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THE ENGINEERING GEOLOGY OF THE DEESIDE AREA

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INTRODUCTION

This report represents the engineering geological contribution to the British Geological Survey report on the thematic geological mapping of the Deeside area of North Wales for the Department of the Environment on behalf of the Welsh Office (Campbell and Hains 1988).

The project area covers approximately 265 sq km of ground south west of the Dee Estuary in the county of Clwyd (including part of the estuary) and encompasses the towns of Holywell and Bagillt in the north, Flint, Connah's Quay, and Mold and Buckley in the south.

The land rises from sea level along the Dee Estuary to over 400 m in the Clwyd Hills to the west. The local geology ranges in age from the Silurian to the Triassic with a cover of Pleistocene and Holocene deposits, but consists primarily of Coal Measures strata beneath glacial deposits.

The engineering geological part of the project was largely a desk study using data contained in site investigation reports collected from private companies, local authorities and other public bodies. The geotechnical data were extracted from the reports to produce a micro-computer based database, and analysed statistically to determine summary geotechnical values for various parameters for each litho-stratigraphic unit. These summaries are contained in Tables 1 to 18. The mapped units were divided or combined depending upon their geotechnical properties and, for the shales and mudstones, the properties of the weathered materials were summarised.

The results are discussed, together with their engineering implications, in this report and presented in a series of tables and diagrams. For the full discussion of the geology and the complementary thematic geology maps, reference should be made to the full project report where this contribution also appears in a modified form.

THE GEOTECHNICAL DATABASE

The classification of the geological formations into groups or units of similar engineering properties was carried out using geotechnical data extracted from site investigation reports on sites within the area of the project. As much of this area is rural, the coverage of these reports is limited. In general, the reports are of two types in terms of their areal coverage; firstly, they relate to development and other sites in and around current urban areas (for example, Mold, Connah's Quay, Flint, Bagillt and Holywell); and secondly, they relate to road developments (particularly the A55, but including the A550/491 from Shotwick to Ewloe and by-passes to Mold and Penyffordd). This information was supplemented by a very limited amount of information from publications.

The geotechnical information used in this study to classify geological formations in engineering terms does not necessarily represent these formations totally. For some formations (for example, the mudstones of the Elwy Group) no geotechnical data are available. For others (for example, the Kinnerton Sandstone Formation) data are limited to those from a handful of boreholes. For most of the formations, geotechnical information is only available for part of their outcrop. Consequently, summary values of geotechnical parameters given for the various formations should be considered as a general guide only and should not be used in detailed foundation design calculations.

Geotechnical test data were obtained for 4273 individual samples [a single Standard Penetration Test (SPT) counting as one sample] and these data were contained in 68 individual reports. Whilst this geotechnical data set is relatively small compared with some other areas of similar size, considerable time was required to extract and analyse the data. To try to determine whether it was necessary to extract all geotechnical data to obtain an understanding of the geotechnical properties of the different formations, the site investigation reports were divided into four groups in terms of the breadth and depth of factual information contained and any accompanying interpretative reports. In addition, a subjective assessment of the quality of the data in each report was attempted. The nature of reports ranged from those that contained a wealth of geotechnical data, apparently of good quality, detailed engineering geological logs and accompanying interpretative reports

(Category One) to those that contained no geotechnical data, logs of low quality (often by drillers only, or worse), poorly located boreholes, and no interpretation (Category Four).

Of the 68 reports, 15 fell in Category One (the most comprehensive), 24 in Category Two, 25 in Category Three and 4 in Category Four. It was decided that a limited comparison would be made of data from the different categories of report. Consequently, one particular litho-stratigraphic unit [till (boulder clay)] was selected and a simple statistical analysis of some of the geotechnical data was carried out for selected Category One test samples alone (322 samples), and for all test samples (1645 samples). The geotechnical classification test parameters chosen were moisture content, liquid limit, plastic limit, plasticity index, bulk density, clay, silt, sand and gravel contents, and N value from the SPT. In addition, data from consolidation, strength, compaction and chemical tests were compared visually. The comparison of the summary data is shown in Tables 18 and 19.

The analysis carried out measured whether the two sets of data differ from each other significantly in statistical terms for each parameter. For all parameters compared statistically, the two sets of data were identical to within 90% confidence limits (two-tailed Z-test) with the exception of moisture content. The reason for this is not known but variability is, perhaps, not surprising as samples tested may have been wetted during sampling or allowed to dry before weighing, leading to unreliable results. The visual inspection of consolidation, compaction and chemical test data showed the results from the two sets of test samples to be very similar. Only in the case of results from the quick undrained triaxial test was there any divergence. The reason for this is not known but it may be related to the way results are quoted in the reports. Sometimes an undrained shear strength value only is quoted. In this case the undrained angle of friction is assumed to be zero and the undrained cohesion equals the undrained shear strength. In other reports, both undrained cohesion and angle of friction are given. Consequently, cohesion will be less than the shear strength. As no attempt was made to separate the two ways of quoting the results, this may have contributed to the disparity in undrained cohesion values found for the two test sample data sets. Other factors, such as the sampling method used, may have contributed to the disparity but analysis of this is beyond the scope of this report.

From this limited statistical analysis it may be concluded that for the glacial till, at least, there is no significant difference in the summary geotechnical values obtained from a small (20%) sub-set of the test samples when compared with all the test samples. This implies that great savings in time can be made in the handling, manipulation and analysis of the geotechnical data by careful selection of limited "good quality" data only. However, further work is required to confirm that this conclusion is applicable generally.

The results of the following geotechnical tests and measurements (laboratory and in-situ) were abstracted from all categories of site investigation report and entered into the database.

1. Standard penetration test (SPT)
2. Rock quality designation (RQD)
3. Moisture content
4. Liquid limit
5. Plastic limit
6. Bulk density
7. Dry density
8. Specific gravity
9. Triaxial compression (undrained)
10. Consolidation
11. Permeability
12. Particle size analysis (PSA)
13. Compaction
14. California bearing ratio (CBR)
15. pH
16. Sulphate content

For most samples, only a few of these tests had been carried out, and rarely was a full range of test results available for a particular lithological/geotechnical unit within one borehole.

The classification of the rocks and soils in terms of their engineering characteristics was aided by analysis of the results from a number of tests, from the list above, which were more commonly carried out:

1. Standard penetration test (SPT)
2. Moisture content
3. Liquid limit
4. Plastic limit
5. Bulk density
6. Undrained triaxial
7. Consolidation
8. Particle size analysis

The results of this analysis are quoted in Tables 6 to 18 which give summary values of the geotechnical properties for the different soil units. There were insufficient data available for the rock units to produce a summary table of geotechnical properties except in the case of the mudstones of the Buckley Formation and the shales of the Productive Coal Measures and the Halkyn Formation (Tables 1 to 7).

The summarised geotechnical properties values given in Tables 1 to 18 are intended as a general guide to the engineering properties of the materials, and their variation. In most cases, the difference between the maximum and minimum values is high. The classification should only be used, therefore, as a guide when planning a site investigation and not as a substitute for a sampling and testing programme.

ENGINEERING CLASSIFICATION OF THE ROCKS AND SOILS

The division of the rocks and soils of the Deeside area into groups of materials of like engineering properties is based, in part, on the engineering properties themselves, as determined in the various tests and recorded in the site investigation reports, and partly on geological grounds. For example, whilst on the basis of their engineering properties there is little reason to divide, say, estuarine alluvial sands from sands of terrestrial alluvium, the two deposits can be clearly separated in terms of their environment of deposition and are, consequently, shown separately on the maps accompanying the main report. Hence, they have been described separately for engineering geological purposes.

In engineering terms, the rocks and soils of the Deeside area can be divided into eight broad groupings:

1. Rock
2. Fill, made ground
3. Organic soils
4. Normally consolidated cohesive soils
5. Loose non-cohesive soils
6. Heterogeneous soils (cohesive/non-cohesive soil mixes in varying proportions)
7. Overconsolidated cohesive soils
8. Dense non-cohesive soils

In general, group 1 materials are shown on the bedrock geology maps, and groups 2 - 8 on drift (unconsolidated deposits) maps, these maps showing rocks and soils respectively. It should be noted, however, that this simple division is complicated by the fact that on weathering, some of the rocks gradually change into soils in situ. Geotechnical data for these weathering deposits are not usually available, except in the case of the mudstones of the Buckley Formation and the shales of the Productive Coal Measures and the Halkyn Formation, for which limited summary geotechnical data are presented (Tables 1 to 7).

ENGINEERING GEOLOGY OF THE SOLID FORMATIONS (ROCK)

As indicated above, geotechnical data available from site investigation reports for many formations were limited. This was particularly true of the solid formations. In many cases no tests were carried out and consequently only brief descriptive information was available.

The solid, or bedrock, formations of the area have been divided up on the basis of their lithology, that is, the rock types present. To some extent, the engineering characteristics of the formations can be related to lithology, consequently, the main groupings are of rocks of broadly similar engineering characteristics. It should be noted however, that these broad lithological divisions take no account of the degree and nature of fracturing which can greatly influence engineering behaviour. The broad lithological divisions are set out in terms of decreasing strength, but again, this is generalization. The divisions almost equate to those shown on Thematic Element Map 1 (Bedrock Geology) of Campbell and Hains (1988), except that the Ruabon Marl has been divided into sandstones and siltstones on the one hand and mudstones and shales on the other; feldspathic sandstones of the Productive Coal Measures and the Halkyn Formation have been grouped with the quartzitic sandstones of the Halkyn and Minera Formations. This has been done because limited data on the properties of these rocks are available.

The groupings are:

1. **Limestones**

    i) dark grey, thinly bedded, argillaceous limestones with shales and interbedded with pale grey massive shelly limestones of the Cefn Mawr Limestone and Minera Formation.

    ii) Pale grey, coarse, rubbly, shelly limestones of the Loggerheads Limestone and the Llanarmon Limestone.

    iii) Dark brownish grey, porcellaneous limestones with subordinate fine to coarse shelly limestones of the Leete Limestone.

2. **Chert**

Grey and white laminated and glassy cherts and siliceous shales of the Minera Formation.

3. **Sandstones**

i) Brownish red and yellow, fine to medium sandstones of the Kinnerton Sandstone Formation.

ii) Purplish grey sandstones,weathering to a dense or very dense sand, and siltstones of the Ruabon Marl.

iii) Yellowish brown to grey and white, fine to coarse feldspathic quartzitic (variably calcareous lower in the sequence) sandstones of the Productive Coal Measures, Halkyn Formation and Minera Formation.

4. **Mixed Limestone/Sandstone/Siltstone**

Dolomitic and argillaceous limestone and calcareous sandstones and siltstones of the Minera and Foel Formations.

5. **Mudstones/Shales**

i) Purplish grey and reddish brown mudstones of the Ruabon Marl.

ii) Purple, reddish brown, yellow and grey mottled silty mudstones and seatearths of the Buckley Formation.

iii) Grey mudstones/shales and silty mudstones of the Productive Coal Measures and the Halkyn Formation.

iv) Grey, laminated, often cleaved silty mudstones of the Elwy Group.

Geotechnical data for these rock formations are so sparse that a general summary table of properties cannot be presented. However, where appropriate such summary data as are available are presented for certain lithological sub-divisions.

Limestones

The various limestone formations of the Carboniferous Limestone Group are found on the sparsely populated western side of the project area. Few geotechnical data are available. Only in one site investigation report is any attempt made to describe the limestones.

Limestone 1(i) is described as being dark grey, muddy, fine grained with numerous shale layers, interbedded with pale grey shelly limestones, whereas Limestone 1(ii) is referred to as being massive, with no distinct bedding horizons, light coloured, coarse grained and very fossiliferous. For engineering purposes no differentiation between them has been made because of lack of data.

The limestones are strong but mass strength will be dependent on the presence of mudstone/shale layers, their thickness and separation, and jointing. The degree of jointing is variable. Open or infilled cavities and sinkholes are present in the limestone which, clearly, may present an engineering hazard. Known dissolution features are shown on Thematic Element Map 2 (Superficial [Unconsolidated] Deposits) of Campbell and Hains (1988) as "Foundered ground" and "Swallow holes". The sinkholes are dissolution pipes which may be vertical or inclined. Any infilling in these features usually consists of clays and silts, sometimes as old as the Tertiary. In the project area such features as have been observed are generally of small size, though some substantial areas of "foundered ground" occur. Larger cave systems are known elsewhere in the Carboniferous Limestones of the Vale of Clwyd.

The limestone rockhead is probably very irregular with weathering opening up joints into which overlying superficial deposits may have been washed down. Because of the greater openness of near surface fractures and their greater frequency it is possible that limestones within about 5 m of rockhead can be ripped without blasting. In the more massive limestone however, blasting will be necessary. Cuttings in shaley limestone should not be steeper than 1:1 as weathering of the shaley layers may lead to their erosion and subsequent loss of support to limestone blocks.

The limestone would be suitable for use as a fill material after breaking up and can be classified as a rock for compaction purposes. The proportion of shale present is probably insufficient to affect the quality of the limestone fill.

No engineering information is available for limestone 1(iii), though it is described as a pale grey, coarse grained porcelleneous limestone with horizons of fine to coarse grained, shelly limestone.

Chert

The chert beds are variously described as light and dark brown colour banded, or grey to blue, strong, massive, very brittle, highly jointed rock with near surface joints that are open and clay infilled. Laminae of fine, cherty sandstone or shale have also been noted. Though there are no geotechnical data available for the cherts, they should be considered, from an engineering point of view, as similar to the limestones.

Sandstones

Sandstones of the Kinnerton Sandstone Formation

Sandstone 3(i) belongs to the Kinnerton Sandstone Formation which is of Permo-Triassic age. It is described as brownish red and yellow in colour and fine to medium grained.

Geotechnical data for this sandstone are only available from beneath the alluvial deposits of the Dee Estuary. A maximum of 9.5 m was penetrated and the sandstone is generally highly to moderately weathered [the weathering classification is that described by Anon (1972)] in the 2 to 3 m below rockhead, becoming slightly weathered below that depth. Fractures (from RQD and fracture space index records) range from a very low spacing (6 -

20 mm) within 2 to 3 m of rockhead to a high (200 - 1000 mm) spacing at greater depths.

Sandstones of the Ruabon Marl

Sandstone 3(ii) is described as a very weak to moderately strong, red, purple and grey sandstone weathering to a dense or very dense sand.

Values of uniaxial compressive strength are very variable ranging from around 10 MPa to about 81 MPa with a mean of 51 MPa (16 tests). For siltstones, in the same sequence, the mean strength is 50 MPa with a range from about 16.5 to 95 MPa (17 tests). Initial tangent modulus of elasticity values have also been determined. These are very variable ranging from 1.4 to 8.3 GPa for the sandstones and from 0.7 to 7 GPa for the siltstones.

Maximum nett safe bearing pressures for moderately strong sandstone ranging from 500 kPa for shallow foundations to 1500 kPa for pile foundations have been quoted for sandstones in the Flint area whilst values of around 2000 kPa for piled foundations have been suggested between Flint and Connah's Quay. If this sandstone is used as a pile foundation in the vicinity of the Dee Estuary, any concrete for cast in situ piles will have to be placed beneath water which, in some instances, may be saline. Care should be taken to determine the nature of the strata below the pile founding depth to avoid the possibility of over-stressing any underlying weaker mudstones or shales.

The sandstone will be difficult to excavate by digging and, therefore, will probably require ripping.

Sandstones of the Productive Coal Measures, Halkyn Formation and Minera Formation

The sandstones of group 3(iii) are found in the Productive Coal Measures, the Halkyn Formation and the Minera Formation and include the Halkin Rock and the Gwespyr Sandstone which are thicker members. The group is variable having been described as, reddish brown, yellow, light to dark grey in colour. The sandstones are generally moderately strong to strong and slightly to moderately weathered; however, near rockhead they may be weathered to a weak rock and heavily fissured with clay infilling the fissures. The sandstones vary from fine to coarse grained and may be laminated to medium bedded.

Geotechnical data are virtually absent for this group of sandstones. Maximum nett safe bearing pressures for footings on moderately strong sandstone beneath the weathering zone have been estimated at between 1000 and 3000 kPa. S.P.T.'s in the sandstones gave N values in excess of 50 for a full 450 mm test drive for slightly to moderately weathered rock and between 25 and 47 for completely weathered rock.

For the more massive, strong sandstones in this group, explosives will be required to break up the rock to facilitate excavation. Weathered sandstone near to rockhead may be ripped or dug.

Mixed Limestone/Sandstone/Siltstone

The Foel Formation, which forms the base beds of the Carboniferous Limestone Group, consists of dolomitic and argillaceous limestones with calcareous sandstones and siltstones. No site investigation encountered rocks of this formation, and hence no geotechnical data are available.

Mudstones/Shales

Mudstones of the Ruabon Marl

The purplish grey and reddish brown mudstones of the Ruabon Marl [Mudstone group 5(i)] are difficult to distinguish from the mudstones of the underlying Buckley Formation except, perhaps, in terms of their colour, though in the latter, seatearths are present. However, as the Ruabon Marl has been separately mapped, the group 5(i) mudstones are described separately. Few geotechnical data are available for these mudstones, but it is anticipated that their geotechnical properties and engineering behaviour will be similar to those of the mudstones of the Buckley Formation. Unconfined compression tests in slightly weathered mudstone and shale samples have indicated strengths in the range 16.5 to 18.6 MPa (14 samples) whilst initial tangent elastic moduli are in the range 0.7 to 2.3 GPa. Maximum nett safe bearing pressures for large spread footings have been estimated at 650 kPa. Strength may be expected to increase with depth, depending upon the degree of fracturing.

Mudstones of the Buckley Formation

Mudstones of group 5(ii) are purple, reddish brown, yellow and grey mottled in colour and make up part of the sequence of the Buckley Formation. The mudstone has been described as ranging from a blocky, fractured shale with traces of clay weathering to a stiff to very stiff silty clay with frequent litho-relicts in the top metre.

Geotechnical data for the mudstones are limited, but are summarised in Table 1 for the highly to completely weathered mudstones upon which tests have been carried out. This weathered zone varies in thickness, and up to 6 m have been recorded, though this should not be considered as a maximum. The weathered rock can be classified as a clay of low to intermediate plasticity (CL to CI) (Fig. 1) and low compressibility with a medium to high rate of consolidation. Strength determinations indicate that the clay is firm to stiff but, because of the fractured nature of the material it has been difficult to sample and, therefore, the clay is probably very stiff en masse with shear strengths probably greater than 500 kPa.

For pile foundations which may be considered for sites along the Dee Estuary, allowable end bearing pressures of about 200 kPa may be assumed for the weathered mudstones, with allowable shaft resistance in the weathered mudstones of around 150 kPa. For unweathered mudstones, allowable end bearing pressures of about 1000 kPa may be expected. Piles should be taken 3 m into the rock through the weathered zone. N values from the S.P.T. are in excess of 50, even in the weathered zone.

Mudstones and Shales of the Productive Coal Measures and the Halkyn Formation

These rocks of group 5(iii) are virtually indistinguishable in the field despite the time interval over which they were deposited. Geotechnically they are also very similar (Tables 2 to 7), though data for the Coal Measures shales are more limited. Consequently, the mudstones/shales are considered as one unit from the lithological and geotechnical points of view.

The rocks of mudstone/shale group 5(iii) are described as weak or very weak to moderately strong or strong, dark grey or grey, faintly laminated, jointed shales and mudstones. Strength decreases with increasing weathering. The laminae generally dip at low angles from sub-horizontal up to 20°. A number of joint sets have been observed, the principal one being sub-vertical with generally clean joints and occasional calcite infilling. Subsidiary sets at about 70° (to the horizontal) and 45° have been noted.

The shale weathers to a residual soil/completely weathered rock which is a firm to stiff, brown and grey, laminated, fissured silty clay with litho-relicts of shale. The weathering zone has been observed up to about 4 m thick, for completely weathered rock, but the degree of weathering will decrease beneath this depth. Weathering is greater where drift cover is thin.

The slightly weathered to highly weathered shale can be classified as a clay of intermediate plasticity (CI) (Fig. 1) and low compressibility. The completely weathered rock is a clay of intermediate to high plasticity (CI to CH) (Fig. 1). Shear strengths for the highly to completely weathered shales range from around 50 to about 200 kPa but increase with depth and decreasing weathering. Chemical tests indicate that pH's range from 7.4 to 8.0 and that, with the exception of one sample, sulphate determinations place the materials in class 1 of the BRE classification (Building Research Establishment 1975) so that no special precautions need to be taken over the concrete used (but see the comment under Fill and Made Ground below). For all degrees of weathering N values from the SPT are generally greater than 50, though occasional lower values have been obtained indicating local softening of the weathered clays.

The shales have not been considered suitable as fill materials unless there is no alternative; then it should only be used for low embankments. The weathered clay tends to be too wet and the shale has a tendency to weather rapidly, swell and settle. However, the swelling of the shale has not been quantified.

Side slopes for cuts have been recommended at 1V : 4H with drainage, particularly where the groundwater table is high. As observed above, precautions may have to be taken to prevent weathering and swelling.

Landslips have been observed at a few locations, particularly around Holywell. The failure planes for these slips appear to be located in weathered shale. For stability analysis at one site the following parameters were selected: peak effective strength parameters - c' = 15 kPa, Φ' = 21°; residual effective strength parameters - c' = O kPa, Φ' = 14°; bulk density 2.00 Mg/m^3. Where slopes steeper than about 7° are present on these shales, careful investigation should be made to try to determine if any slope movement has taken place and to check the effect of any proposed structure upon slope stability.

Limited information is available on bearing pressures for the shales. For the completely weathered rock maximum nett safe bearing pressures of 120 kPa for strip footings and 145 kPa for square footings have been suggested. For moderately weathered shales values in the range 300 to 600 kPa are proposed and for slightly weathered shales 1200 to 1600 kPa. For end bearing piles end bearing pressures between 1000 and 1500 kPa are suggested, usually towards the higher end of the range. The piles should be taken through the weathered zone (at least 2.0 m, but dependant on the site) into the hard shale beneath, if possible. Confirmation should be obtained that weathered shale horizons are not present within the stressed zone beneath the piles.

As these shales soften on contact with water, excavations should be protected as far as is possible against wetting and will need support.

Mudstones of the Elwy Group

No geotechnical data are available for the mudstones of this group [5(iv)] which are described as grey, laminated, often cleaved, silty mudstones.

ENGINEERING GEOLOGY OF THE UNCONSOLIDATED (SUPERFICIAL) DEPOSITS (SOILS)

The superficial deposits have been divided up both on the basis of their lithology (which can be directly related to their engineering characteristics) and the environment in which they were deposited. As indicated earlier, the soils of the Deeside area fall into seven main groups, in engineering terms. However, geologically similar soils formed in the same environmental conditions, and occurring in close proximity to each other would fall into separate engineering groups if their lithologies were different. As the soils have been mapped on the basis of their geological environment of deposition, the soils are described here according to their geological groupings. The relationship between engineering and geological groupings is indicated below.

1.	Fill, made ground	Chemical waste, domestic rubbish (including building waste), mine waste
2.	Organic soils	Peat
3.	Normally consolidated cohesive soils	i) Lacustrine alluvial clay ii) Alluvial clay and silt iii) Estuarine alluvial clay and silt
4.	Loose non-cohesive soils	i) Wind-blown sand ii) Some estuarine alluvial sand iii) Stratified glacial sand
5.	Heterogeneous deposits	i) Head ii) Alluvial clayey, silty fine sand iii) Estuarine alluvial clayey, silty sand
6.	Overconsolidated cohesive soils	Till (boulder clay)
7.	Dense non-cohesive soils	i) Glacial sand and gravel ii) Some estuarine alluvial sand

The seven engineering groups correspond with the five groups determined for the purpose of assigning presumed bearing values in C.P. 2004 (British Standards Institution 1972) except that here, the cohesive soils are divided into normally and overconsolidated, and the non-cohesive soils into loose and dense.

These various geological groups are shown on the drift map (Sheet/Map 2), though on that map the alluvial deposits are not divided up on lithological/engineering grounds because of their rapid and unpredictable lateral variation.

Fill and Made Ground

This is a very variable material in terms of areal distribution, composition, thickness and geotechnical properties. In general, it is loose, weak, highly compressible and unsuitable, in its present state, for the support of light-weight single-storey structures (safe bearing pressure >50 kPa). The most common materials are chemical waste, domestic refuse and mining waste. The latter has been observed to produce contamination of groundwater giving high sulphate contents and a subsequent risk of damage to concrete foundations. This risk should be appreciated in the vicinity of such waste tips, regardless of local geology.

Peat

Geotechnical data on peat are only available for three samples, though it was encountered in several boreholes (around Penyffordd, Northop, Holywell and Flint). The peat always occurs at, or near, the ground surface (except in one case where it was buried beneath 6.5 m of fill) and has been recorded as varying in thickness from about 1 m up to almost 3 m. It is usually associated with very soft, grey, organic alluvial clays and is, itself, usually very soft to soft, dark brown to black and fibrous, though often it has a significant clay content. Occasionally, sandy bands are found in it.

The peat is highly compressible with high moisture contents and Atterberg limits, and low density. If peat is identified as being present at a site, care will have to be taken to determine its thickness and extent (which is not likely to be great) to avoid structures being founded on it. The peat will not necessarily be present at the surface but may be thinly covered by alluvial sands or clays.

The limited geotechnical data for peat in the area are summarised in Table 8.

Wind-Blown Sand

As this deposit was not recorded in any site investigation report, no geotechnical data are available. The sand is generally fine grained.

Alluvium

Mainly restricted to the floor of the valley of the River Alyn and several smaller river valleys in the area, the alluvium varies from a soft, compressible, silty clay or clayey silt of low to intermediate plasticity (CL to CI) (Fig. 2) to a clayey silty fine sand, with occasional gravel. The geotechnical properties of the alluvium are summarised in Tables 9 and 10. In most respects it resembles the estuarine alluvium (see below) but with a distinctive lack of sea shells, a greater abundance of gravel, and greater heterogeneity produced by the more energetic depositional environment. Generally low maximum nett bearing pressures (23 kPa has been quoted) make it unsuitable for support of lightweight, single storey structures, though gravelly layers show much higher bearing pressures (100 kPa for foundations 1.2 m wide, reducing to 80 kPa for foundations greater than 3.0 m wide). The heterogeneity of the deposit makes foundation problems much more site-specific than in the case of the estuarine alluvium. No groundwater problems in terms of high sulphate contents were identified.

Estuarine Alluvium

Description

These deposits are the sediments of the Dee Estuary and occupy the area below the high water mark as well a narrow coastal strip bounded to the south west by the Chester to Holyhead railway, and, east of Connah's Quay, by the continuation of the A458. Three lithological units showing distinctive geotechnical properties were identified in borehole logs:

1. Alluvial clay: very soft to soft, occasionally firm, grey and brown, occasionally sandy, silty clay with sandy layers and organic zones. The deposit reaches 3.0 m in thickness, is of low to intermediate plasticity (CL to CI) (Fig. 2), normally consolidated and is of high compressibility and low strength.
2. Clayey silty fine sand: an infrequent lithology, intercalated with the alluvial clay and alluvial sand.
3. Alluvial sand: grey, brown and grey/brown, uniformly graded, slightly silty sand, commonly with sea-shells and occasional gravel. Generally saturated, loose but becoming denser with depth, reaching medium dense at about 10 m, in general.

The geotechnical properties are summarised in Tables 11 to 13.

A broad trend of coarsening with depth and away from the estuary margins is paralleled by increasing N values from the SPT, and decreasing moisture contents. This trend is a regional one, though on a site scale the different lithologies are observed to be intercalated.

Engineering Conditions for the Estuarine Alluvium as a whole

Inherent in their low-lying, tidal to supratidal location, the estuarine alluvium has characteristic groundwater problems associated with it. The water-table is ubiquitously near-surface (1-3 m depth) and may reach the surface in wet weather. The sediments are generally saturated with moisture contents commonly approaching liquid limits, for the cohesive soils, and the risk of 'running' conditions being produced in the non-cohesive soils. Excavations require groundwater control in the form of well-point dewatering and/or sheet piling with pumping to prevent flooding, and 'piping' and 'running' conditions. In general, such work should be avoided at peak tides. Tidally-induced fluctuations in hydrostatic pressures make SPT's, and the subsequently calculated safe bearing pressures, unreliable, unless such effects are accounted for. The loose nature of the material necessitates shoring of vertical excavations to ensure stability. Calculated safe bearing pressures are low, but increase to reasonable values with depth, making piling of foundations to these denser materials the most reliable technique. Safe bearing pressures for various sites on estuarine alluvium are summarised in Table 20. No serious groundwater sulphate problems are apparent as long as a good quality dense cement is used below the water-table, since seawater chloride counteracts the effects of the sulphate. However, higher groundwater sulphate concentrations have been observed in the vicinity of mine waste tips.

Engineering Conditions for the Estuarine Alluvial Clay

Very low safe bearing pressures (23 - 50 kPa) and high compressibility combine to make this material unsuitable for support of light-weight, single-storey structures. Large settlements can occur with small loads, and differential settlements can also occur. Better materials are found at depth, though skin-friction values in the soft clay can vary from zero to 7 kPa for bored piles or 8 kPa for driven piles, making this foundation type, where adopted, complex to assess.

Excavated faces soften rapidly on exposure to rain, so should be protected as soon as possible. Bogging down of plant in the clay, particularly in wet weather, is avoidable by placing a layer of gravel over the site prior to work commencing or by the use of geotextiles.

Engineering Conditions for the Estuarine Alluvial Sand

This material has higher safe bearing pressures (107 - 220 kPa) than the alluvial clay, which increase with depth, making the depth of foundations critical to their stability. Loose sand is marginally suitable for the support of light-weight, single-storey structures at surface level. However, the situation improves with depth,as the sand becomes medium dense at about 10 m depth. The loose nature, particularly of the shallow material may cause 'blowing' during SPT's, rendering the results unreliable, and may necessitate deeper than usual sheet piling to prevent 'boiling'. Water ingress at coarser, cleaner horizons is also common.

CBR's of 10% have been assumed for preliminary design purposes.

Heterogeneous Deposits - Head

Superficial deposits of a heterogeneous nature are found as a relatively thin veneer on many hillslopes and minor valley sides. These are transported deposits formed by the processes of solifluction (in glacial and immediately post-glacial times) and hillcreep. Typically, the material, known as head, is a sandy, silty clay with occasional gravel. The composition is very much controlled by the nature of the soils and rocks immediately up-slope, from which these deposits are formed. For example, south east of Holywell the head consists of completely weathered shale with a large proportion of fresh and slightly weathered mudstone fragments and occasional rounded grey limestone gravel. This originates from the shales of the Halkyn Formation and the Carboniferous limestones of the area. The soil is stiff to very stiff.

Head does not necessarily occur at the surface only. Where it was formed prior to the deposition of till (boulder clay) it is found underlying the till. The thickness of the head is very variable, ranging from less than 1 m up to at least 5 m.

Geotechnical data on the head are limited for the Deeside area, being restricted to sites around Holywell and Penyffordd; however, the head is much more extensive than this. The geotechnical properties are summarised in Table 14.

The more cohesive head can be described as a soft to firm clay and silt of low to intermediate, and sometimes high, plasticity (CL to CI and ML to MI and MH) (Fig. 2) and medium to high compressibility. SPT N values are extremely variable, probably reflecting the heterogeneous nature of the deposit. Where the head is coarser grained, and overlying finer grained deposits, perched water tables may be found within the head leading to possible running conditions in excavations and necessitating pumping.

Because of the way in which the head is formed by slow down slope movement and accumulation, there is always a possibility that these deposits may contain shear planes and that increases in pore pressure, undercutting of slopes or loading at the top of slopes may reactivate movement, causing failure. If head is identified, therefore, care should be taken to examine samples for the presence of shear planes. Sloping sites (including those where the gradient is only a few degrees) should also be inspected for the presence of small curved ridges representing the toes of solifluction lobes.

Piles founded on stronger underlying strata may be necessary where the head is weak and sheared. No appropriate bearing pressure data are available. There is no evidence of high sulphate contents associated with head, all but one of the few samples tested falling into class 1 (Building Research Establishment 1975). (However, note the comment in the Fill and Made Ground section, above, with regard to sulphate contents in the vicinity of mine tips).

Stratified Glacial Sand

This is a sporadically outcropping deposit of medium dense to dense, occasionally loose, silty to very silty sand with gravel. It contains sand and gravel and lenses or bands of soft red clay and laminated red/brown silty clay. Layers of rounded sub-angular gravel which may represent channel floor deposits are also found. It is a very heterogeneous deposit.

Maximum safe bearing pressures are variable and largely depend on gravel content; where no sand and gravel layers are present, safe bearing pressures of 80 kPa (strip footing 1 m wide) have been quoted, while with the presence of sand and gravel values of 130 kPa have been used. Water ingress is a problem at the coarser horizons, and intercalated clays and sand and gravel can give rise to perched water tables. The heterogeneity of the deposit can give rise to variations in geotechnical properties on a site scale, making differential settlements a possible problem. As a fill material the glacial sands are suitable only where no clay is present. The geotechnical properties are summarised in Table 15.

Lacustrine Alluvium

This is a very rare lithology, found at only one or two localities in the area, and consisting mainly of a soft to stiff, laminated compressible clay of low to high plasticity (CL to CH) (Fig. 3), or a soft to stiff gravel-free compressible clay of low to high plasticity associated with till (boulder clay). Problems of stability and settlement have been observed, mainly in the softer clays, and "as a consequence of the variable composition and fabric of glacial clays, the presence of weaker, more compressible layers under stronger, stiffer layers should be expected rather than considered as an exception" (Marsland 1977). S.P.T. results should be treated with scepticism. The limited geotechnical data available are summarised in Table 16.

Glacial Sand and Gravel (and Sandy Gravel associated with Till)

This is a widely occurring and generally thick deposit, found as large irregular spreads, as well as smaller accumulations within the till (boulder clay). It is a very variable, heterogeneous deposit; the material is in general, medium dense and well sorted. The main lithology is a well-sorted sand and gravel, with several subordinate lithologies. Clay can occur as a soft to firm matrix with the granular material but more commonly forms thin bands which can vary from very weak to very stiff. Loose gravelly or sandy silt layers, and zones/bands of silty sand are also found. Geotechnical properties are summarised in

Table 17.

Being well-sorted and of low cohesion these materials are characteristically prone to 'running' conditions, causing rapid deterioration of excavations. Shoring is, therefore, a necessity, and, where long term stability is required, cutting slopes back to 1 : 1 and perhaps covering with a protective material (for example, coarse sand and gravel, or vegetation) may be needed. Groundwater control during excavation can reduce these problems, especially in coarser material where high water inflows can occur, though support by timbering or sheet-piling will be required during pumping.

High hydrostatic pressures can occur in granular materials, exacerbating the above problems. Where the sand and gravel are overlain by a clayey layer, heaving and sagging conditions can arise on excavation, and there is an ever-present danger of such conditions existing just below the depth of investigation. These hydrostatic pressures, where present, render S.P.T. results unreliable, and give rise to perched water within the sand and gravel deposit.

Differential settlements are a problem caused by the heterogeneity of the deposit, and, combined with the effect of hydrostatic pressures on S.P.T. results, makes safe bearing pressures very site-specific, and difficult to generalize upon. Nevertheless, this soil group can be characterized, to some degree, in this respect (Table 21).

For cuttings, side slopes of 1V : 2H have been recommended for the glacial gravels, grading to 1V : 3H for the more sandy deposits. Counterfort and toe drains may be necessary where the water table is high. Well-graded glacial gravels can be used as embankment fill with side slopes of 1V : 2H. C.B.R.'s of 10% may be used for preliminary pavement design purposes.

For sand and gravel with clayey or silty layers, nett safe bearing pressures are generally between 100 and 200 kPa, but where with clay occurs as a matrix, lower safe bearing pressures of 53 to 107 kPa can be expected. The clay layers themselves vary from very weak to very stiff, with the latter giving safe nett bearing pressures of 500 to 600 kPa for pad foundations, with an end bearing pressure of about 900 kPa for piles.

Till (Boulder Clay)

Description

This is often a very thick and widespread deposit occupying, together with glacial sand and gravel, most of the area to the south east of the A548. Dominantly, it is a red-brown to dark brown, and occasionally grey, low to intermediate and occasionally high plasticity (CL to CI and CH) (Fig. 3) and low to high compressibility, silty and sandy clay with coarse, medium and fine gravel and cobbles and boulders. Subordinate bands and lenses of sand or sand and gravel, and occasionally firm to stiff grey laminated sandy, silty, pebbly clay also occur. Generally, the till is massive to well-bedded, but occasionally laminated. A weathered upper layer, up to 2.5 m thick, of orangey brown and grey, sometimes mottled, sandy clay with pebbles is usually present, but may be absent where alluvium overlies the till. The weathered material is soft to stiff while the underlying fresh till is generally stiff to hard. Granular materials are very loose to loose while the clayey materials are firm to stiff. Gravel content includes local Carboniferous rock types as well as exotic ones, and in some areas a correlation exists between gravel and subcrop lithology. Such local correlations are not mappable, and with the exception of the Carboniferous shale gravel, were found to have no significant effect on geotechnical properties. In the case of the tills with shale gravel content, higher plasticities (CH) have been measured and the till becomes darker grey in colour.

Engineering Conditions

Geotechnical properties are very variable within the till (Table 18), with three factors, namely lithology, depth and weathering state, being the main cause of these variations. These three factors combine in a complex manner and make generalisations and predictions about engineering behaviour in this material difficult. Nevertheless, several engineering problems have been observed to be associated with one, or more, of these factors.

Maximum nett safe bearing pressures range from 80 to 500 kPa. The upper weathered zone and softer clays occupy the lower part of this range from 80 to 240 kPa, with an improvement in weathering grade or an increase in depth of the clay layer giving higher values. Piles in the weathered zone can be expected to have skin friction values of 16 to 19 kPa (bored), 24 to 29 kPa (driven), and safe end bearing pressures of 240 to 270 kPa compared with 40 kPa skin friction (driven or bored) and 630 to 950 kPa safe end bearing pressures in the fresh, underlying material. The upper part of the safe bearing pressure range is occupied by very stiff to hard, unweathered clays, and unweathered to slightly weathered stony clays. These are end-member conditions, and various combinations of depth, weathering and lithology give rise to intermediate properties. Safe bearing pressures for various sites are summarised in Table 22. SPT and triaxial test results respond similarly to these variables, and 'piping' in sands can render the former unreliable. Softer horizons may not support piles or may make them uneconomical, and granular materials are subject to seepage and collapse during boring, making driven piles preferable in such lithologies. In addition, sloping sites have been observed to reduce expected safe bearing pressures, and differential settlements can result from localised variations in geotechnical properties. Underlying lithologies can produce unexpected local variations, notably a decrease in safe bearing pressures in sands where they are underlain by clay.

Groundwater can present problems in the form of perched water tables and seepages in more permeable granular materials. Clays are liable to softening on exposure to rain, so excavations should be protected as soon as possible. No groundwater sulphate problems were observed, with the exception of one locality where a high sulphate content and a pH of 3.2 occurred. This may have been due to contamination from a nearby mine waste tip. Excavations may require well-point dewatering in areas of high water-table, and will require support to ensure stability.

At natural moisture content the till is in general suitable as a fill material. Side slopes of 1V : 2H have been recommended.

Cuttings in the material should be stable at slopes of 1V : 2H to 1V : 2.5H, though shallower angles and provision of drainage to avoid pore-pressure build-up may be required, especially in the granular deposits, most particularly the sands. C.B.R. values of 5% may be adopted for preliminary pavement design in these materials.

ENGINEERING GEOLOGICAL IMPLICATIONS FOR PLANNING AND DEVELOPMENT

For planning and development purposes, the deposits of the project area can be divided into five principal engineering groups:

1. Rock
2. Overconsolidated cohesive soils (clay and silt)
3. Dense non-cohesive soils (sand and gravel)
4. Heterogeneous deposits (mainly head)
5. Normally consolidated cohesive soils (clay and silt), organic soils (peat) and loose non-cohesive soils (sand and gravel)

Rock

Whilst the rock, as a whole, often provides a suitable foundation for most light-weight structures, this is not universally true because the mudstones and shales, in particular, which are found near-surface in many parts of the area west of the Dee Estuary, are weathered to varying degrees and depths. The weathering process reduces the strength of the rock as it is weathered to a clay soil, increases compressibility and consequently is likely to reduce allowable bearing pressures. This weathering zone has been observed to a depth of 6 m, though will probably be thicker than this in places. In particular, weathering is greater where drift cover is less. As a general rule, piles would need to be taken through the worst of the weathered zone into fresh or slightly weathered rock.

A small number of landslips have been observed within the weathered shale and, consequently, careful investigation of sites on weathered shale slopes, particularly those steeper than about 7°, should be made to confirm the long term stability of the slope, and to enable suitable foundation designs to be prepared should instability be identified. The weathered shales also may be prone to swelling on wetting so that the material is generally unsuitable as fill and care is needed when constructing cuttings to reduce exposure of the shales to wetting.

The other rock types, that is, limestones, sandstones and cherts, generally provide good foundations for most light-weight structures. However, potential hazards are present and need to be considered in any site investigation. These include infilled or open cavities in the limestones, uneven rockhead surface with open and infilled joints and sandstone weathered to a dense sand. The limestones are generally strong but this strength may be reduced en masse by interbedded shale bands. Ease of excavation in the rock will also vary with the degree of weathering. Near rockhead limestones and sandstones will probably be rippable but blasting to loosen will be required at greater depths. The weathered shales are diggable but may need ripping when fresh.

Overconsolidated Cohesive Soils

The tills that make up the overconsolidated soils are very widespread in the Deeside area. They vary vertically from soft, weak materials at the top of the weathered zone to stiff to hard silty and sandy clays with gravel. They vary considerably in lithology laterally, and in their foundation conditions. They generally provide a satisfactory foundation for light-weight structures though the lateral variation on a site-scale may make differential settlements a possibility.

Granular materials in the till are prone to running or piping conditions or high water flows and hence cuttings may need good drainage. Clays in the till may soften rapidly on wetting leading to possible "boggy" conditions and unstable pit or trench walls. The till is generally suitable as a fill material. As a whole, the till needs careful investigation to determine its geotechnical variability in three dimensions at the scale of the particular site.

Dense Non-Cohesive Soils

These consist of the widespread glacial sands and gravels and some of the deeper and coarser estuarine alluvium. The glacial deposits are more variable, often containing clayey and silty horizons. Where dense or very dense, the deposits provide an adequate foundation for most purposes. However, for the estuarine alluvial sands piling to some depth may be necessary to find a suitable bearing horizon.

Excavations will need support and "running" conditions are likely, particularly in the estuarine alluvial sands, so that de-watering may be required. Cuttings through these deposits will need adequate drainage and high water pressures may be found in sands or gravels overlain by less permeable clays leading to possible heaving or sagging on excavation.

Heterogeneous Deposits - Head

This deposit is very heterogeneous in composition and is found in variable thicknesses (frequently less than 1 m) on hillslopes and valley sides. Its formation, involving downslope movement, makes the presence of shear planes within it a possibility and consequently there is a risk of landslipping. Where slopes are steeper than about 7° careful site assessment should be made to determine if shear planes or morphological features associated with slope movement are present.

Where head is thin, it may not pose any problems for development as it likely to be stripped off the site prior to construction. In other areas however, piled foundations may be necessary with piles founded beneath the head. The heterogeneity of the deposit makes generalisation difficult.

Normally Consolidated Cohesive, Organic and Loose Non-Cohesive Soils

These deposits are found in the Dee Estuary, in river valleys and as isolated pockets. The cohesive soils are often very soft to soft, very weak and highly compressible and usually unsuitable as a foundation for even light-weight, single-storey structures with large settlements possible even at low loads. The deposits can be variable in composition leading to possible differential settlement. Occasional gravel layers may provide better foundation conditions (though the nature of the deposits underlying the gravel must be determined). Conversely, peat horizons in the alluvium or organic clays in the estuarine alluvium may also be found, both of which are very weak, very highly compressible and unsuitable for any foundation.

Hydrostatic pressures in the estuarine alluvium may fluctuate because of the tides and this will affect bearing capacities. Groundwater control for excavations, together with shoring are essential.

In the estuary, piles founded in denser, coarser deposits at depth will usually be required, though skin friction values in the near-surface deposits will be low.

The non-cohesive loose deposits have higher bearing capacities than the cohesive soils in this group but they are often variable lithologically leading to possible differential settlements. Perched water tables and high permeabilities in this deposit mean that groundwater control for excavations and cuttings will be necessary. It can be used as a fill material only where clay is absent. Lithological and geotechnical variation on a site scale will need careful investigation.

Groundwater Chemistry

Few problems are anticipated with groundwater sulphate contents or pH values for most deposits except where the groundwater has been contaminated by water draining through mine waste tips. For sites in the vicinity of such tips groundwater quality should be carefully monitored to determine whether higher quality concrete is required and whether steelwork needs protection.

Peats may show acid groundwater conditions.

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APPENDIX

GEOTECHNICAL TESTS QUOTED IN THE DATABASE AND THEIR APPLICATIONS

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. [Details are given in B.S. 5930 (British Standards Institution 1981)]. 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 penetrometer 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 a value of 50, corresponding to very dense in granular soils and hard cohesive soils. The SPT is also frequently used in harder materials, that is, rocks and heavily overconsolidated soils, 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 the test, the number of mm penetration for 50 blows is quoted. When the results are quoted 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:

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	< 300
Rock/Heavily overconsolidated soils		< 200

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 excavatibility 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).

Moisture Content

The moisture content of a soil is defined as the mass of water in a soil divided by the mass of solids in a soil expressed as a percentage. It is determined by weighing a sample before and after drying to constant weight at a temperature of 105°C [details are given in B.S. 1377 (British Standards Institution 1975)]. Moisture content is a basic soil property and influences soil behaviour with regard to, for example compaction, and plasticity.

Atterberg or Consistency Limits

As the moisture content of a cohesive soil increases it will pass from a solid state to a semi-solid state in which changes in moisture content cause a change in volume. The moisture content at this change is the shrinkage limit. As the moisture content is increased further, the soil will become plastic and capable of being moulded; the moisture content when this change takes place, is the plastic limit.

Ultimately, as moisture content is increased, the soil will become liquid and capable of flowing under its own weight. This change takes place at a moisture content called the liquid limit.

The plasticity index is defined as the liquid limit minus the plastic limit and gives the range of moisture content over which the soil behaves as a plastic material. The methods and apparatus for determining the consistency limits are described in B.S. 1377.

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. If plasticity index is plotted against liquid limit on a plasticity diagram, a soil may be classified in terms of its plastic behaviour. The consistency limits also give an indication of soil strength and compressibility.

Density Tests

Bulk density is calculated by dividing the total mass of a soil (solids and water) by its total volume. It may be determined by the sand replacement method (in the field) or the core cutter method. In each of these a measured volume of soil at its natural moisture content is weighed and its density calculated.

Dry density is calculated by dividing the mass of soil after drying (that is, solids only) at 105°C to constant weight, by its total volume before drying.

Saturated density is calculated by dividing the mass of soil with its pore spaces filled with water, by its total volume. Full details of the determination of soil density are given in B.S. 1377. The density of soil in its various states of saturation are basic soil properties which are used in a variety of calculations including assessing overburden pressure, slope stability, surcharge pressure, and earth pressure on retaining walls.

Specific Gravity

The specific gravity of a soil is the mass of a dry soil divided by the mass of water displaced by that soil and is, therefore, dimensionless. For fine grained soils, a 50 ml density bottle is used, whilst for coarse grained soils, a 500 or 1000 ml pycnometer should be used. Full details of the test are given in B.S. 1377.

Specific gravity is a basic soil property and represents an average for the particles of different minerals present in the soil. The parameter is used to enable calculation of other useful soil properties. For example, voids ratio (which is related to porosity)can be calculated for a saturated soil if the moisture content and specific gravity are known.

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 the simplest most common method (quick undrained) a 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 and all air is removed. The 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 either the specimen shears or a maximum vertical stress is reached. Vertical displacement, axial load and pore pressure within the sample are measured during the test. The test is repeated on two further specimens from the same sampling point but at two different confining pressures. The results obtained from the three tests enable the undrained shear strength to be calculated as C_u, the apparent cohesion, and Φ_u, the angle of shearing resistance. The parameters obtained from this test may be used to determine the immediate bearing capacity of foundations in saturated clay.

Other variations on the test are suited to different applications. In the consolidated undrained test, free drainage of the specimen is allowed under cell pressure for 24 hours before testing (that is, the sample consolidates). Drainage is then prevented and the test carried out as before. This test is applicable to situations where a sudden change in load takes place after a period of stable conditions, for example where rapid drawdown of the water behind a dam takes place.

In the drained triaxial test, free drainage is allowed during the consolidation phase and also during the test itself. The results obtained would be applicable to long term slope stability assessment.

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 away to areas at a lower pressure at a rate controlled by the soil permeability. The removal of water causes a decrease in volume of the soil, the process is called consolidation.

The consolidation parameters are measured in the laboratory by placing a disc of soil in a metal ring, in a water filled cell. A constant axial 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, M_v, 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, C_v, which is a measure of the rate at which the volume change will take place for a given increase in stress, is also calculated.

The consolidation test results are important for designing the foundations of a structure and calculating the settlement that will take place during and after the construction of a building to ensure that settlement is neither excessive nor uneven over the foundation. It may also be important to ensure that the settlement (consolidation) which is caused by an early stage of construction has ceased before a second stage is started.

Permeability

The permeability of a soil is its capacity to allow water to flow through it. It may be measured in the laboratory on samples or in the field using boreholes.

In the laboratory, two tests are commonly employed, the constant head test for coarse grained soils and the falling head test for fine grained soils. In the constant head test a sample of granular soil is confined in a perspex tube, a constant head of water is applied to one end and water is allowed to flow through the sample. Manometers are connected through the cylinder walls to monitor the pressure along the flow path. Permeability may then be calculated, using Darcy's Law, from the path length, pressure difference, cross-sectional area of the sample and the quantity of water passed in a given time.

In a falling head test a sample of fine grained soil containing clay or silt is placed in a cylinder standing in a tray of water, a glass standpipe is connected to the top of the sample and filled with water. The time taken for the water level in the standpipe to drop a given distance is then measured. Permeability may then be calculated from the time, the drop in height, the cross-sectional area of the standpipe, the cross-sectional area of the sample and the length of the sample. (Details are given in B.S. 1377).

Laboratory tests do not take into account the structural differences in the soil and may not give a true permeability of the ground en masse. Pumping tests using boreholes give a more representative value but are more expensive.

In a field permeability test water is pumped out of a borehole and the effect on the water level in adjacent boreholes monitored or if a single borehole is being used it may be pumped out and the water level recovery time recorded. An alternative approach is to pump water into a borehole under pressure and measure the volumes of water flowing into the borehole at a number of different pressures (details are given in B.S. 5930). The information obtained from either method enable a coefficient of permeability for the ground as a whole to be calculated.

Permeability is used to predict the inflow of water during excavation or tunnelling and to design groundwater control schemes to deal with it. Permeability is important when assessing waste disposal sites or the siting and construction of water retaining structures such as dams, lagoons and canals. The assessment of potential well yields requires field permeability determination for the formations concerned.

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 subsample 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. The samples are dried and the contained solids weighed. 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 B.S. 1377.

Particle size distribution is used for classifying soil in engineering terms (B.S. 5930). Particle size distribution curves will indicate 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.

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 B.S. 1377).

The results of the compaction test show the moisture content at which it is best to place a given soil as fill or in an embankment.

California Bearing Ratio (CBR)

The California Bearing Ratio test is a penetration test carried out in the field, or in the laboratory, which compares the resistance of a soil to penetration by a standard plunger to the resistance to penetration shown by a standard crushed stone.

A series of samples are compacted in a 152 mm diameter mould at moisture contents around the optimum moisture content for maximum compaction. A surcharge weight is placed on the soil which is then immersed in water for four days. The mould is placed in a load frame and a plunger 48.5 mm in diameter is forced into the sample to a penetration of 2.5 and 5 mm. The CBR value is determined as the higher of the ratios of the resistance at 2.5 mm and 5 mm penetration to the standard resistance of crushed stone at the same penetrations. (Details are given in B.S. 1377). In the field, the plunger is jacked into the ground against the reaction of a heavy lorry. (Field values are usually lower than laboratory values). The results of the CBR test are used to assess the suitability of soils for use as base, sub-base and sub-grade in road construction.

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 B.S. 1377.

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 supersulphated 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 ion exchange resin which converts the sulphate content to hydrochloric acid. The acid content, and hence sulphate, is then determined by titration with sodium hydroxide. Details are given in B.S. 1377.

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

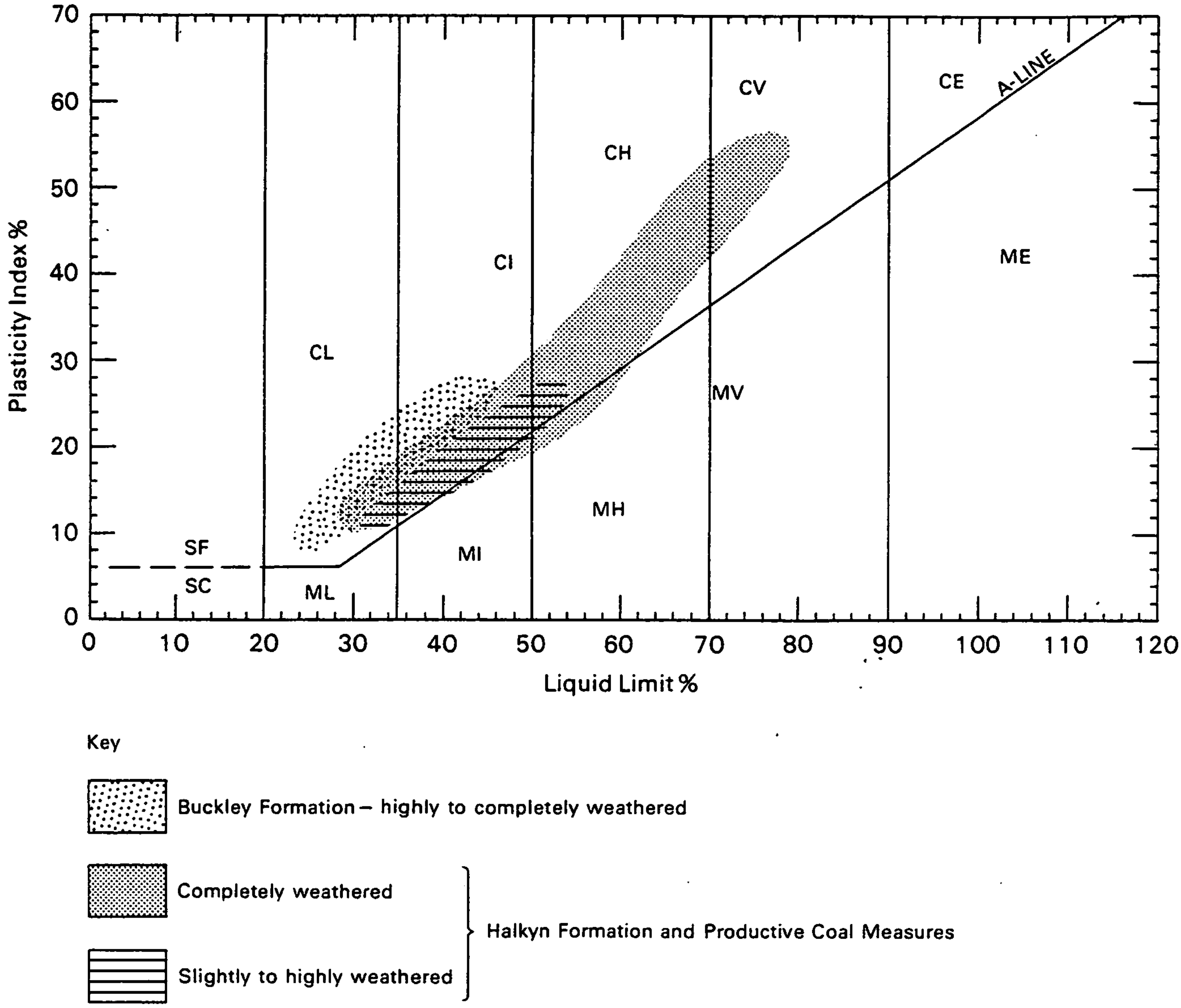

Figure 1 Plasticity Diagram for Weathered Mudstones and Shales

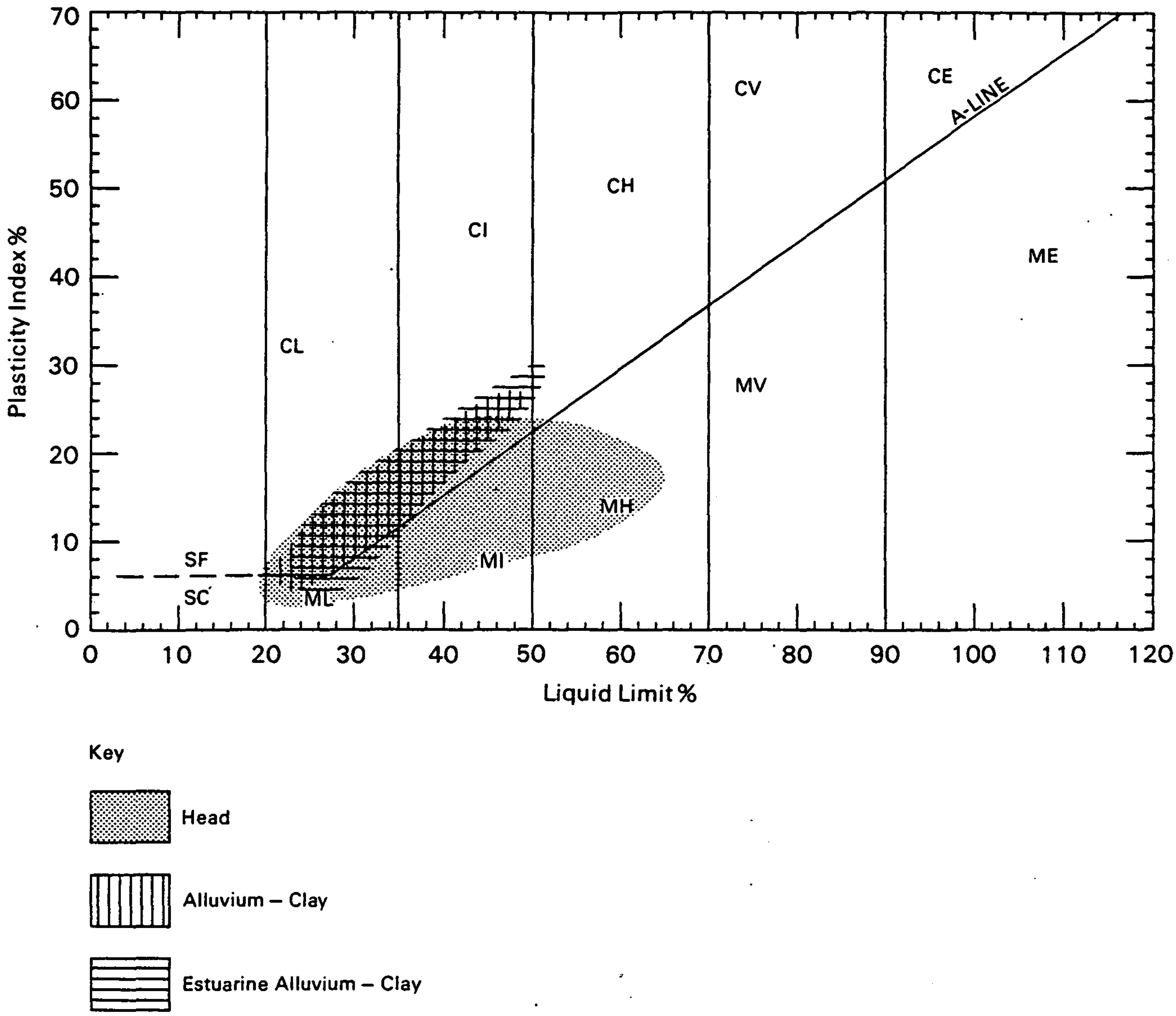

Figure 2 Plasticity Diagram for Head, Alluvial and Esturine Alluvial Cohesive Soils

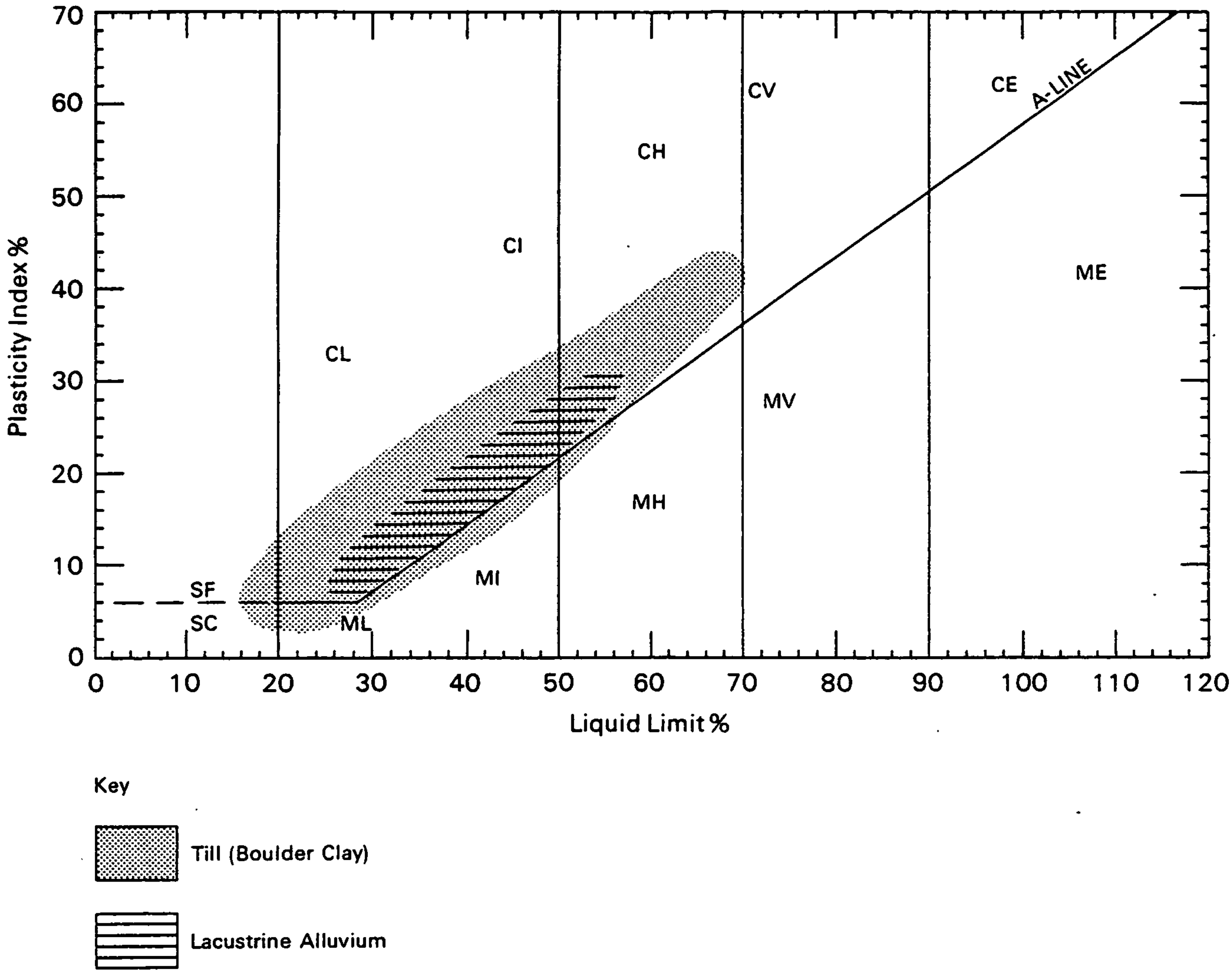

Figure 3 Plasticity Diagram for Till (Boulder Clay) and other Glacial Cohesive Soils

Table 1 Summary geotechnical data for Mudstones of Buckley Formation (Highly to Completely Weathered)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Mudstones of Buckley Formation - highly to completely weathered.	10	14	11	11	11	15						
	All > 50	10(2.8)	37(7.4)	18(2.5)	19(5.8)	2.28(0.07)						
		5–16	24–47	15–22	9–25	2.20–2.43						

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	6	6	3	3							**Weathered zone observed up to 6m thick. Low compressibility, medium to high rate of consolidation. Firm to stiff or very stiff. Piles require 3m penetration into rock, through weathered zone.**
Average (Standard Deviation)*			2,3	3,4							
Range	16–90	0–28									

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 2 Summary geotechnical data for Shales/Mudstones of Productive Coal Measures (Fresh to Slightly Weathered)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales/Mudstones of Productive Coal Measures; fresh to slightly weathered.	5 All $\geq$ 50	4 2-8					1 2.29					

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
* quoted where no. of samples > 10 + class values see relevant tables below	1 100	1 20						9 7.4-8.0	9 1		Shear strength increases with depth and decreasing weathering. Unsuitable as fill material. Side slopes 1:4 with drainage. Landslipping possible, slopes $>7^{\circ}$ at particular risk. Piles require to be taken through weathered zone.

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 3 Summary geotechnical data for Shales/Mudstones of Productive Coal Measures (Moderately to Highly Weathered)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales/Mudstones of productive Coal Measures; moderately to highly weathered.	27 23 values > 50 34-> 50	18 9(3.1) 4-16				2 2.13-2.26						

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
	1 359	1 0									Shear strength increases with depth and decreasing weathering. Unsuitable as fill material. Side slopes 1:4 with drainage. Landslipping possible, slopes >7° at particular risk. Piles require to be taken through weathered zone.

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 4 Summary geotechnical data for Shales/Mudstones of Productive Coal Measures (Completely Weathered to Residual Soil)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales/Mudstones of Productive Coal Measures completely weathered to residual soil.	5 All 50	40 18(5.6) 10–34	3 30–47	3 16–23	3 14–24	5 2.00–2.16						

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3 +	CBR %	Comments
* quoted where no. of samples ≥ 10 + class values see relevant tables below	9 72–170	9 0						1 8.0	1 1		Completely weathered zone observed to 4m. Shear strengths increase with depth and decreasing weathering. Unsuitable as fill material. Landslipping possible. Slopes >7° at particular risk. Piles need to be taken through weathered zone. Slopes 1:4 with drainage.

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975).

SULPHATES IN SOILS AND GROUNDWATERS

Table 5 Summary geotechnical data for Shales of Halkyn Formation (Fresh to Slightly Weathered)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales of Halkyn Formation (Holywell Shales) fresh to slightly weathered.	20 All > 50	5 8–18	4 38–44	4 19–24	4 17–23							

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
* quoted where no. of samples > 10 + class values see relevant tables below											Shear strength increases with depth and decreasing weathering. Unsuitable as fill material. Side slopes 1:4 with drainage. Landslipping possible, slopes >7° at particular risk. Piles require to be taken through weathered zone.

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
1	<0.1	Greater than	CLAYS Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
5	>100		SILTS

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2–0.5%		30 - 120 parts in 100 000
3	0.5–1.0%	1.9 - 3.1 grams/litre	120–150 parts in 100 000
4	1.0–2.0%	3.1 - 5.6 grams/litre	250–500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 6 Summary geotechnical data for Shales of Halkyn Formation (Moderately to Highly Weathered)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales of Halkyn Formation (Holywell Shales) moderately weathered to highly weathered.	17 9 values ≥ 50 21-> 50	14 17(5.6) 10-26	7 32-51	7 21-26	7 11-25	3 1.90-2.22	1 1.55					

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
No. of samples	2	2						1	1		Shear strength increases with depth and decreasing weathering. Unsuitable as fill material. Side slopes 1:4 with drainage. Landslipping possible, slopes >7° at particular risk. Piles require to be taken through weathered zone.
Average (Standard Deviation)*									2		
Range	124-154	0						7.4			

* quoted where no. of samples ≥ 10
+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 7 Summary geotechnical data for Shales of Halkyn Formation (Completely Weathered to Residual Soil)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Shales of Halkyn Formation (Holywell Shales); completely weathered to residual soil.	30	24	15	15	15	12	1					
	23 values 50	26(11.0)	49(12.4)	23(4.0)	27(11.2)	1.96(0.08)						
	8- 50	7-49	32-77	17-32	12-53	1.84-2.11	1.47					

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	12	12						3	3		Completely weathered zone observed to 4m.Strength increases with depth and decreasing weathering. Unsuitable as fill Landslipping possible particularly for slopes $>7^o$.Piles need to be taken through weathered zone.Cut slopes 1:4 with drainage.
Average (Standard Deviation)*	92(53)								1		
Range	23-198	0						7.6-7.7			

* quoted where no. of samples $\geq$ 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/ litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/ litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/ litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 8 Summary geotechnical data for Peat

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Peat; dark brown, fibrous, with occasional sand or clay.	1	3	1	1	1	2		1				
	4	59-364	370	137	233	1.00-1.60		2.19				

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	2	2						1	1		Very soft, highly compressible 1to 3m thickness recorded. Limited extent in area. Remove or avoid.
Average (Standard Deviation)*									1		
Range	9-32	0						5.0			

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
1	<0.1	Greater than	CLAYS Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
5	>100		SILTS

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 9 Summary geotechnical data for Alluvium (Clay and Silt)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Alluvium; soft compressible silty clay/clayey silt.	9	36	28	25	25	29		4	8	8	8	8
		22(6.1)	30(7.2)	17(2.2)	15(5)	2.04(0.11)						
	5–19	11–40	13–48	11–23	5–25	1.80–2.32		2.34–2.64	1–35	40–60	5–50	0–2

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	31	31	11	11				1	1		**Very heterogeneous. Low nett bearing capacities, unsuitable for support of lightweight structures.**
Average (Standard Deviation)*	64(80)	9(12.2)	3,4	2,3,4					2		
Range	0–140	0–32						7.0			

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/ litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/ litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/ litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 10 Summary geotechnical data for Alluvium (mainly clayey, silty fine sand)

Geotechnical Group		N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Alluvium, mainly clayey, silty fine sand.	No. of samples	10	12	4	4	4	7						
	Average (Standard Deviation)*	29(9.1)	16(15)										
	Range	17–45	9–60	20–57	13–23	7–34	1.84–2.34						

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	5	5									Very heterogeneous. Low nett bearing capacities, unsuitable for support of lightweight structures, though gravelly layers show much higher bearing capacities.
Average (Standard Deviation)*											
Range	65–119	0									

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2–0.5%		30 - 120 parts in 100 000
3	0.5–1.0%	1.9 - 3.1 grams/litre	120–150 parts in 100 000
4	1.0–2.0%	3.1 - 5.6 grams/litre	250–500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 11 Summary geotechnical data for Esturine Alluvium (Clay and Silt)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Estuarine Alluvium; soft, compressible silty clay/clayey silt.	22	36	11	10	10	21	1		8	8	8	8
	7(7.25)	33(14.3)	37(8.6)	21(3)	17(8)	1.87(0.27)			22(12.7)	38(18.8)	36(17.1)	5(11.1)
	1–19	5–75	23–52	14–24	6–31	1.13–2.30	1.62		3–36	18–67	15–73	0–32

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	21	21	5	5				7	12		**High water-table, tidally induced fluctuations in hydrostatic pressure. Very low bearing capacity, unsuitable for light structures. Prone to large differential settlements. Excavations need support and de-watering.**
Average (Standard Deviation)*	24(19.9)	7(9.6)	2,4	3,4					1,2		
Range	3–70	0–33						6.5–7.9			

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 12 Summary geotechnical data for Esturine Alluvium (mainly clayey, silty fine sand)

Geotechnical Group		N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Estuarine Alluvium; mainly clayey, silty fine sand.	No. of samples	87	25	1			6	2		17		17	17
	Average (Standard Deviation)*	20(13.4)	25(13)							5(7.9)		86(16.4)	4(9.7)
	Range	2-50	8-70	19			1.61-1.98	1.53-1.58		0-31		38-98	0-35

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	6	6	1	1	2			4	5		Near-surface water-table, prone to tidally induced fluctuations in hydrostatic pressure. Risk of 'running' and 'piping'. Excavations require shoring and de-watering. Unsuitable for support of lightweight structures.
Average (Standard Deviation)*			4	3					1,2		
Range	0-16	0-38			$4-7 \times 10^{-5}$			6.5-7.8			

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 13 Summary geotechnical data for Esturine Alluvium (Sand)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Estuarine Alluvium; alluvial sand with or without shells, no significant gravel.	549	38	1	1	1	38			163		163	163
	31(20.9)	21(5.6)				1.84(0.06)			2(4.9)		94(14.6)	34(45.8)
	6–120	5–38	35	20	15	1.72–2.03			0–27		0–100	0–100

FILE FORMAT

No. of samples Average (Standard Deviation)* Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
	5	5	1	1	4			25	28	34	High water-table, risk of 'running' and 'piping'. Tidally induced fluctuations in hydrostatic pressure. Excavations need shoring and de-watering. Bearing capacities increase with depth. SPT unreliable due to 'blowing' in sands. CBR 10%.
			2	4				7.9(0.4)	1,2	38(10)	
	3–30	0–34			$2x10^{-4}–7x10^{-6}$			7.2–9.2		5–45	

* quoted where no. of samples ≥ 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 – 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 – 0.3	Fluvio-glacial clays Late clays
2	Low	0.05– 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 – 1	25	High plasticity
3	1 – 10	25 – 5	Medium plasticity
4	10 – 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2–0.5%		30 – 120 parts in 100 000
3	0.5–1.0%	1.9 – 3.1 grams/litre	120–150 parts in 100 000
4	1.0–2.0%	3.1 – 5.6 grams/litre	250–500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 14 Summary geotechnical data for Lacustrine Alluvium

Geotechnical Group		N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Lacustrine Alluvium; soft to stiff, laminated, compressible clay.	No. of samples	10	53	24	24	43	4	4		10	10	10	10
	Average (Standard Deviation)*	40(23.5)	18(5.1)	34(9.1)	17(2.7)	17(7.7)	2.09(0.11)			16(10.4)	30(23.4)	34(27.5)	20(25.1)
	Range	10–60	10–34	21–54	14–24	6–34	1.90–2.35	1.64–2.04		5–33	5–63	8–85	0–58

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
No. of samples	32	32	9	9				7	7	4	Problems of stability and settlement, mainly in softer clays, the presence of which should be assumed rather than considered as an exception when this deposit is identified.
Average (Standard Deviation)*	80(46.6)	5(8.6)	2,3	3,4					1		
Range	20–140	0–32						6.6–8.2		7.3–19.0	

* quoted where no. of samples > 10
+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
1	<0.1	Greater than 25	CLAYS Montmorillonite
2	0.1 - 1		High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
5	>100		SILTS

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2–0.5%		30 - 120 parts in 100 000
3	0.5–1.0%	1.9 - 3.1 grams/litre	120–150 parts in 100 000
4	1.0–2.0%	3.1 - 5.6 grams/litre	250–500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 15 Summary geotechnical data for Hetrogeneous Deposits (Head)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Heterogeneous Deposits (Head)	37	100	68	63	63	69	2	8	13	13	13	13
	23(15.5)	18(9.3)	29(8.1)	18(5.6)	13(5.3)	2.10(0.19)			9(6.4)	14(11.3)	41(23)	34(23.3)
	3-75	8-72	18-67	12-48	1-26	1.49-2.46	1.90-1.97	2.61-2.71	0-23	1-40	20-93	0-70

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
No. of samples	70	55	14	1		2	2	9	9	4	Very heterogeneous, variable SPT N-values, very extensive as a thin veneer. Possible presence of shear planes, which loading may reactivate, with sloping sites particularly at risk. Perched water and running conditions in coarser horizons.
Average (Standard Deviation)*	36(28.6)	4(6.7)	3,4	4					1,2		
Range	10-120	0-30				1.93-2.04	9-10	7.1-7.9		3.8-10.2	

* quoted where no. of samples ≥ 10
+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 16 Summary geotechnical data for Glacial Sand and Gravel (or Sandy Gravel associated with Till)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Glacial Sand and Gravel or Sandy Gravel associated with Till.	398	88	6	5	5	18			64	64	64	64
	37(26.9)	20(99.5)				21.2(1.65)			5(2.75)	7(8.2)	23(21)	50(24)
	0–140	0–21	24–57	13–25	11–32	1.80–2.35			0–12	0–55	2–87	0–97

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
No. of samples	18	18				4	4	21	21		Very heterogeneous. High hydrostatic pressures in granular materials cause sagging/heaving on excavation, water ingress and unreliable SPT results. Bearing pressures variable, differential settlements possible. Cuttings 1:2-3 with drainage. CBR 10%. Excavations need support and de-watering.
Average (Standard Deviation)*	122(141.5)	10(12.6)						7.5(0.5)	1,2		
Range	20–140	10–40				1.90–2.22	6–9	6.5–8.5			

* quoted where no. of samples > 10
\+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 17 Summary geotechnical data for Glacial Sand/stratified

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Glacial Sand/Stratified; silty sand with gravel and occasional clay.	74	43	4	3	3	6	1		22	22	22	22
	25(16.6)	16(6.4)							8(9)	13(15.6)	69(23)	9(14.5)
	3–81	3–40	20–29	12–14	12–15	1.98–2.22	1.72		0–35	0–62	29–100	0–47

FILE FORMAT

No. of samples / Average (Standard Deviation)* / Range	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv+ m^2/MN	Cv+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3+	CBR %	Comments
No. of samples	4	4	1			3	9	9	9	8	Very heterogeneous; bearing pressures variable, dependent on gravel content and prone to differential settlement. Water ingress at coarser horizons, perched water above clays. Suitable as fill material where no clay is present.
Average (Standard Deviation)*			3						1,2		
Range	10–323	0–3				1.84–1.98	10–11	6.7–7.9		0.15–12.5	

* quoted where no. of samples > 10
+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 – 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 – 0.3	Fluvio-glacial clays Late clays
2	Low	0.05– 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv m^2/year	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 – 1	25	High plasticity
3	1 – 10	25 – 5	Medium plasticity
4	10 – 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2–0.5%		30 – 120 parts in 100 000
3	0.5–1.0%	1.9 – 3.1 grams/litre	120–150 parts in 100 000
4	1.0–2.0%	3.1 – 5.6 grams/litre	250–500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 18 Summary geotechnical data for Till (Boulder Clay)

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Till (Boulder Clay) sandy, silty clay with gravel.	596	997	286	284	284	549	51	11	106	106	106	106
	38(36.5)	19(70.4)	31(8.0)	16(3.75)	13(4.3)	2.18(0.15)	2.25(0.25)	2.62(0.07)	10(8.4)	14(12.8)	37(21.9)	38(24.7)
	1-180	2-50	3-67	7-33	2-39	1.60-2.50	1.49-2.30	2.45-2.68	0-42	0-68	0-91	0-100

FILE FORMAT

No. of samples
Average (Standard Deviation)*
Range
* quoted where no. of samples > 10 + class values see relevant tables below

Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
603	603	48	26	2	29	29	84	81	46	Variable deposit, stiff to hard when fresh. Weathering proved to 2.5m, stronger with depth. Water ingress and perched water in granular soils. Suitable as fill at natural mc. Embankment slopes 1:2. Cutting slopes 1:2-2.5 with drainage. CBR 5%.
112(73.1)	2(5.9)	3,2,4	3,2,4		1.96(0.13)	11.3(3.4)	7.6 (0.5)	1,2	10(7)	
0-619	0-34			$3x10^{-8}$-$1x10^{-5}$	1.61-2.21	6-21	6.2-8.4		1.0-3.7	

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
			CLAYS
1	<0.1	Greater than	Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
			SILTS
5	>100		

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
2	0.2-0.5%		30 - 120 parts in 100 000
3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 19 Summary geotechnical data for Till (Boulder Clay) Selected data

Geotechnical Group	N-value	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Bulk Density Mg/m^3	Dry Density Mg/m^3	Specific Gravity	Clay %	Silt %	Sand %	Gravel %
Till (Boulder Clay); sandy, silty clay with gravel.	141	215	66	65	65	76	3	11	46	46	46	46
SELECTED DATA	39(30.6)	14(6.3)	33(10.3)	17(4.5)	15(7.2)	2.14(0.15)		2.62(0.07)	9(6.7)	17(12.9)	31(18.0)	39(21.2)
	6–173	2–43	15–66	10–31	2–39	1.63–2.37	1.49–1.96	2.45–2.68	0–34	1–68	2–74	0–97

FILE FORMAT

	Undrained Cohesion kPa	Angle of Internal Friction (°)	Mv^+ m^2/MN	Cv^+ m^2/yr	Permeability m/s	Maximum Dry Density Mg/m^3	Optimum Moisture Content %	pH	SO_3^+	CBR %	Comments
No. of samples	64	30	10	4		14	14	6	13		
Average (Standard Deviation)*	63(39.1)	9(11.3)	3	3		2.02(0.12)	10.3(3.6)		1		
Range	0–175	0–33				1.74–2.21	6–21	7.2–8.0			

* quoted where no. of samples > 10
+ class values see relevant tables below

Class	Description of Compressibility	Mv m^2/MN	Examples
5	Very high	above 1.5	Very organic alluvial clays and peats
4	High	0.3 - 1.5	Normally consolidated alluvial clays, eg. estuarine clays
3	Medium	0.1 - 0.3	Fluvio-glacial clays Late clays
2	Low	0.05- 0.1	Boulder Clays
1	Very low	below 0.05	Heavily overconsolidated Boulder Clays. Stiff weathered rocks

After Head (1982)

COEFFICIENT OF VOLUME COMPRESSIBILITY Mv

Class	Cv $m^2/year$	Plasticity Index Range	Soil Type
1	<0.1	Greater than	CLAYS Montmorillonite
2	0.1 - 1	25	High plasticity
3	1 - 10	25 - 5	Medium plasticity
4	10 - 100	15 or less	Low plasticity
5	>100		SILTS

After Lambe and Whitman (1979)

COEFFICIENT OF CONSOLIDATION Cv

Class	Total SO_3	SO_3 in soil 2:1 water extract	In Groundwater
1	<0.2%		30 parts in 100 000
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3	0.5-1.0%	1.9 - 3.1 grams/litre	120-150 parts in 100 000
4	1.0-2.0%	3.1 - 5.6 grams/litre	250-500 parts in 100 000
5	>2.0%	5.6 grams/litre	500 parts in 100 000

After Building Research Establishment (1975)

SULPHATES IN SOILS AND GROUNDWATERS

Table 20 Foundation loading criteria for Esturine Alluvium at different individual sites within the Deeside area as quoted in site investigation reports

SITE LITHOLOGY	SAFE BEARING PRESSURES		
	Unpiled	Piled	
		End Pressure	**Skin Friction**
Alluvial clay	25-50 kPa		7 kPa (bored) 9kPa (driven)
Alluvial silty clay	23 kPa		
Alluvial sandy silt	50-70 kPa		
Mixed Alluvium	20-57 kPa		
Rafts on compacted, crushed rock	95 - 100 kPa		
Alluvial sand greater than 3.0 m depth	100-160 kPa 300 kPa (allowable)	335-850 kPa	0 kPa (loose)
Clay lenses	115 kPa		

Table 21 Foundation loading criteria for Glacial Sand and Gravel at different <u>individual</u> sites within the Deeside area, as quoted in site investigation reports

SITE LITHOLOGY	SAFE BEARING PRESSURES	
	Unpiled	Piled
		End Pressure
Sand and gravel clay zones and lenses of clay	160 kPa (B>1.0m) 160 x B kPa (B<1.0m)	
Sand and gravel with clayey zones	200 kPa (B>1.0m) 200 x B kPa (B<1.0m)	
Very stiff clay zones	500 kPa (strip) 600 kPa (pad)	900 kPa
Sand and gravel with thin clay band	133 kPa	
Sand and gravel with clay layers	200 - 300 kPa (0.5 m wide strip footings)	
Sand and gravel with clay binding	100 kPa	630 kPa
Sand and gravel with firm clay binding	53 - 107 kPa	

Table 22 Foundation loading criteria for Till (Boulder Clay) at different individual sites within the Deeside area as quoted in site investigation reports

SITE LITHOLOGY	SAFE BEARING PRESSURES		
	Unpiled	Piled	
		End Pressure	Skin Friction
Unweathered gravel in a matrix of clayey, very sandy silt, with cobbles & boulders	230 kPa (strip) 290 kPa (square)	630 kPa at 7.0 m depth	
Firm sandy clay with stone inclusions, slightly weathered	100 - 150 kPa at 1 m depth		
Stiff sandy clay with stone inclusions and lenses of sand, and sand and gravel	150 kPa (strip) 180 kPa (pad)		
Firm to hard sandy clay with stone inclusions and sand lenses. Slightly weathered	145 kPa (strip) 170 kPa (pad)	265 kPa	19 kPa (bored) 29 kPa (driven)
Unweathered		950 kPa	40 kPa (bored/driven)
Firm to stiff sandy clay with stone inclusions and lenses of sand. Unweathered, very stiff to hard			40 kPa (bored/driven)
Slightly weathered		260 kPa	18 kPa (bored) 27 kPa (driven)

Moderately weathered		240 kPa	16 kPa (bored) 24 kPa (driven)
Firm, brown, stony clay	133 kPa		
Firm to stiff, sandy clay with stone inclusions and occasional silt pockets	160 - 185 kPa (strip) 160 - 245 (pad)		
Stiff, sandy silty clay with a little coarse, medium & fine gravel	Allowable 268 kPa (strip) (F.S. = 3)		
Clay	125 - 300 kPa (strip) 145 - 350 kPa (pad)		
Sandy clay, laminated with stones and sand and gravel	250 kPa		
Sandy clay	250 kPa		
Clays in stony ground	160 - 520 kPa		
Sandy clay with stones	Allowable		
Below weathered zone	220 - 290 kPa (F.S. = 3)		
In weathered zone	180 - 230 kPa		
Stiff (top) to very stiff clay at depth, with thin layers of water-bearing sand	150 - 200 kPa		
Sandy, silty clay with a little gravel. Weathered	80 - 110 kPa		
Unweathered	100 - 150 kPa		

Stiff upper glacial clay	200 kPa (strip) 240 kPa (pad)
Firm, weathered	80 kPa
Glacial sand (loose-medium dense)	150 - 200 kPa (B>1 m) 150 - 200 x B kPa (B<1 m) (less if underlain by clay)
Stiff lower glacial clay	110 - 140 kPa (strip) 130 - 170 kPa (pad)
Very stiff to hard	230 - 420 kPa (strip) 500 kPa (pad)