and may be subdivided into overconsolidated and normally consolidated mudrocks

4. Granular rocks and deposits.

Materials which are essentially composed of grains or clasts more or less bound together by a cement or matrix of finer material. These may be subdivided into coarse and fine materials each subdivided in terms of the strength of the inter-granular bonding.

This classification forms the basis for the engineering geological map of the Exeter district (Fig. 19).



1:250 000

Fig. 1 Outline of the area covered by 1:50 000 geological map sheet 325, Exeter sheet

Fig. 2 CRACKINGTON FORMATION

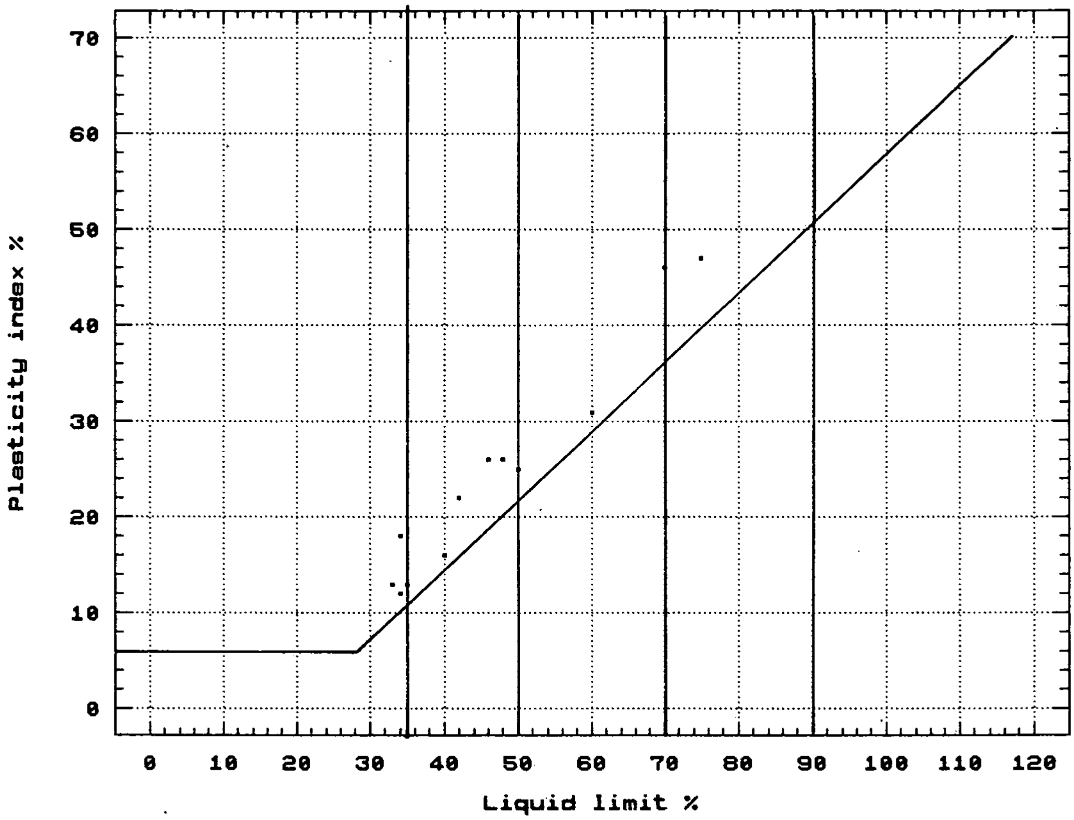


Fig. 3 HEAVITREE BRECCIA

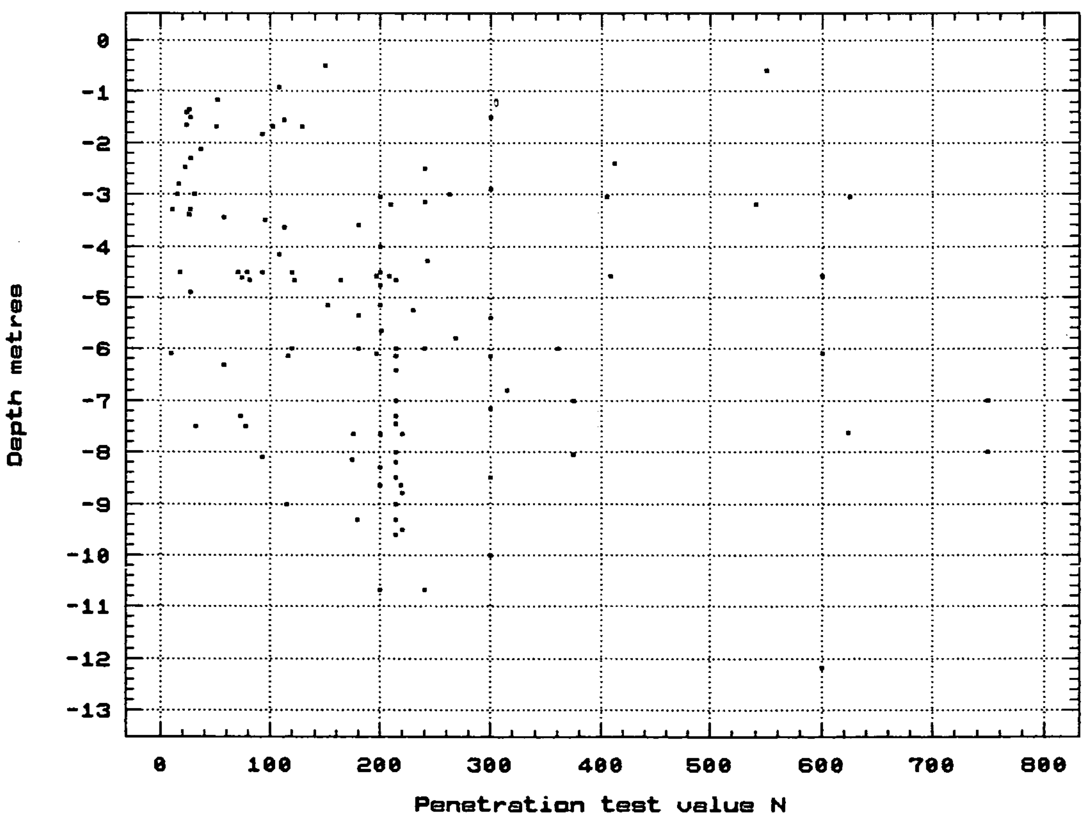


Fig. 4 HEAVITREE BRECCIA

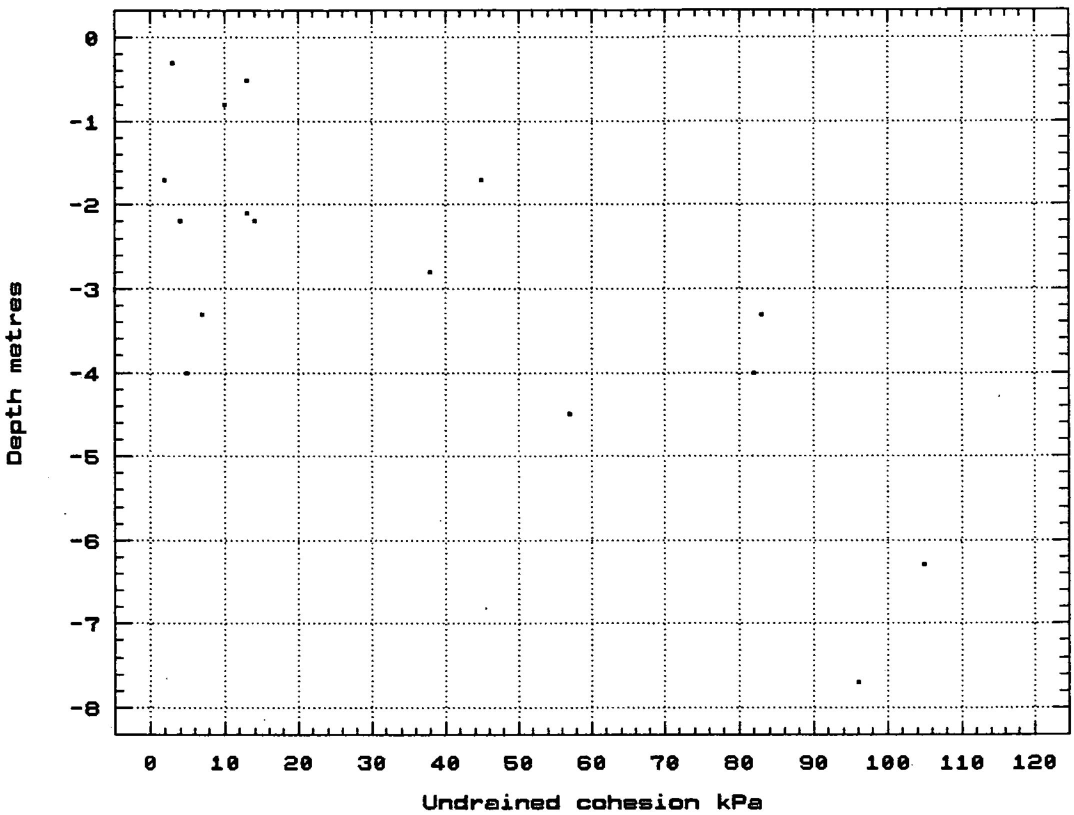


Fig. 5 HEAVITREE BRECCIA

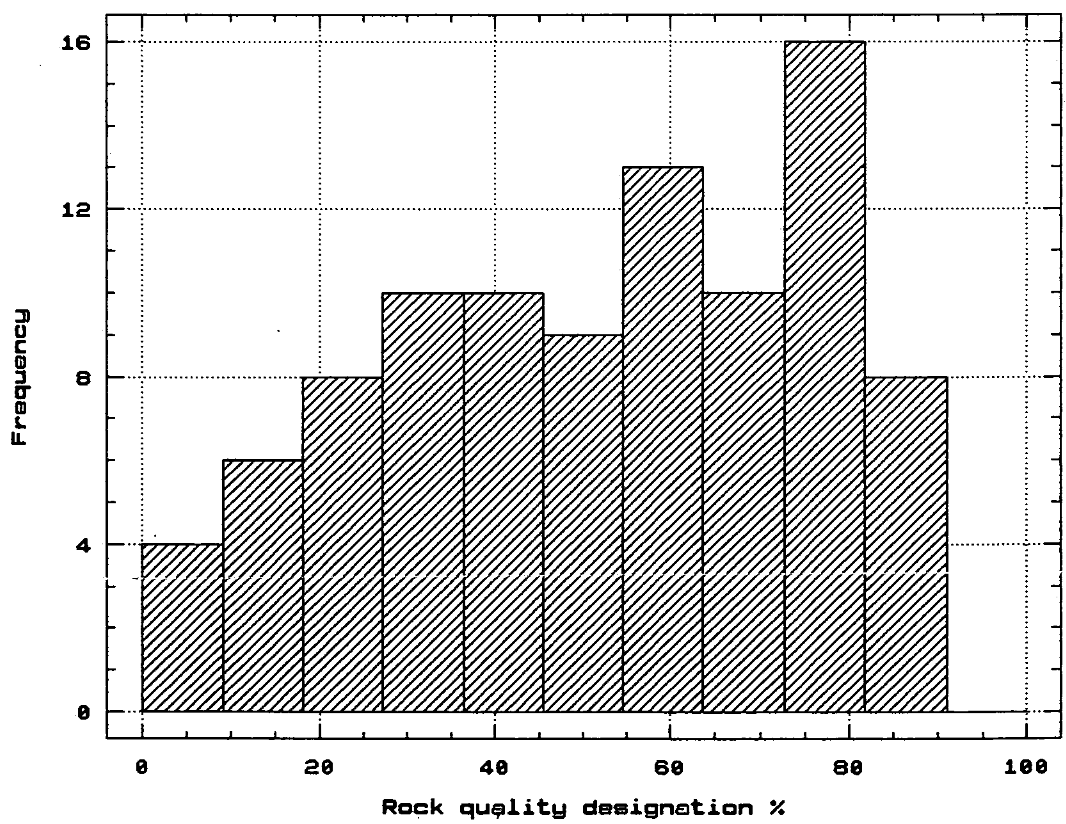


Fig. 6 HEAVITREE BRECCIA

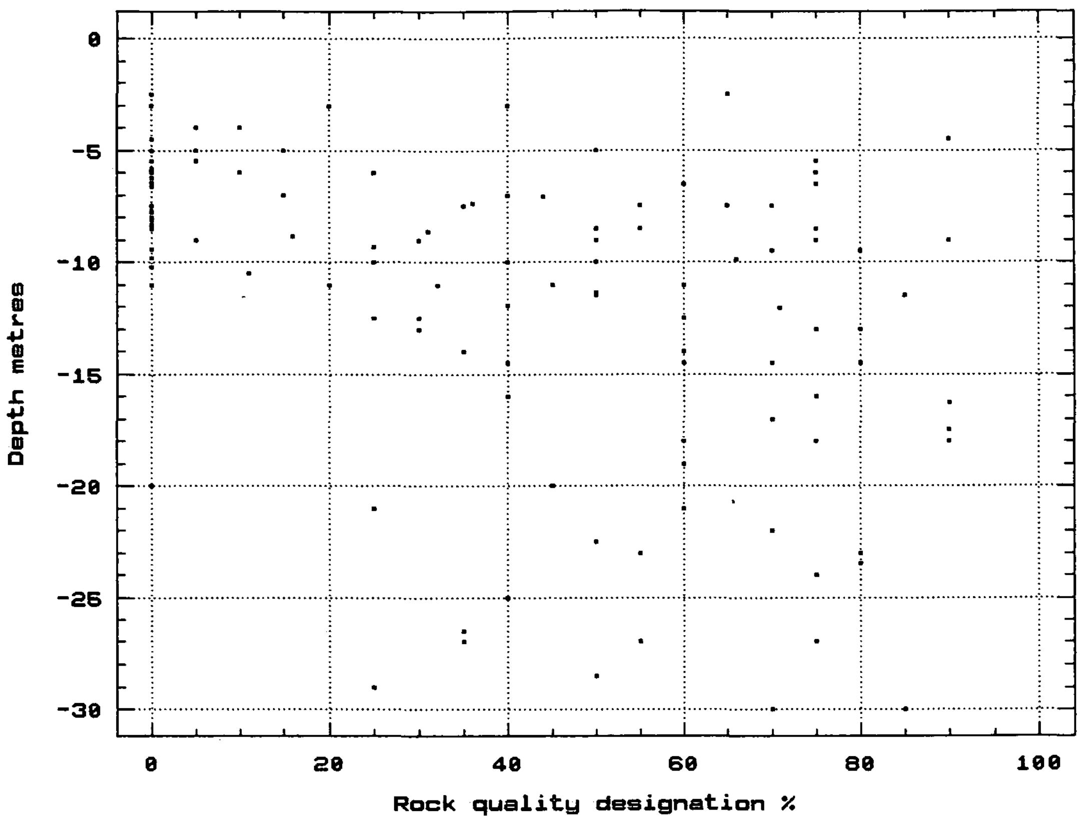


Fig. 7 DAWLISH SANDSTONE

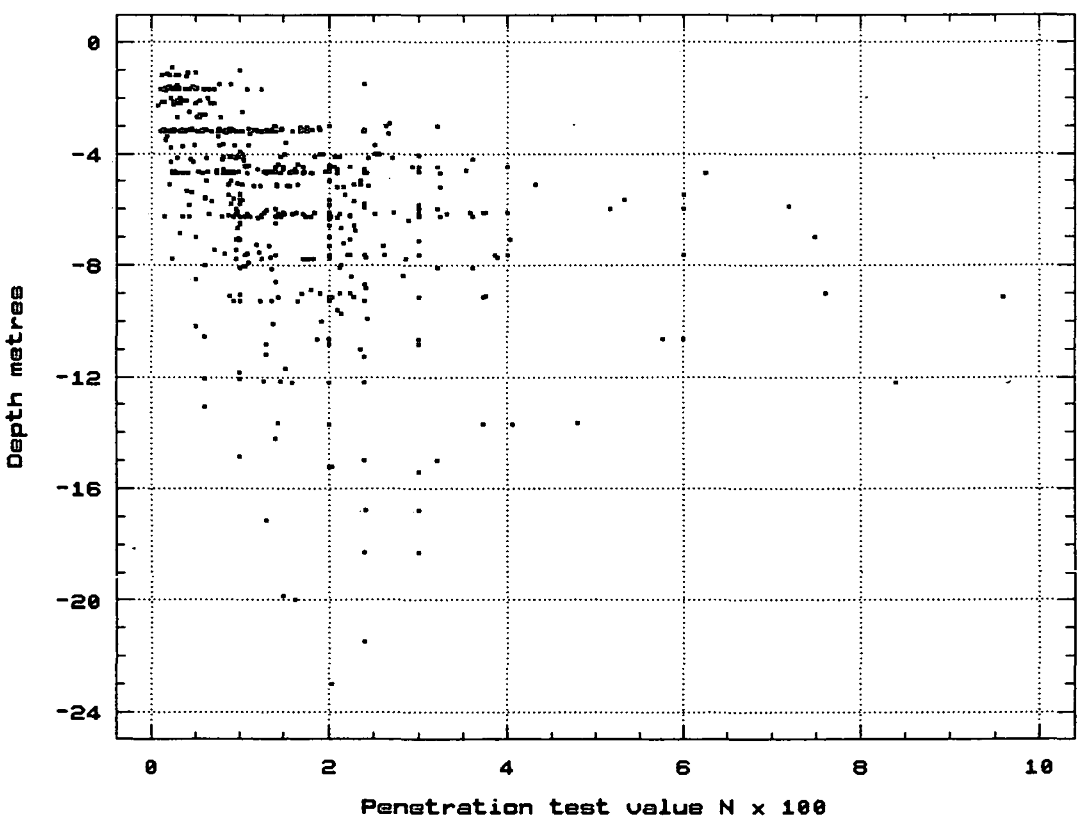


Fig. 8 DAWLISH SANDSTONE

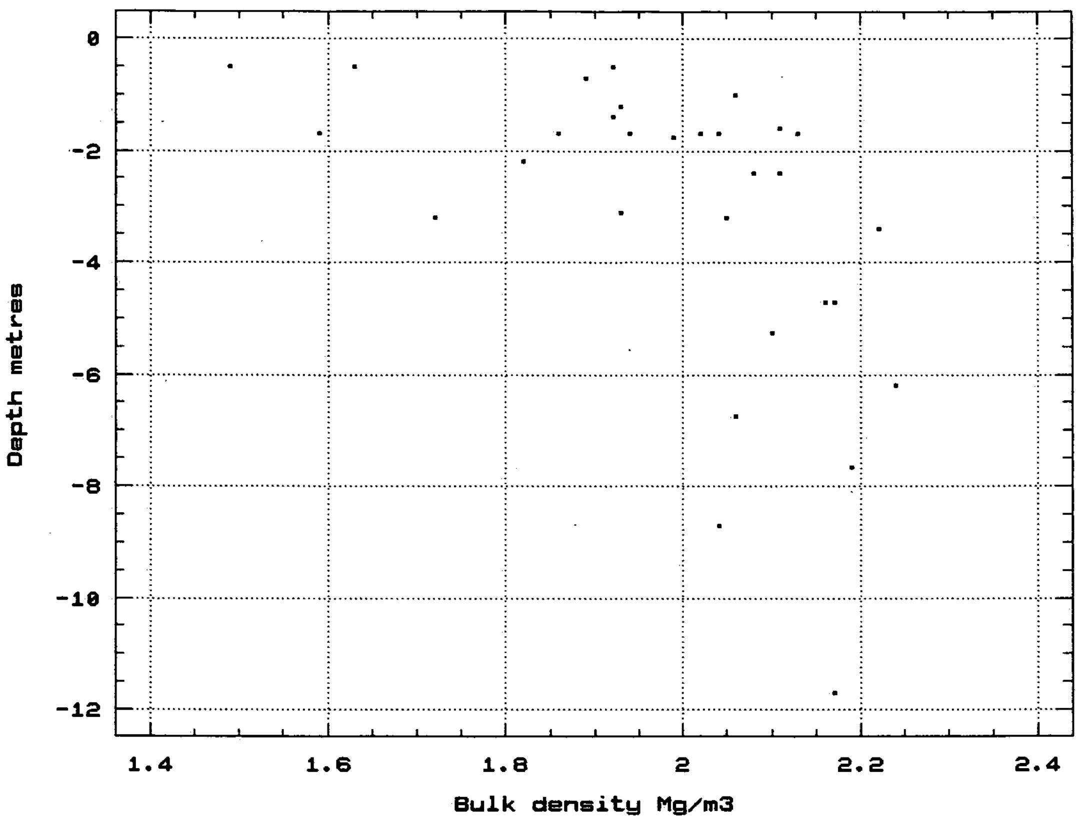


Fig. 9 AYLESBEARE MUDSTONE

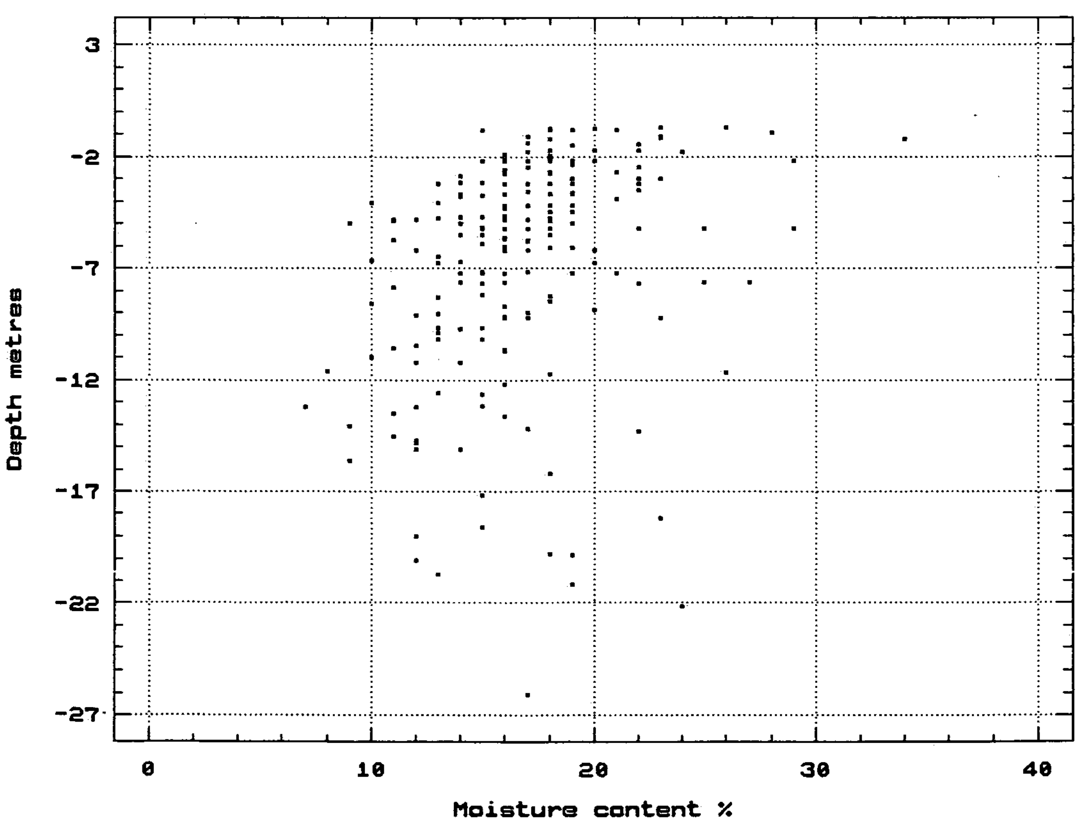


Fig. 10 AYLESBEARE MUDSTONE

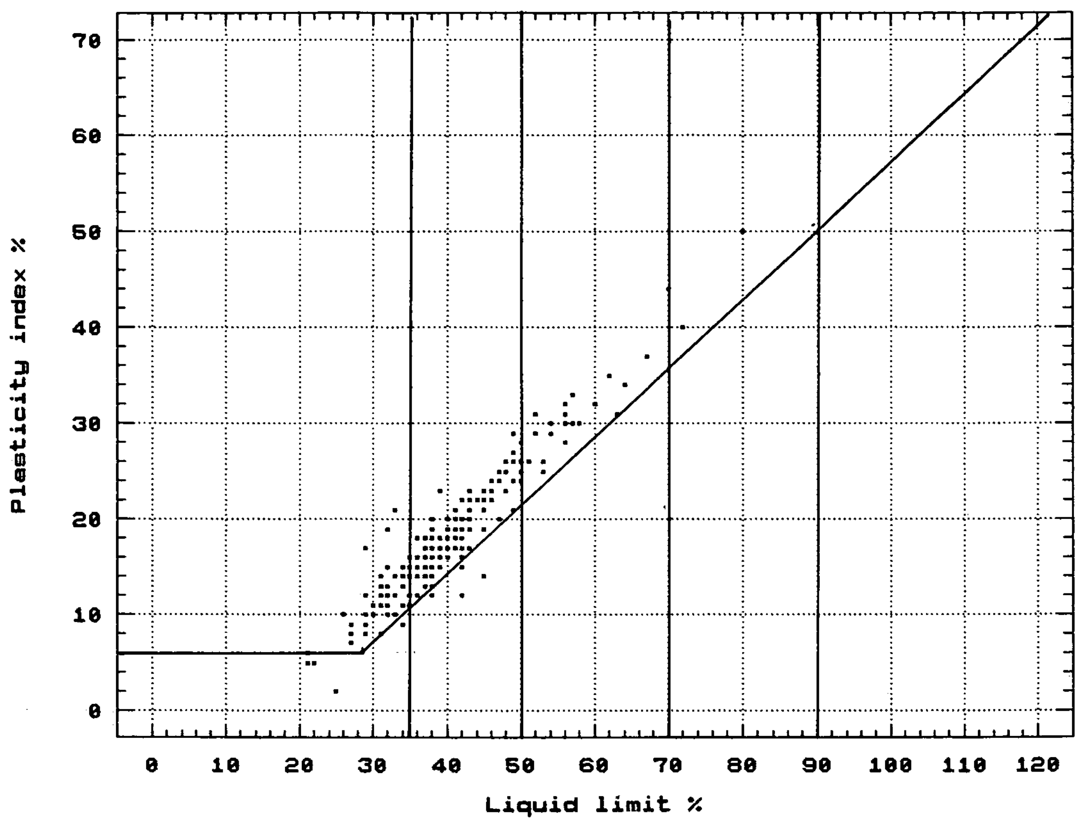


Fig. 11 AYLESBEARE MUDSTONE

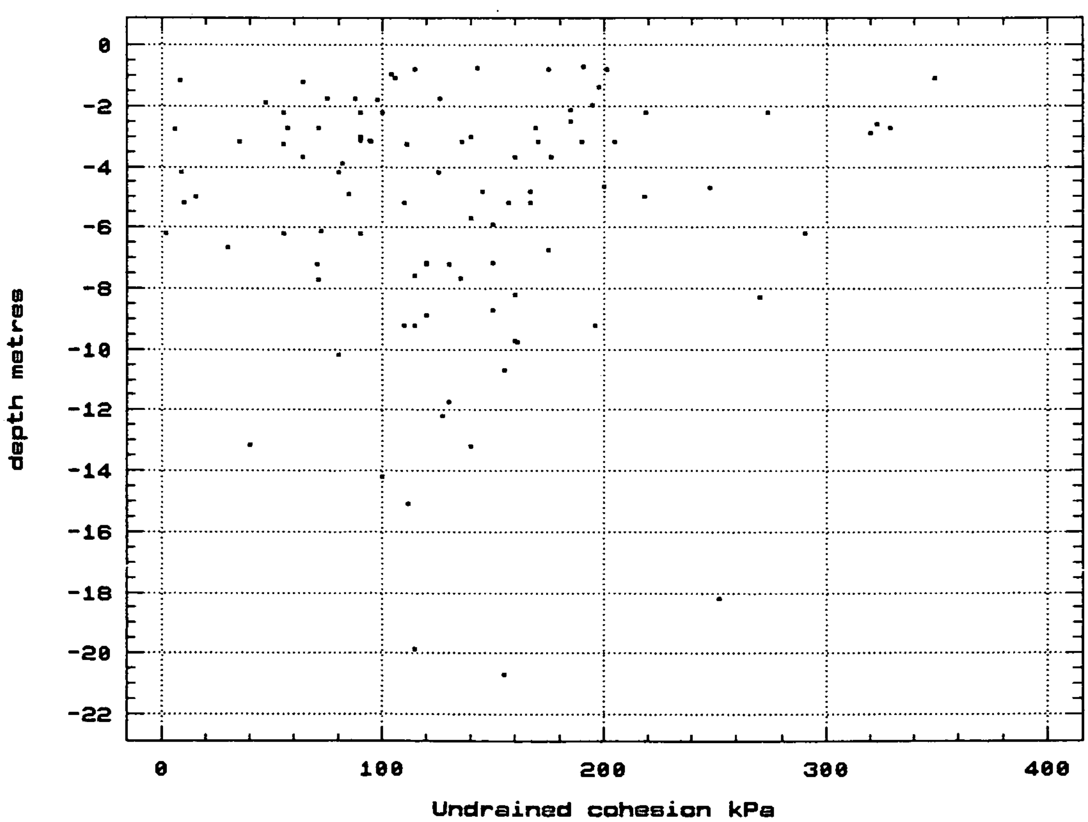


Fig. 12 BUDLEIGH SALTERTON PEBBLE BEDS

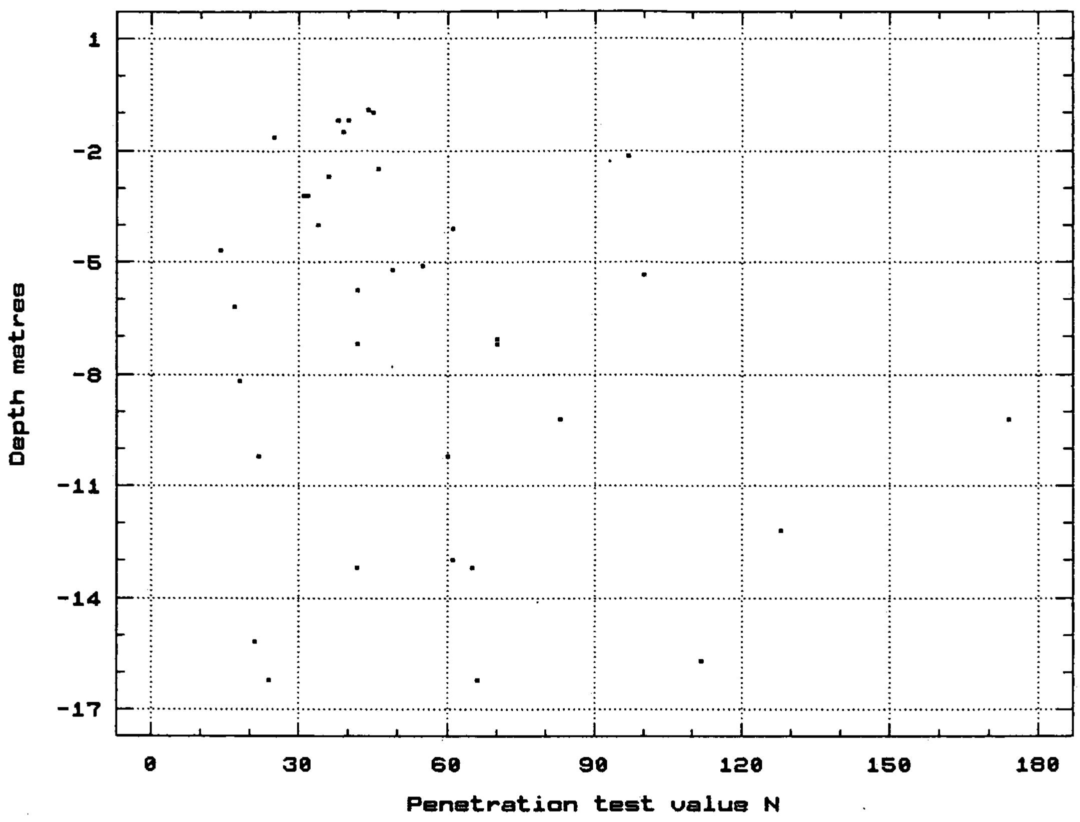


Fig. 13 ALLUVIUM

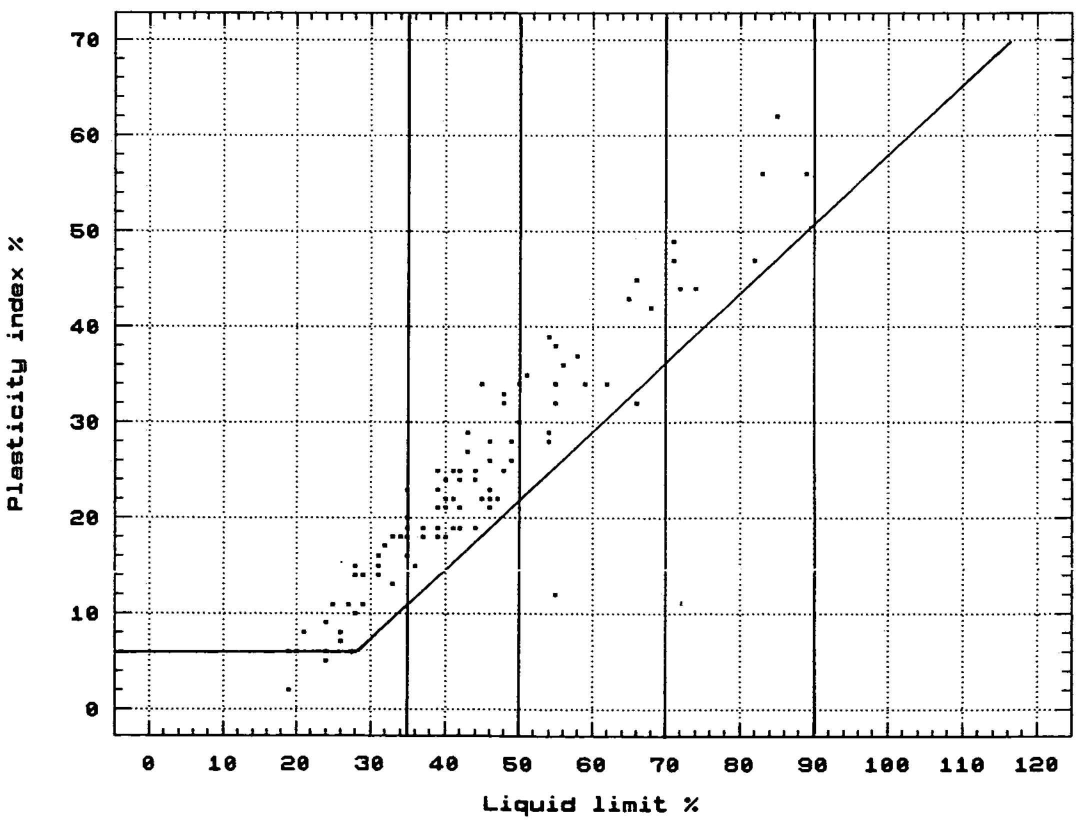


Fig. 14 ALLUVIUM

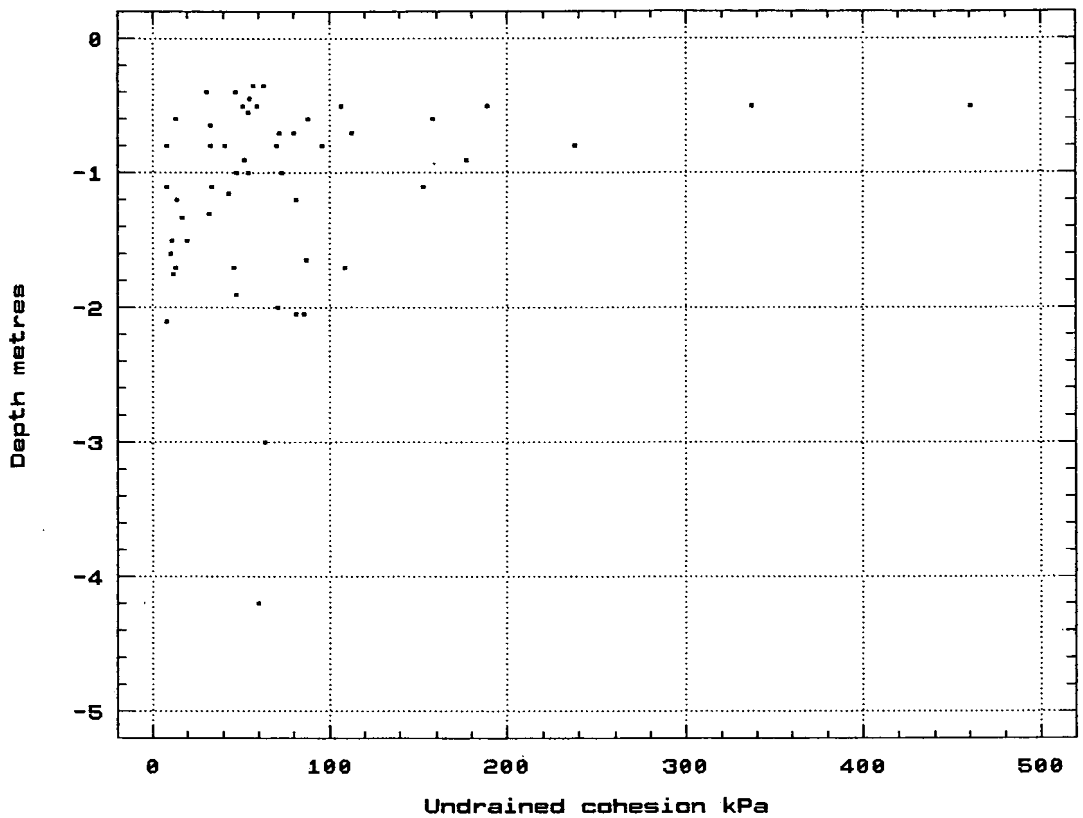


Fig. 15 MARINE DEPOSITS

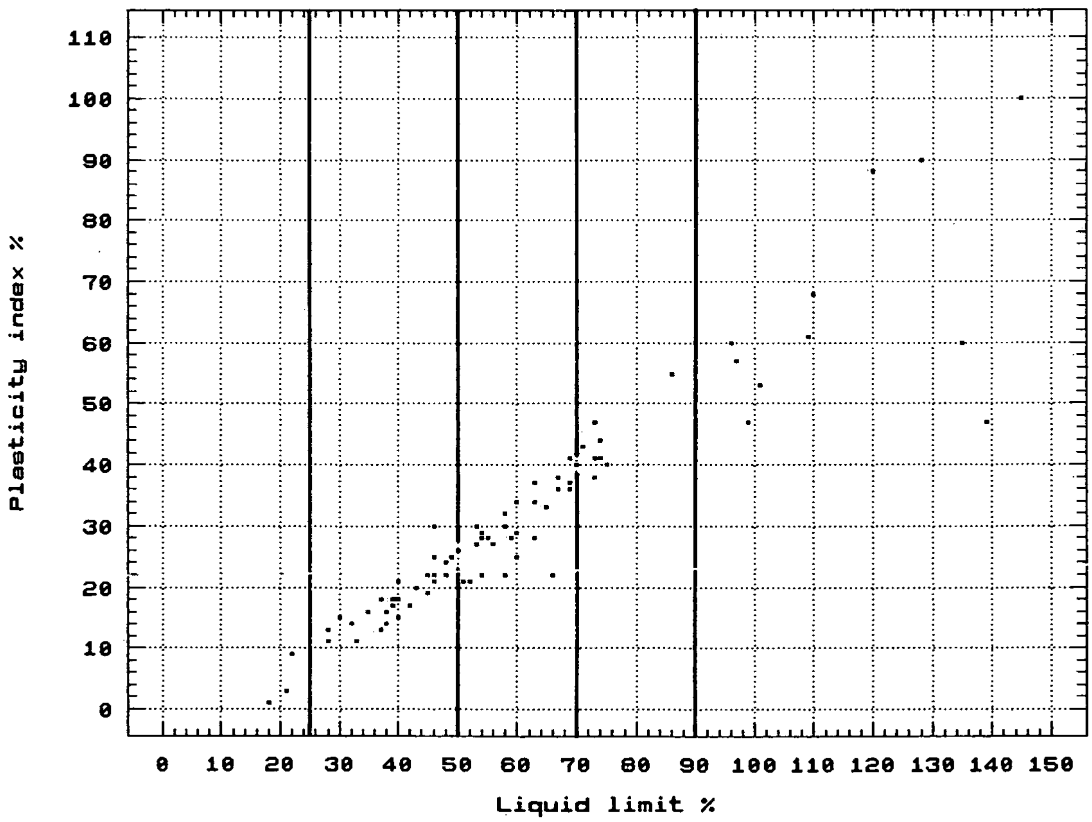


Fig. 16 MARINE DEPOSITS

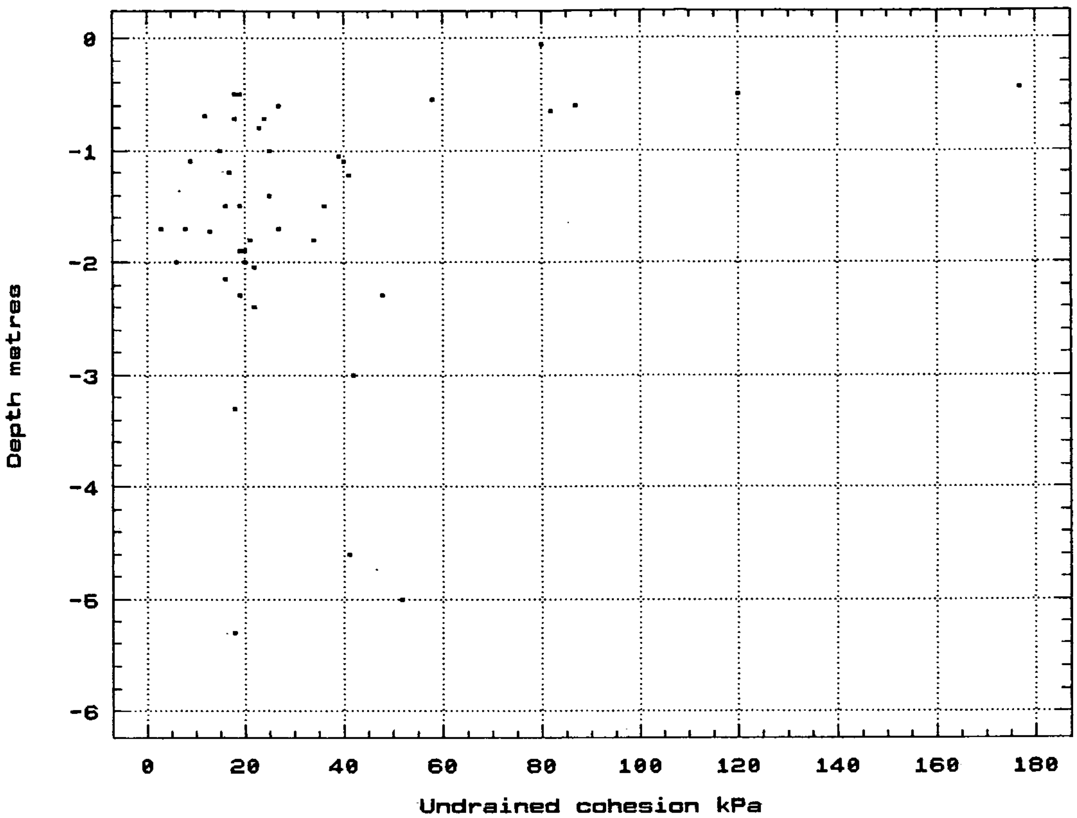
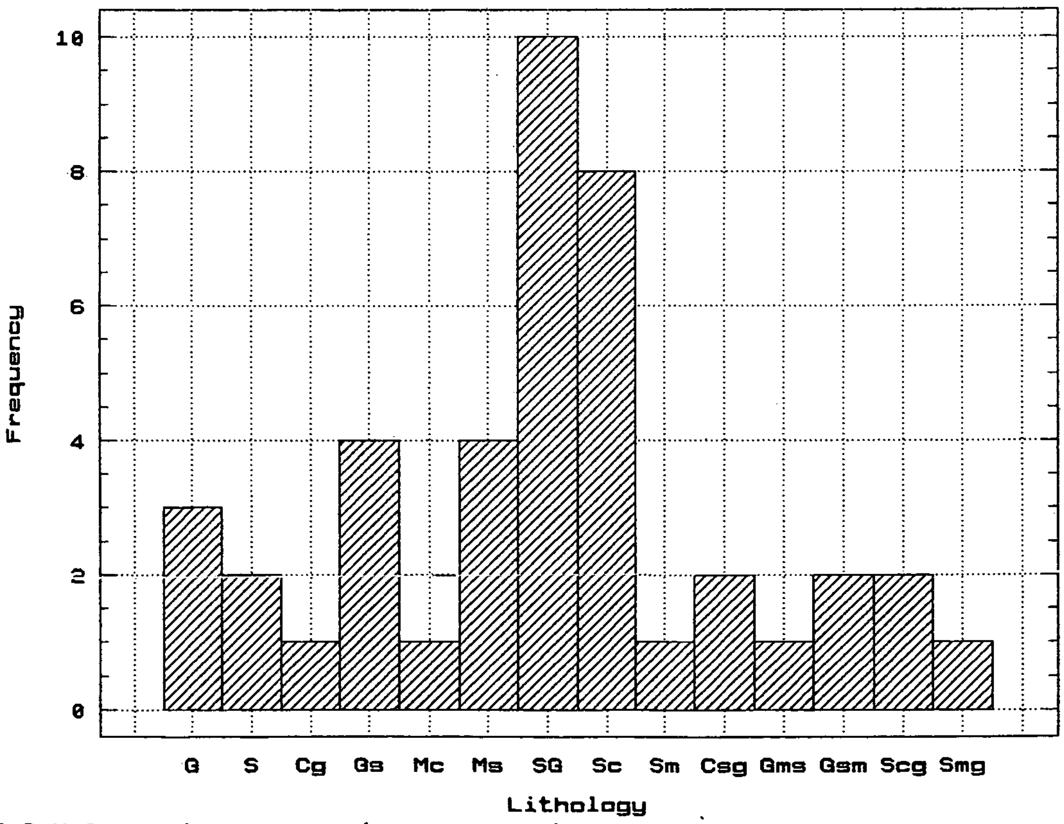
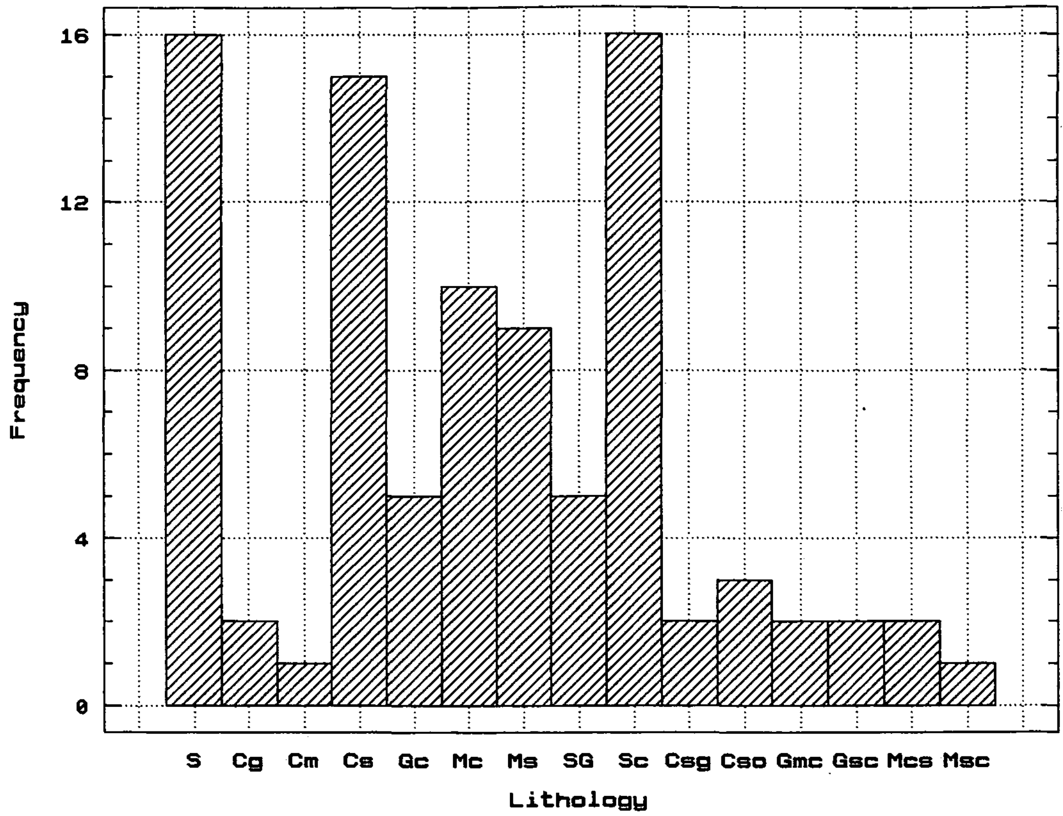


Fig. 17 RIVER TERRACE DEPOSITS
G = Gravel, S = Sand, M = Silt, C = Clay



G,S,M,C = major component g,s,m,c= minor

Fig. 18 VALLEY HEAD
G-Gravel S-Sand M-Silt C-Clay O-Organic*



* Uppercase major component, lower minor

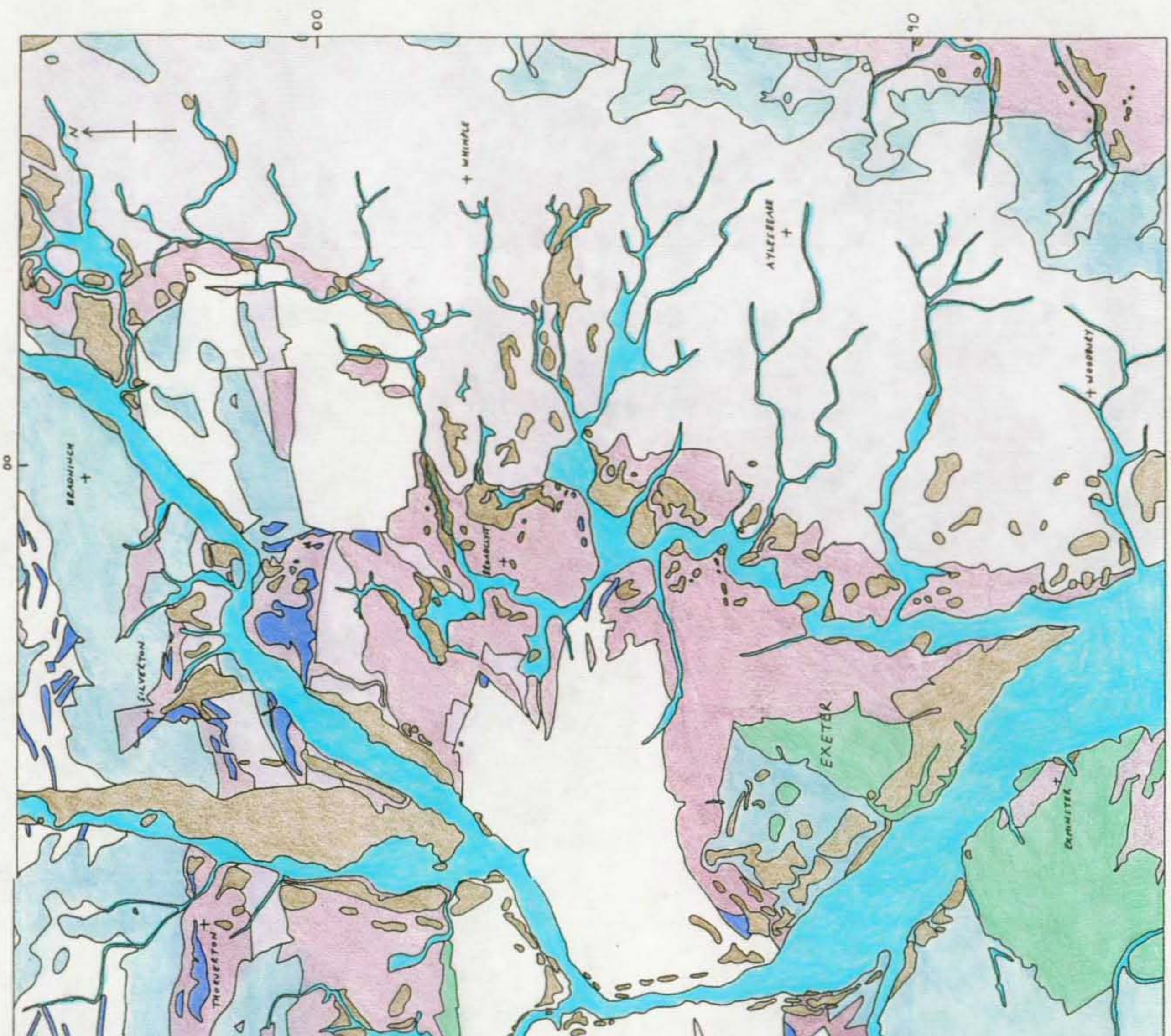


Figure 19. Engineering geological map of the Exeter district. The map is intended as a general guide only to the distribution of the engineering geology units, and it is recommended that for a more precise depiction of their distribution, the component 1:10,000-scale geological maps should be consulted.

Engineering classification			Lithostratigraphical unit	Foundations	Excavations	Slope stability	Weathering
STRONG ROCKS			GRANITE LAVAS DOLERITE AND METAMORPHOSED COMBE SHALE SANDSTONES IN BUDE FORMATION	Excellent bearing capacity on fresh rock, but granite, lavas and dolerite may be deeply weathered to a more compressible residual soil of lower bearing capacity. Chemical attack on buried concrete is unlikely.	Blasting, hydraulic or pneumatic tools needed in fresh rock. Ripping or digging in more weathered material. Fresh core-stones may remain in completely decomposed soil.	Shallow slips may occur in weathered material if undercut. Block slides and wedge failures in cut faces if discontinuities dip out of face. Rockfalls and/or topples possible on steep faces.	Deeply weathered to residual soil. Lavas may also have been chemically altered at time of formation.
INTERBEDDED STRONG ROCKS & MUDSTONE			TEIGN CHERT CRACKINGTON FORMATION (UNDIVIDED PART) BUDE FORMATION (EXCLUDING MAPPED SANDSTONES)	Generally good bearing capacity for fresh material. Possible differential settlement across sandstone/shale interfaces. Higher plasticity clays from weathered shale may give shrink/swell problems. Chemical attack on buried concrete is unlikely.	Blasting, hydraulic or pneumatic tools needed for thicker sandstones. Thinner sandstones interbedded with shale are rippable or diggable. Chert may cause high rate of wear on machinery.	Shallow slips where shale dominant, weathered material is undercut, or water table is high. Falls, block slides and wedge failures in cuttings in sandstone-dominant lithologies.	Mudstones soften to clay, and thinner sandstones are reduced to angular rubble.
MUDSTONES, SHALES AND CLAYS		Overconsolidated	HYNER SHALE & TRUSHAM SHALE (UNDIFFERENTIATED) COMBE SHALE ASHTON SHALE UPTON FORMATION YELLOWFORD FORMATION BUSSELL'S MEMBER POLTIMORE MUDSTONE AYLESBEARE MUDSTONE	Generally good bearing capacity if soft superficial material removed. Weathered material of high plasticity may give shrink/swell problems. Chemical attack on buried concrete is unlikely.	Excavations by digging in weathered material, ripping in fresher, harder mudstone. May heave in bottom of excavations due to stress relief.	Shallow slipping in weathered Ashton Shale.	May be deeply weathered and in a softened remoulded condition in the near-surface zone.
		Normally consolid.	ALLUVIUM (EXCLUDING GRAVELS) MARINE DEPOSITS	Low and possibly uneven bearing capacity, high and differential settlement. Possible chemical attack on buried concrete in Marine Deposits.	Easily dug, but may need dewatering and extra support to maintain stability. Running sands may be present.	No natural slope instability.	Surface desiccation may cause a higher strength layer in the near-surface zone.
GRANULAR ROCKS AND DEPOSITS	FINE-GRAINED DEPOSITS	Well cement.	KNOWLE SANDSTONE	Good bearing capacity. Chemical attack on buried concrete unlikely.	Diggable in weathered material; ripping required in fresh rock.	No known stability problems.	May be weathered to sand in near-surface zone.
		Weakly Cemented	HIGHER COMBEROY FORMATION THORVERTON SANDSTONE CREEDY PARK SANDSTONE SHUTE SANDSTONE WHIPTON FORMATION MONKERTON FORMATION DAWLISH SANDSTONE CLYST ST LAWRENCE FORMATION OTTER SANDSTONE	Generally good bearing capacity if near-surface weathered material is removed. Low, rapid settlement. Chemical attack on buried concrete unlikely. Fine silty sands may be frost susceptible.	Usually diggable, with ripping in the more cemented parts. Running sand conditions may occur below the water table.	No known natural instability. Cut slopes stable if protected from surface run-off, and if erosion by seepage from perched water tables is controlled.	May be deeply weathered to sand. Loose at the surface, becoming denser with increasing depth.
	COARSE-GRAINED DEPOSITS	Well Cemen.	NEWTON ST CYRES BRECCIA HEAVITREE BRECCIA	Very good bearing capacity. Chemical attack on buried concrete unlikely.	Rippable and diggable in weathered material. Strongest material may need blasting.	No natural instability known, Cut slopes stand well at high angles.	Moderately weathered near surface.
		Weakly Cemented	CADBURY BRECCIA BOW BRECCIA CREDITON BRECCIA ALPHINGTON BRECCIA BUDLEIGH SALTERTON PEBBLE BEDS	Generally good bearing capacity in fresh material. Weathered material may be uncemented and loose near the surface. Settlement low and rapid. Chemical attack on buried concrete unlikely. Silty lithologies may be frost susceptible.	Diggable in weathered and weakly cemented material. Rippable in fresh and better cemented material. Running sand may occur in the weathering zone below the water table.	No natural slope instability known. Cut slopes stable at steep angles if protected from surface runoff and if seepage from perched water tables is controlled.	May be deeply weathered to clayey silty soil with cobbles.
		Uncement-ed	ALLUVIAL GRAVEL RIVER TERRACE DEPOSITS HEAD (NOT SHOWN ON MAP)	Generally good bearing capacity. Settlement low and rapid. Chemical attack on buried concrete unlikely.	Diggable; may meet high water inflow below the water table.	No natural slope instability known, except relict slip surfaces may be present in head leading to potential slope instability.	Surface zone in loose condition in places.

TABLE 2 Lithostratigraphic Unit - CRACKINGTON FORMATION................................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT N	RQD %	FSI mm
75	...	...
140	...	...
57 220	... ...	... ...
23 260	... ...	... ...
9 300	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m3	DRY DENSITY Mg/m3	pH	SULPHATE class*
26	24	24	24	8	....	...	...
23	44	22	24	1.90	....	...	.
16 30	35 55	20 25	15 29	1.84 2.05	....	... ...	. .
15 35	34 70	20 28	13 46	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION CLAY %	SILT %	SAND %	GRAVEL %
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH Cu kPa	φu °
8	8
63	0
58 70	0 0
... ...	.. ..
... ...	.. ..

CONSOLIDATION Mv class*	Cv class*
...	...
.	.
. .	. .
. .	. .
. .	. .

* classes defined in appendix A.

TABLE 3 Lithostratigraphic Unit - CREEDY PARK SANDSTONE ....

Key to data presentation

	PARAMETER		
		N	No of samples
Percentile values	50%		Median value
	25%	75%	Quartile values
	10%	90%	
	2.5%	97.5%	

SPT		RQD		FSI	
N		%		mm	
	21		...		...
30		...		...	
19	40	...	...	...	...
11	61	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m^3		DRY DENSITY Mg/m^3		pH		SULPHATE class*	
	...		...		...		...		....		....		...		...
...		...		...		...		....		....		...		.	
...	...	...	...	...	...	...	...	....	....	....	....	...	...	.	.
...	...	...	...	...	...	...	...	....	....	....	....	...	...	.	.
...	...	...	...	...	...	...	...	....	....	....	....	...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	...		...		2		2
..		..		25		46	
..	..	..	..	24	25	42	49
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		ϕu °	
	...		...
...		..	
...	...	..	..
...	...	..	..
...	...	..	..

CONSOLIDATION			
Mv class*		Cv class*	
...		...	
.		.	
.	.	.	.
.	.	.	.
.	.	.	.

Ultimate Compressive strength MPa		Point load strength Axial MPa		Point load strength Diametral MPa	
	2				2
0.9				6.5	
0.8	1.0			6.0	7.0

* classes defined in appendix A.

TABLE 4 Lithostratigraphic Unit - CREDITON BRECCIA......................................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT N	RQD %	FSI mm
125	...	...
50	...	...
32 80	... ...	... ...
23 122	... ...	... ...
14 170	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
18	3	3	3	3	....	...	...
12	42	27	12	1.82	....	...	.
10 13	37 44	22 32	10 20	1.58 1.97	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	...	8	10
..	..	26	45
.. ..	.. ..	11 28	36 67
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	ϕu °
...	...
...	..
... ...	.. ..
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
...	...
.	.
. .	. .
. .	. .
. .	. .

Ultimate Compressive strength MPa	Point load strength Axial MPa	Point load strength Diametral MPa
30	17	13
4.0	5.4	1.5
1.4 9.7	2.8 10.0	0.5 2.4
0.6 15.5		

* classes defined in appendix A.

TABLE 5 Lithostratigraphic Unit - CREDITON BRECCIA (FORMERLY YENDACOTT BRECCIA)

Key to data presentation

	PARAMETER		
		N	No of samples
	50%		Median value
Percentile values	25%	75%	Quartile values
	10%	90%	
	2.5%	97.5%	

SPT		RQD		FSI	
N		%		mm	
	6		...		...
28		...		...	
15	36	...	...	...	...
...	...	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m³		DRY DENSITY Mg/m³	
	3		2		2		2		3		....
13		37		25		12		1.96		....	
12	14	35	38	25	25	10	13	1.77	1.98	....	....
...	...	...	...	...	...	...	...	....	....	....	....
...	...	...	...	...	...	...	...	....	....	....	....

pH		SULPHATE class*	
	...		...
...		.	
...	...	.	.
...	...	.	.
...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	...		...		...		...
..		..		..		..	
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		ϕu °	
	...		...
...		..	
...	...	..	..
...	...	..	..
...	...	..	..

CONSOLIDATION			
Mv class*		Cv class*	
...		...	
.		.	
.	.	.	.
.	.	.	.
.	.	.	.

* classes defined in appendix A.

TABLE 6 Lithostratigraphic Unit - NEWTON St CYRES BRECCIA...............................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT N	RQD %	FSI mm
14	...	...
39	...	...
21 49	... ...	... ...
... ...	... ...	... ...
14 170	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
...	...	...	...	...	....	2	2
..	..	..	..	....	....	7.7	1
.. ..	.. ..	.. ..	.. ..	....	....	7.0 8.3	1 1
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION				STRENGTH		CONSOLIDATION	
CLAY %	SILT %	SAND %	GRAVEL %	Cu kPa	ϕu °	Mv class*	Cv class*
...	...	4	4	...	...	...	...
..	..	36	39	...	..	.	.
.. ..	.. ..	20 42	25 69	... ...	.. ..	. .	. .
.. ..	.. ..	.. ..	.. ..	... ...	.. ..	. .	. .
.. ..	.. ..	.. ..	.. ..	... ...	.. ..	. .	. .

Ultimate Compressive strength MPa	Point load strength Axial MPa	Point load strength Diametral MPa
10	4	6
2.5	3.2	0.9
1.1 5.4	2.3 4.5	0.3 1.4

* classes defined in appendix A.

TABLE 7 Lithostratigraphic Unit - WHIPTON FORMATION....................................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
35	...	...
71	...	...
33 114	... ...	... ...
26 176	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
4	4	4	4	....	....	...	...
13	30	13	18	....	....	...	.
12 13	27 32	13 13	17 20	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	ϕu °
...	...
...	..
... ...	.. ..
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
...	...
.	.
. .	. .
. .	. .
. .	. .

* classes defined in appendix A.

TABLE 8 Lithostratigraphic Unit - HEAVITREE BRECCIA....................................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

	SPT N	RQD %	FSI mm
N	116	116	...
50%	200	45	...
25% 75%	92 240	13 70	... ...
10% 90%	27 408	0 80	... ...
2.5% 97.5%	15 625	0 90	... ...

	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
N	37	17	17	17	27	25	5	5
50%	12	26	17	9	2.13	1.91	7.4	1
25% 75%	10 15	24 27	14 19	4 12	1.86 2.36	1.62 2.03	7.0 8.0	1 1
10% 90%	8 18	16 34	12 21	1 17	1.52 2.47	1.35 2.16	... ...	. .
2.5% 97.5%	... ...	... ...	... ...	... ...	....	....	... ...	. .

	PARTICLE SIZE DISTRIBUTION CLAY %	SILT %	SAND %	GRAVEL %
N	27	27	35	35
50%	7	15	42	30
25% 75%	3 10	10 20	37 55	17 43
10% 90%	0 15	5 30	30 76	1 63
2.5% 97.5%	.. ..	.. ..	.. ..	.. ..

	STRENGTH Cu kPa	φu °
N	16	15
50%	14	10
25% 75%	6 70	2 14
10% 90%	... ...	.. ..
2.5% 97.5%	... ...	.. ..

	CONSOLIDATION Mv class*	Cv class*
N	...	...
50%	.	.
25% 75%	. .	. .
10% 90%	. .	. .
2.5% 97.5%	. .	. .

* classes defined in appendix A.

TABLE 9 Lithostratigraphic Unit - MONKERTON FORMATION.................................

Key to data presentation

	PARAMETER	
	N	No of samples
	50%	Median value
Percentile values	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
20	...	...
116	...	...
50 209	... ...	... ...
30 279	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX	BULK DENSITY	DRY DENSITY	pH	SULPHATE class*
%	%	%	%	Mg/m^3	Mg/m^3		
3	3	3	3	2	2	1	1
22	39	22	16	....	....	6.9	1
22 25	22 40	16 24	6 17	2.02 2.06	1.62 1.70	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY	SILT	SAND	GRAVEL
%	%	%	%
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu	ϕu
kPa	o
...	...
...	..
... ...	.. ..
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv	Cv
class*	class*
1	1
4	3
. .	. .
. .	. .
. .	. .

* classes defined in appendix A.

TABLE 10 Lithostratigraphic Unit - DAWLISH SANDSTONE FORMATION..........................

Key to data presentation

PARAMETER		
	N	No of samples
50%		Median value
25%	75%	Quartile values
10%	90%	Percentile values
2.5%	97.5%	Percentile values

SPT N		RQD %		FSI mm	
	549		...		...
136		...		...	
74	204	...	...	...	...
29	300	...	...	...	...
16	516	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m^3		DRY DENSITY Mg/m^3	
	173		30		27		26		31		14
12		36		19		22		2.04		1.74	
10	16	26	56	13	26	13	34	1.89	2.11	1.72	1.80
7	20	16	68	10	33	8	38	1.63	2.17	1.68	1.91
5	25	...	...	...	...	...	...	....	....	....	....

pH		SULPHATE class*	
	20		20
7.1		1	
6.6	7.2	1	1
6.0	7.5	1	1
...	...	.	.

PARTICLE SIZE DISTRIBUTION CLAY %		SILT %		SAND %		GRAVEL %	
	63		63		130		130
3		9		84		1	
1	10	7	16	78	88	1	1
0	19	4	20	61	92	1	10
0	26	0	43	27	94	1	25

STRENGTH Cu kPa		ϕu °	
	15		14
80		0	
33	135	0	13
13	151	0	30
...	...	..	..

CONSOLIDATION Mv class*		Cv class*	
	2		2
.		.	
3	4	3	3
.	.	.	.
.	.	.	.

* classes defined in appendix A.

TABLE 11 Lithostratigraphic Unit - AYLESBEARE MUDSTONE..................................

Key to data presentation

PARAMETER		
	N	No of samples
50%		Median value
25%	75%	Quartile values
10%	90%	Percentile values
2.5%	97.5%	Percentile values

SPT N		RQD %		FSI mm	
	10		...		...
86		...		...	
79	98	...	...	...	...
...	...	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m^3		DRY DENSITY Mg/m^3	
	215		214		214		214		158		158
16		39		22		17		2.14		1.84	
15	19	34	45	20	24	14	22	2.09	2.19	1.78	1.89
12	22	31	52	19	26	11	28	2.05	2.24	1.70	1.96
10	26	27	63	16	30	8	34	2.00	2.26	1.62	2.01

pH		SULPHATE class*	
	16		16
6.3		1	
5.8	7.0	1	1
5.4	7.5	1	1
...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	4		4		4		4
14		42		41		0	
5	29	24	55	16	71	0	0
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		ϕu °	
	98		97
126		11	
85	170	6	19
47	219	0	25
8	323	0	32

CONSOLIDATION			
Mv class*		Cv class*	
	13		13
2		2	
1	3	2	3
.	.	.	.
.	.	.	.

* classes defined in appendix A.

TABLE 12 Lithostratigraphic Unit - BUDLEIGH SALTERTON PEBBLE BEDS........................

Key to data presentation

PARAMETER		
	N	No of samples
50%		Median value
25%	75%	Quartile values
10%	90%	Percentile values
2.5%	97.5%	Percentile values

SPT		RQD		FSI	
N		%		mm	
	35		...		...
44		...		...	
32	66	...	...	...	...
21	100	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m³		DRY DENSITY Mg/m³	
	9		5		5		5		....		....
9		35		20		17		....		....	
64	14	31	37	17	22	14	18	....	....	....	....
...	...	...	...	...	...	...	...	....	....	....	....
...	...	...	...	...	...	...	...	....	....	....	....

pH		SULPHATE class*	
	...		...
...		.	
...	...	.	.
...	...	.	.
...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	4		4		8		8
8		15		33		49	
2	17	8	15	25	40	41	59
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		φu °	
	...		...
...		..	
...	...	..	..
...	...	..	..
...	...	..	..

CONSOLIDATION			
Mv class*		Cv class*	
	...		...
.		.	
.	.	.	.
.	.	.	.
.	.	.	.

* classes defined in appendix A.

TABLE 13 Lithostratigraphic Unit - EXETER VOLCANIC ROCKS..........................

Key to data presentation

	PARAMETER	
	N	No of samples
	50%	Median value
Percentile values	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
4	1	...
50	45	...
36 54	... ...	... ...
... ...	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
2	2	2	2	2	....	...	..
..	...	...	...	....	....	...	.
15 16	40 44	16 22	22 24	2.12 2.23	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	ϕu ^{o}
2	2
...	..
212 223	0 0
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
...	...
.	.
. .	. .
. .	. .
. .	. .

Point Load MPa	Unconfined Compressive Strength MPa
18	18
1.53	37
1.31 1.88	31 45
1.20 2.15	29 51
....	... ...

* classes defined in appendix A.

TABLE 14 Lithostratigraphic Unit - BLANKET HEAD...

Key to data presentation

PARAMETER		
	N	No of samples
50%		Median value
25%	75%	Quartile values
10%	90%	Percentile values
2.5%	97.5%	Percentile values

SPT N	RQD %	FSI mm
3	...	...
12	...	...
1 51	... ...	... ...
... ...	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
4	...	...	...	3	....	...	...
16	...	...	...	2.00	....	...	.
14 21	... ...	... ...	... ...	1.97 2.09	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH		CONSOLIDATION	
Cu kPa	φu °	Mv class*	Cv class*
3	3	...	...
60	0	.	.
51 63	0 0	. .	. .
... ...	.. ..	. .	. .
... ...	.. ..	. .	. .

* classes defined in appendix A.

TABLE 15 Lithostratigraphic Unit - VALLEY HEAD..

Key to data presentation

	PARAMETER		
		N	No of samples
Percentile values	50%		Median value
	25%	75%	Quartile values
	10%	90%	
	2.5%	97.5%	

SPT N		RQD %		FSI mm	
	39		...		...
21		...		...	
13	40	...	...	...	...
9	68	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m^3		DRY DENSITY Mg/m^3		pH		SULPHATE class*	
	54		36		35		35		22		20		8		9
14		34		18		17		2.10		1.87		6.4		1	
11	18	22	43	16	21	8	23	1.95	2.16	1.74	1.94	5.9	6.8	1	1
8	21	20	53	13	22	5	33	1.76	2.22	1.63	1.99	...	...	.	.
...	...	...	...	...	...	...	...	....	....	....	....	...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	10		20		20		20
6		12		60		18	
5	8	7	22	36	77	1	44
2	11	1	29	23	84	1	68
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		φu °	
	19		19
72		7	
24	175	0	12
15	210	0	21
...	...	..	..

CONSOLIDATION			
Mv class*		Cv class*	
	7		7
2		2	
2	3	2	3
1	3	2	4
.	.	.	.

* classes defined in appendix A.

TABLE 16 Lithostratigraphic Unit - OLDER HEAD..

Key to data presentation

	PARAMETER		
		N	No of samples
	50%		Median value
Percentile values	25%	75%	Quartile values
	10%	90%	
	2.5%	97.5%	

SPT		RQD		FSI	
N		%		mm	
	...		...		...
...		...		...	
...	...	...	...	...	...
...	...	...	...	...	...
...	...	...	...	...	...

MOISTURE CONTENT %		LIQUID LIMIT %		PLASTIC LIMIT %		PLASTIC INDEX %		BULK DENSITY Mg/m^3		DRY DENSITY Mg/m^3	
	9		7		7		7		6		6
18		42		23		21		2.11		1.76	
14	21	34	52	16	23	18	28	2.09	2.12	1.71	1.78
...	...	...	...	...	...	...	...	....	....	....	....
...	...	...	...	...	...	...	...	....	....	....	....

pH		SULPHATE class*	
	...		...
...		.	
...	...	.	.
...	...	.	.
...	...	.	.

PARTICLE SIZE DISTRIBUTION							
CLAY %		SILT %		SAND %		GRAVEL %	
	3		3		3		3
15		22		32		28	
6	24	14	25	24	56	24	30
..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..

STRENGTH			
Cu kPa		ϕu °	
	3		3
149		0	
115	285	0	10
...	...	..	..
...	...	..	..

CONSOLIDATION			
Mv class*		Cv class*	
	2		2
.		.	
1	2	2	2
.	.	.	.
.	.	.	.

* classes defined in appendix A.

TABLE 17 Lithostratigraphic Unit - RIVER TERRACE DEPOSITS................................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
24	...	...
34	...	...
24 92	... ...	... ...
18 100	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
21	9	9	9	6	6	3	4
12	29	15	15	2.12	1.81	7.0	1
11 14	28 33	14 18	13 17	2.10 2.22	1.70 1.95	6.8 7.3	1 2
7 21	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
10	10	12	12
3	7	37	49
0 6	0 9	16 78	10 74
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	φu °
5	4
50	14
20 50	9 18
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
2	2
.	.
1 3	2 3
. .	. .
. .	. .

* classes defined in appendix A.

TABLE 18 Lithostratigraphic Unit - ALLUVIUM...

Key to data presentation

	PARAMETER	
	N	No of samples
	50%	Median value
Percentile values	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT N	RQD %	FSI mm
21	...	...
23	...	...
16 27	... ...	... ...
12 32	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3
112	105	104	104	59	42
25	42	19	23	1.95	1.61
21 30	33 54	16 23	16 33	1.87 2.01	1.50 1.72
16 40	25 66	14 26	9 43	1.74 2.14	1.28 1.82
13 66	19 85	13 35	6 56	1.56 2.21	....

pH	SULPHATE class*
12	13
6.4	1
6.1 6.7	1 1
... ...	. .
... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
6	10	10	10
12	22	54	3
7 16	13 32	39 76	1 42
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	ϕu °
53	53
55	0
33 86	0 0
12 158	0 7
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
34	34
3	3
3 4	2 3
2 4	2 3
. .	. .

* classes defined in appendix A.

TABLE 19 Lithostratigraphic Unit - SUB-ALLUVIAL GRAVEL........................

Key to data presentation

	PARAMETER	
	N	No of samples
Percentile values	50%	Median value
	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
136	...	...
33	...	...
25 52	... ...	... ...
20 112	... ...	... ...
15 162	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
8	1	1	1	....	....	2	2
9	57	20	37	....	....	...	.
8 12	... ...	... ...	... ...	....	....	6.1 6.5	1 1
... ...	... ...	... ...	... ...	....	....	... ...	. .
... ...	... ...	... ...	... ...	....	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	29	30	30
..	1	12	85
.. ..	0 4	7 19	79 93
.. ..	0 12	4 44	52 96
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	ϕu °
...	...
...	..
... ...	.. ..
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
...	...
.	.
. .	. .
. .	. .
. .	. .

* classes defined in appendix A.

TABLE 20 Lithostratigraphic Unit - MARINE DEPOSITS.......................................

Key to data presentation

	PARAMETER	
	N	No of samples
	50%	Median value
Percentile values	25% 75%	Quartile values
	10% 90%	
	2.5% 97.5%	

SPT	RQD	FSI
N	%	mm
8	...	...
10	...	...
5 14	... ...	... ...
... ...	... ...	... ...
... ...	... ...	... ...

MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTIC INDEX %	BULK DENSITY Mg/m^3	DRY DENSITY Mg/m^3	pH	SULPHATE class*
102	86	86	86	60	18	5	5
36	54	26	28	1.82	1.37	6.9	1
29 54	40 70	23 32	18 40	1.60 1.95	1.18 1.44	6.7 7.1	1 1
25 79	32 101	18 42	3 57	1.49 2.00	0.85 1.54	... ...	. .
20 147	22 135	15 52	6 88	1.25 2.06	....	... ...	. .

PARTICLE SIZE DISTRIBUTION			
CLAY %	SILT %	SAND %	GRAVEL %
...	...	...	...
..	..	..	..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..
.. ..	.. ..	.. ..	.. ..

STRENGTH	
Cu kPa	φu °
45	3
22	2
18 40	1 5
... ...	.. ..
... ...	.. ..

CONSOLIDATION	
Mv class*	Cv class*
18	18
4	3
3 4	2 3
3 4	2 3
. .	. .

* classes defined in appendix A.

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APPENDIX A

TABLES DEFINING GEOTECHNICAL PARAMETER CLASSES

COEFFICIENT Of VOLUME COMPRESSIBILITY M_v

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	>1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays e.g. estuarine clays.
3	Medium	0.1 - 0.3	Fluvio-glacial clays, lake clays.
2	Low	0.05 - 0.1	Tills (Boulder clay).
1	Very low	<0.05	Heavily overconsolidated Tills, stiff weathered rocks.

After Head (1982)

COEFFICIENT OF CONSOLIDATION C_v

Class	C_v M^2/year	Plasticity Index Range	Soil Type
1	<0.1		CLAYS: Montmorillonite
2	0.1 - 1	>25	High plasticity
3	1 - 10	25 - 15	Medium plasticity
4	10 - 100	<15	Low plasticity
5	>100		SILTS

After Lambe and Whitman (1979)

SULPHATES IN SOILS AND GROUNDWATERS

Class	Total SO^3 %	SO_3 in soil 2:1 water extract grams/litre	SO_3 in groundwater parts in 100 000
1	<0.2		<30
2	0.2 - 0.5		30 - 120
3	0.5 - 1.0	1.9 - 3.1	120 - 250
4	1.0 - 2.0	3.1 - 5.6	250 - 500
5	>2.0	>5.6	>500

After Anon (1981a)

APPENDIX B

GEOTECHNICAL TESTS QUOTED IN THE DATABASE AND THEIR APPLICATIONS

FIELD TESTS

The Standard Penetration Test (SPT)

The standard penetration test (SPT) is a dynamic test carried out at intervals during the drilling of a borehole. A standard 50 mm diameter split barrel sampler is driven into the soil at the bottom of the hole for a distance of 450 mm by the blows of a standard weight (65 kg), falling through a standard distance (0.76 m). The number of blows (N) required to drive the last 300 mm is recorded. [Test details are given in Anon. (1981b)].

A modification of the test for hard material and coarse gravel uses a solid cone instead of a cutting shoe and is called a cone penetration test (CPT).

Although this is a field test which is subject to operational errors, the SPT is widely used to give an indication of the relative density of granular soils (very loose to very dense) and the consistency of cohesive soils (very soft to hard). Correlations have also been made between SPT and the bearing capacity of a soil.

The results of the SPT are meaningful up to and including an N-value of 50, corresponding to very dense granular soils and hard cohesive soils. The SPT is also frequently used in harder materials, ie. rocks and heavily overconsolidated soils or "mudrocks", in which case the test is normally terminated before the shoe has been driven the full 300 mm. Rather than extrapolate the number of blows to represent the full 300 mm of test, the amount of penetration in mm for 50 blows can be quoted. When the results are given in this manner the test is referred to as the Rock Penetration Test (RPT). The relationship between the two methods of quoting the results is tabulated below:

Relative Density/Consistency	SPT	RPT
Very loose/very soft	0 - 5	
Loose/soft	5 - 10	
Medium dense/firm	10 - 30	
Dense/stiff	30 - 50	
Very dense/hard	50	30
Rock/Heavily overconsolidated soils		200 - < 100

In the Exeter database where partial penetration has been achieved the "N" value has been calculated as if for full penetration to give an "extrapolated" N value.

Shear Vane Tests

Shear strength parameters obtained from laboratory tests are prone to errors resulting from sample disturbance, giving results that are frequently lower than those obtained from back-analysis. In order to address this problem the shear vane test was developed to determine the undrained shear strength of clays within the very soft to firm consistency range, on in situ samples.

The apparatus comprises four thin rectangular blades arranged radially around a circular shaft to form a cross. Advancing this vane into undisturbed soil and rotating at a constant rate of shear using a torque head, allows the torque causing soil failure along the surface of the volume of revolution of the vane to be measured. Undrained shear strength is calculated from the relationship between torque and angular rotation, the vane dimensions and the maximum torque applied. Remoulded shear strength is measured similarly, after remoulding the soil by rapid rotation of the vane and allowing a short time period for pore pressures to dissipate.

The results of the test are intended to give a rapid indication of undrained shear strength and may be used in stability analyses. However, the results obtained should not be generally considered as a replacement for the more accurate shear strength parameters determined from triaxial or shearbox tests. Further details of the test can be found in Carter (1983).

Rock Quality Designation (RQD)

Rock quality designation (RQD) was introduced by Deere (1964) to give an indication of rock quality in relation to the degree of fracturing from drill cores. It is defined as the sum of the core sticks in excess of 100 mm in length expressed as a percentage of the total length of core drilled. The parameter takes no account of the degree of fracture opening or the fracture condition and does not distinguish between fracture spacings of more than 100 mm RQD has been used with uniaxial compressive strength to give an indication of excavatability and as one input for the classification of rock masses to assist in the design of tunnel support systems (Bieniawski 1974, Barton et al 1974).

Point Load Test

The point load test provides a rapid, economical and reasonably accurate index of rock strength and strength anisotropy, the principles of which have been described by Broch and Franklin (1972) and Bieniawski (1975). The test basically involves loading a sample of rock to failure between two standard-shaped steel cones mounted in a portable hydraulically operated press. The load at failure is determined from appropriately calibrated pressure gauges and the Point Load Strength (Is) calculated from the relationship: $Is = P/D^2$, where D is the initial distance between the loading cones holding the sample; and P is the maximum pressure recorded on the gauge (ie. the failure load).

The test is usually carried out on pieces of rock core placed both diametrically and axially between the loading cones to obtain an indication of strength anisotropy. For diametral tests the length:diameter ratio (L/D) of the core should be > 1.4; for axial tests the diameter : length ratio (D/L) should be 1.1 (where D is the axial distance between the cones). The test may also be performed, less accurately, on irregular lumps with D/L ratios of 1.0 - 1.4. It is usual practice to correct the point load strength (Is) to a standard 50 mm reference diameter using a set of standard curves to give the Point Load Index, Is(50). This index is essentially equivalent to the point load strength multiplied by correction factors for shape and size. The Anisotropy Index (Ia) of the rock may be expressed as:

$$Ia = \frac{Is(50)\ [\text{diametral}]}{Is(50)\ [\text{axial}]}$$

LABORATORY TESTS

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Moisture Content

The moisture content of a soil sample is defined as the ratio of the weight of water in the sample to the weight of solids, normally expressed as a percentage, ie:

$$\text{Moisture content, m} = \frac{\text{weight of water}}{\text{weight of solids}}$$

[Standard test procedure is given in Anon. (1975)]. Moisture content is a basic soil property and influences soil behaviour with regard to compaction, plasticity, consolidation and shear strength characteristics.

Atterberg or Consistency Limits (Plasticity Tests)

As moisture is removed from a fine-grained soil it passes through a series of states, i.e. liquid, plastic, semi-solid and solid. The moisture contents of a soil at the points where it passes from one stage to the next are known as 'consistency limits'. These limits are defined as:

Liquid Limit (LL) The minimum moisture content at which the soil will flow under its own weight.

Plastic Limit (PL) The minimum moisture content at which the soil can be rolled into a thread 3 mm diameter without breaking up.

Shrinkage limit The maximum moisture content at which further loss of moisture does not cause a decrease in the volume of the soil.

The range of moisture content over which the soil is plastic is known as the **plasticity index (Ip)**, and is defined as:

$$Ip = LL - PL$$

[Test procedures are given in Anon (1975)]

The factors which control the behaviour of the soil with regard to consistency are the nature of the clay minerals present, their relative proportions, and the amount and proportions of silt, fine sand and organic material. A soil may be classified in terms of its plastic behaviour by plotting plasticity index against liquid limit on a standard plasticity (or Casagrande) chart. The consistency limits also give an indication of soil strength and compressibility.

Density

Density of a soil, ie the mass per unit volume, may be measured in various ways.

The total or **bulk density** is the mass of the entire soil element (solids + water) divided by the volume of the entire element.

The **dry density** is the mass of dry solids divided by the volume of the entire soil element.

The **saturated density** is the mass of the entire soil element with its pore spaces filled with water (i.e. totally saturated) divided by the volume of the entire soil element.

Density measurements are simple if an undisturbed specimen of known, or easily measured, volume is obtained. If this is not possible in the field, the sand replacement method is used to determine the volume of a hole from which the soil sample is excavated by filling with a measured quantity of dry, uniformly graded sand of known density. Density measurements are usually expressed as Mg/m^3 and full test details are given in Anon (1975).

Soil density measurements may be used to assess various earth loads such as soil mass, overburden pressure, surcharge pressure and earth pressure on retaining walls.

Particle Size Analysis

The particle size distribution of a soil is determined by sieving and sedimentation. A sample of soil is dried, weighed and sieved to remove the fraction greater than 20 mm in size. It is then immersed in water with a dispersing agent such as sodium hexametaphosphate to break up soil aggregates. The sample is then wet sieved to remove particles less than 63 μm. The fraction retained on the 63 μm sieve is dried and passed through a nest of sieves of mesh size ranging from 20 mm to 63 μm. The fraction retained on each sieve is weighed and the cumulative percentage passing each sieve is calculated. A grading curve of percentage passing against sieve size is plotted.

The fines which passed through the 63 μm sieve are graded by sedimentation. A representative sub-sample is made up into a suspension with distilled water, placed in a tall jar and made up to a volume of 500 ml. It is then agitated vigorously and allowed to settle. Samples are removed by pipette from a given depth at specific times, dried and the contained solids weighed or, alternatively, hydrometer readings of the soil-water suspension are recorded at specific time intervals. The size distribution can then be calculated using Stokes' Law which relates settling time to particle size. The entire grading curve for coarse and fine material can then be plotted. Full details are given in Anon. (1975).

Particle size distribution is used for classifying soil in engineering terms (Anon 1981b). Particle size distribution curves will give an indication of soil behaviour with regard to permeability, susceptibility to frost heave or liquefaction, and will give some indication of strength properties. Particle size analysis does not, however, indicate structure and will not distinguish between a sandy clay and a laminated sand and clay which may behave very differently in situ, but may show similar particle size distribution in a bulk test sample.

CHEMICAL TESTS

pH

About 30 g of soil are weighed and placed in 75 ml of distilled water in a beaker. The mixture is stirred and allowed to infuse overnight. A glass electrode connected to a pH meter is then placed in the stirred mixture and the pH reading taken. The electrode and meter may also be used to determine the pH of groundwater samples; pH may also be determined colorimetrically. Details are given in Anon (1975).

The pH of soil or groundwater is important when designing concrete structures below ground surface. Ordinary Portland cement is not recommended in situations with a pH below 6, high alumina cement can be used down to pH 4 and super-sulphated cement has been used to pH 3.5. Acidic groundwaters can also cause corrosion in buried iron pipes.

Sulphate

The sulphate content of soil is determined by leaching a weighed sample of soil with hydrochloric acid and precipitating the dissolved sulphate by the addition of an excess of barium chloride. The precipitate is then filtered, ignited in a furnace and weighed.

The sulphate content of groundwater or an aqueous soil extract is determined by passing the water through a column of strongly-acidic cationic exchange resin activated with hydrochloric acid. The groundwater or soil-water washings are collected and titrated against standardised sodium hydroxide solution, using a suitable indicator. From the amount of sodium hydroxide used during titration the quantity of dissolved sulphates can be determined and expressed in terms of SO_3 content, as grams per litre or as parts per 1000 000. Full test details are given in Anon (1975).

It is important that the sulphate content of groundwater and soil is known as ordinary Portland cement deteriorates in the presence of sulphate. Knowledge of sulphate concentrations enables a suitable sulphate resisting or high alumina cement to be used in appropriate concrete mixes for applications below ground level.

Organic Content

A small (0.2 - 5 g) dry, representative sample of soil is weighed and reacted with 10 ml of normal potassium dichromate solution. The potassium dichromate which is left after the organic material has been oxidised is then determined by titration against a standard solution of ferrous sulphate. The organic content of the soil can then be calculated. Details are given in Anon (1975).

Organic silts and clays are weak soils and usually highly compressible. In extremely organic soils, such as peat, very high and rapid compressibility is found and permeability is high. Although weak and permeable, density is low and cuttings in peat may therefore stand better than a clay soil of the same height and slope angle.

STRENGTH TESTS

Triaxial Compression Test

The triaxial compression test is the most widely used test for determining the shear strength of cohesive soils and a number of different methods may be used depending on the application of the results.

In general terms, an undisturbed cylindrical specimen (usually 76 mm x 38 mm) is placed between rigid end caps and covered with a rubber membrane. The assembly is then placed in a triaxial cell which is filled with water, taking care that all air is removed. The confining water pressure in the cell is then maintained at a prescribed constant value while the axial load on the specimen is increased at a constant rate of strain. The test continues until the specimen shears or a maximum vertical stress is reached. Vertical displacement and axial load on the sample are measured during the test. The test is repeated on two further specimens (from the same sampling point) at different confining pressures. From the results obtained from the three tests, a standard graphical construction (based on the Mohr-Coulomb failure criterion) enables the measured principal stresses to be plotted so that the shear strength of the soil can be determined in terms of its cohesive and frictional components (ie. cohesion, C, and angle of internal friction, ϕ).

The test may be carried out with the sample either drained or undrained (with or without pore pressure measurement), and the type of test will depend upon the site conditions and type of engineering works being undertaken.

An **unconsolidated-undrained (UU) test** is used for foundations on normally consolidated clay soils (where drainage would be slow). The test normally takes only a few minutes, as pore pressures are not allowed to dissipate, and is thus often known as a **quick-undrained (QU)test**. The strength parameters determined in this test are the total or apparent undrained cohesion and friction values (Cu and ϕu, respectively).

In a **consolidated-undrained (CU) test**, free drainage of the specimen is allowed under the cell pressure for 24 hours before testing (that is, the sample consolidates). The drainage valve is then closed and the load increased rapidly to failure. This test is applicable to situations where a sudden change in load takes place after a period of stable conditions (eg. as a result of rapid draw down of water behind an earth dam).

A consolidated-undrained test with pore pressure measurement may also be carried out. In this test, the measurement of pore pressure enables calculation of the effective strength parameters, C'and ϕ' (sometimes referred to as the "true" cohesion and "true" angle of internal friction), in addition to the undrained parameters, Cu and ϕu.

A **drained (CD) test** is suitable for sandy soils or for clay embankments in which drainage blankets have been laid. Free drainage of the sample is allowed during both the consolidation and loading stages of the test, with the sample loading applied at a rate slow enough to allow dissipation of pore pressures. The test conditions enable the determination of the effective strength parameters, C',ϕ'.

Unconfined Compression Test

This test measures the unconfined (uniaxial) compressive strength of rock samples of regular geometry and is mainly intended for strength classification and characterisation of intact rock.

Test specimens are required to be right circular cylinders having a height:diameter ratio of 2.5-3.0 and a diameter preferably of not less than 54 mm. The sides and ends of the specimen should be smooth and end surface treatment other than machining is not permitted. Samples should be stored and tested in such a way that the natural water content is preserved. If other moisture conditions are used, they should be reported with the test results.

The prepared rock cylinders are placed in a suitable load frame and the load applied continuously at a constant stress rate such that failure will occur within 5 - 10 minutes of loading, alternatively the stress rate should be within the limits of 0.5 - 1.0 MPa/sec. The specimen is loaded to failure and the maximum load recorded. The number of specimens tested should be determined from practical considerations, but at least 5 are preferred. The uniaxial compressive strength is calculated by dividing the maximum load carried by the specimen during the test, by the original cross-sectional area.

CONSOLIDATION and COMPACTION TESTS

Consolidation Test

If a saturated cohesive soil is subjected to an increase in loading the pressure of the water in the pore spaces will increase by the same amount as the applied stress. The water will therefore tend to flow towards areas of lower pressure at a rate controlled by the soil permeability. The removal of water causes a decrease in volume of the soil, a process known as consolidation.

The consolidation parameters are measured in the laboratory by placing a disc of soil confined in a metal ring, in a water filled cell. A constant normal load is applied to the disc and its decrease in thickness measured with time. When it reaches a constant thickness for a given load, the load is increased (usually doubled) and the readings repeated. The loading is continued depending on the soil type and the structure for which the data is required. The coefficient of volume compressibility, Mv (m^2/MN), can then be calculated. This is a measure of the amount of volume decrease that will take place for a given increase in stress. The coefficient of consolidation, Cv (m^2/year) is also calculated, and is a measure of the rate at which the volume change will take place for a given increase in stress. The coefficient of permeability (k) also can be calculated from results obtained in the consolidation test where:

$$k = Cv \times Mv \times 0.31 \times 10^{-9} \text{ m/s}$$

Consolidation test results are important for foundation design and calculating the likely settlements that will take place during and after construction. The test results also enable the planning of phased construction stages to allow full consolidation settlement (dissipation of pore pressures) to take place prior to successive load stages.

Compaction

The compaction test determines the moisture content (the "optimum") at which a soil may be compacted to its maximum dry density. A quantity of soil (5 kg) is compacted in a standard mould using a standard rammer (2.5 or 4.5 kg) which is dropped from a standard height (300 mm or 450 mm) a standard number of times

(27). The density of the compacted soil is then measured and its moisture content determined. The procedure is then repeated using the same soil at different moisture contents.

The dry density of the compacted soil is plotted against its moisture content and the moisture content at which maximum compacted density may be achieved is read from the curve. [Details are given in Anon (1975)].

The results of the compaction test are used to determine the optimum moisture conditions at which to place a given soil as general or embankment fill.