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THE ENGINEERING GEOLOGY OF SOUTH-CENTRAL LEEDS

by
K.J. Northmore

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Preface

This report forms an integral part of the environmental geological mapping project of the south-central Leeds area, undertaken on behalf of the Department of the Environment and described in the *British Geological Survey Technical Report* WA/92/1 by Lake, R.D, Northmore, K.J., Tragheim, D.G. and Dean, M.T., 1992. This latter report contains a series of environmental geological maps (listed below) which complement the text presented here.

Map 1: Distribution of borehole sites.

Map 2: Solid and drift geology.

Map 3: Distribution of superficial deposits and levels of natural rockhead.

Map 4: Distribution of made ground.

Map 5: Deep coal mining.

Map 6: Quarrying and shallow mining.

Map 7: Engineering geology of the bedrock formations.

Map 8: Engineering geology of the superficial (drift) deposits.

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Contents

THE ENGINEERING GEOLOGY OF SOUTH-CENTRAL LEEDS

1. Introduction

This report describes the engineering geology of the South-central Leeds area, covered by the seven 1:10 000 quarter sheets:

SE23NW
SE23NE
SE23SE
SE33NW
SE33NE
SE33SW
SE33SE

The report forms an integral part of the environmental geological mapping project of the south-central Leeds area, undertaken on behalf of the Department of the Environment and described in the *British Geological Survey Technical Report* WA/92/1 by Lake, et al 1992.

The current report describes the classification of the deposits encountered in the area of south-central Leeds in terms of engineering geological units (units of similar engineering characteristics), and the assessment of these units with respect to their geotechnical properties and anticipated engineering behaviour. Classification and assessment was based on the creation of a geotechnical database, containing geotechnical data extracted from site investigation reports, and subsequent statistical analysis of the dataset.

The report concludes with a section highlighting the engineering implications for planning and development in the area.

Summary geotechnical data for all the geological deposits encountered are presented in tabular form in Appendix I..

2. Geotechnical Database

The combination of the geological units into groupings with similar engineering properties (engineering geology units) was carried out using geotechnical data extracted from site investigation reports on sites within the study area. The reports are mainly of three types, connected with:

(1) Building/construction sites in and around the urban areas

(2) Infrastructure construction and improvement works (ie roads and sewerage schemes)

(3) Land reclamation sites, landfill sites and existing or potential waste disposal sites (limited in number).

Geotechnical data were obtained from 173 site investigation reports (which were considered to contain data of sufficient quantity and quality to warrant incorporation into the geotechnical database). These reports presented data relating to 1666 boreholes. An assessment of the geological formations in terms of engineering geology was undertaken by analysis of the geotechnical database. This was supplemented by taking account of comments recorded in the site investigation reports, relevant to engineering behaviour, problems and hazard, to provide a summary appraisal of engineering experience in the area.

As part of the area is rural, the coverage of the site investigation reports is geographically restricted and the spread of data across the outcrop and subcrop areas of most formations is generally uneven, even for those formations for which a large number of geotechnical data have been obtained. The geotechnical data used here to classify geological formations in engineering terms are not, therefore, necessarily comprehensive nor representative of the entire outcrop of any particular deposit or formation. For some deposits (for example the superficial 'moundy glacial deposits') no geotechnical data are available. For others (for example the Lower Magnesian Limestone, or Cadeby Formation) data are limited to those from a small number of boreholes. In part, this lack of data reflects the limited areal extent to which these deposits crop out in the study area.

The various tests, from which the data were obtained, should have been carried out to the appropriate British Standard. However, experimental and operator error may have resulted in variation in results between one contractor and another, and between one report and the next. As very little information is presented in the site investigation reports relating to these possible errors, the data, as compiled, will inevitably include some which are inaccurate to a greater or lesser degree, resulting in a greater spread of results than would probably be found within any one report. However, during assessment of the site investigation data prior to databanking, efforts were made to reject clearly erroneous test results, the remaining records being accepted as valid. Nevertheless, the resulting statistics remain useful guides to the engineering properties of the various formations provided these points are considered and over-interpretation is avoided. Summary values of the geotechnical parameters are presented in Appendix I, Tables 1 to 13. It should be stressed that these provide only a general guide and should not be used as a substitute for adequate site investigation, or in detailed design calculations.

Geotechnical test data were obtained from 7698 borehole 'samples', including *in situ* field measurements. Results from the following tests were entered into the database:

(a) *In Situ* (Field) Tests

1. Standard penetration test (SPT)
2. Rock penetration test (RPT)
3. Permeability
4. Rock Quality Designation (RQD)
5. Fracture spacing index (If)

(b) Laboratory Tests

1. Moisture content
2. Liquid limit
3. Plastic limit
4. Bulk density
5. Dry density
6. Specific gravity
7. Particle size analysis
8. pH (acidity/alkalinity)
9. Sulphate content (of soil and/or groundwater)
10. Triaxial compression (undrained and/or drained)

11. Uniaxial compressive strength
12. Point load (axial and diametral)
13. Consolidation
14. Compaction
15. California Bearing Ratio (CBR)

Only rarely was a full range of test results available for any particular engineering geology unit. Tables defining various parameter classes recorded in the summary geotechnical data are presented in Appendix I, Annex A and a glossary of the geotechnical tests in Appendix II.

The geotechnical data were entered onto manuscript proforma and then keyed into a computer database. For this purpose an IBM compatible micro-computer and commercially available software were used for database creation and storage. Backup copies of the database are held by the Engineering Geology and Geophysics Group of the British Geological Survey. The validation and analysis of the stored data set to provide a statistical assessment of the geotechnical properties and the relevant graphical plots, were carried out using a commercial statistics and graphics software package. Results from this analysis were used to produce the summary geotechnical data in Appendix I, Tables 1 - 13 and as a basis for the assessment of the engineering geological characteristics of the various geological formations. For each geotechnical parameter in the tables, the values quoted are dependent on the number of samples (observations) as follows:

Number of samples	Parameter values
>5	50th percentile (Median)
>10	25th & 75th percentile (lower & upper quartiles)
>25	10th & 90th percentile
>100	2.5th & 97.5th percentile

These values are considered to represent the spread of data for each parameter more accurately than by quoting the mean, standard deviation and range, because the latter are sensitive to atypical values, often giving rise to misleading results (Hallam 1990).

The distribution of the main areas of 'made ground' is presented on Map 4 in the *British Geological Survey Technical Report* WA/92/1. No assessment is made here of the engineering characteristics of specific sites because the material and geotechnical variability of these spreads, coupled with the lack of sufficient data, make them difficult to classify in general engineering terms (but see pages 21 and 35). For planning purposes, the suitability of any potential development site will be dependent not only on the engineering characteristics of the in situ rocks and soils but also on the nature, thickness and proximity of areas of made ground.

3. Engineering Classification of Rocks and Soils

The grouping of the rocks and soils of the area into classes of like engineering characteristics (engineering geological units) has been based on an assessment of the recorded geotechnical parameters and those lithologies identified on the geological maps. The distribution of the engineering geology units (Maps 7 and 8, *BGS Report* WA/92/1) do not necessarily correspond with the divisions presented on the geological maps. For example, the Millstone Grit and the Coal Measures are shown separately on the geological map (Map 2, *BGS Report* WA/92/1) and the Coal Measures are further divided into the Lower and Middle Coal Measures. Each stratigraphical unit consists predominantly of interbedded mudstones, sandstones and siltstones. In engineering geological terms, a two-fold division has been made into 'moderately strong sandstones' and the dominantly argillaceous mudstone ('mudrock') sequences because the Millstone Grit and Coal Measures each comprise similar lithologies with broadly corresponding engineering characteristics. Similar groupings based on dominant lithologies and like engineering characteristics apply to the Quaternary superficial deposits. However, in Appendix I, Tables 1 - 13, geotechnical data are presented for the major lithologies in each stratigraphical unit to allow summary geotechnical information to be extracted on the basis of the divisions shown on the geological maps and in borehole logs.

Over significant parts of the study area, the bedrock is obscured by a variable thickness of superficial deposits. A broad distinction has been made, therefore, between the engineering geological characteristics of the bedrock formations (Map 7, *BGS Report* WA/92/1) and those of the superficial, or drift, deposits (Map 8, *BGS Report* WA/92/1).

4. Engineering Geology of the Bedrock Formations

The bedrock formations of the Leeds area have been divided into three engineering geological units based on their geotechnical and lithological characteristics. The relationship between these units and the geological (lithostratigraphic) units are shown below.

Engineering Geological Units	Geological Units
1. Limestones	Lower Magnesian Limestone (Cadeby Formation)
2. Moderately strong sandstones	i) Lower and Middle Coal Measures sandstones ii) Millstone Grit sandstones
3. Mudrocks	i) Lower and Middle Coal Measures mudstones and siltstones ii) Millstone Grit mudstones and siltstones

The 'Mudrocks' Unit includes those dominantly argillaceous deposits with geotechnical characteristics bordering on those of a 'weak' rock and a 'strong' (highly overconsolidated) engineering soil. The siltstones of the Coal Measures and Millstone Grit grade laterally and vertically into mudstones and sandstones and, geotechnically, straddle the boundary between the 'Moderately strong sandstones' and the argillaceous 'Mudrocks' Units. They are included in the latter unit because they are not distinguished separately on the geological maps. However, summary geotechnical data for the siltstones are given in Appendix I (Tables 6 - 7). Weathering of the bedrock formations has caused breakdown and softening of the fresh material into an engineering soil and the degree and depth of weathering is of particular importance with regard to engineering behaviour. The variability of the weathering products precludes the identification of distinct profiles in a study such as this. However, the effects of weathering on the engineering properties of the bedrocks are described and, where sufficient records exist, the geotechnical data presented in Appendix I, Tables 1 - 7 distinguish between those obtained for 'fresh to slightly weathered' and for 'moderately to completely weathered' rock.

1. Limestones

This unit comprises the limestone of the Lower Magnesian Limestone (Cadeby Formation) which crops out to a limited extent in the north-east of the study area. The limestone dips gently to the east and consists of a slightly to completely weathered, buff, fine to coarse-grained, marly dolomite with local oolitic horizons. Bed thicknesses tend to range from 0.06 to 0.6 m (thinly to medium-bedded), but beds over 2 m (thick-bedded) may also occur. Joint spacings range from between about 60 and 200 mm (closely-spaced) to over 2 m (very widely-spaced) and the joints are steep to vertical.

The lowest few metres tend to be sandy but unlike in the Castleford-Pontefract area to the south-east, the underlying Basal Permian Sand appears to be only patchily developed within this district. However, in 1981 a site investigation for the proposed A1-M1 link encountered at least 5.5 m of Basal Permian Sand (overlying Coal Measures) about 1.5 km north-west of Barwick in Elmet. Here the deposit was described as an 'off-white, highly weathered, thinly-bedded, fine-grained weak sandstone'. Geotechnical data relating to this sand in the study area are sparse and the reader is directed to the *British Geological Survey Technical Report No.* WA/90/3 for a geotechnical appraisal of this deposit as encountered in the Castleford-Pontefract area.

At outcrop, the topmost 0.5 to 1.5 m of the Lower Magnesian Limestone is often recorded as being weathered to a firm silty calcareous clay with limestone fragments. Below this zone the deposit may be weakened by weathering to permit a further 3 to 4 m of penetration by shell and auger boring. The depth to which weathering extends below rockhead is variable, but zones of highly weathered rock (with SPT N-values of about 50 or less) have been recorded to depths of c. 7 m below ground surface. The porosity and permeability of the limestone generally increase with the degree of dolomitisation, but permeability is primarily dependent on the size and concentration of fractures and joints. A single recorded field permeability value of 10^{-4} m/sec is indicative of a moderately permeable rock mass, but values may be expected to vary widely both areally and with depth. The percolation of water along joints and bedding planes has resulted in widening of these discontinuities and the creation of significant cavities by dissolution, leading to karstic rockhead conditions.

The limited geotechnical data obtained for the limestones in the study area are given in Appendix I, Table 1 where parameter ranges are principally controlled by lithology (particularly degree of dolomitisation) and weathering grade. Standard penetration test (SPT) results obtained for depths down to 7 m below ground surface range from 13 to 130 and show a clear trend of increasing N-value with depth and decreasing weathering grade.

The very few plasticity data plot on or near the A-line, typical of inorganic silts and clays of low to medium plasticity. Intermediate plasticity values relate to more marly material, whereas the low plasticity values probably reflect data for dolomitic limestone weathered to slightly cohesive silt.

No strength data were obtained for the limestone in the study area. However, twenty-four values of unconfined compressive strength for fresh to slightly weathered Cadeby Formation limestones in the Castleford-Pontefract area ranged from 2.4 to 36 MPa, with median and mean values classifying the rock as moderately weak. It should be stressed that strength tests carried out on intact material in the laboratory cannot properly represent the *in situ* rock strength if the rock is jointed or fractured. It is important that the influence of such discontinuities is taken into account in the assessment of strength and deformability for design purposes.

Design Considerations

i) Foundations

Because of the limited extent and rural nature of the Magnesian Limestone outcrop in the study area, no major foundation difficulties have been recorded in the collected site investigation reports. However, should development of these areas be considered in the future, potential foundation problems similar to those encountered in the Castleford-Pontefract area may arise. These are briefly summarised in Table 1. A limited number of tests for groundwater sulphate content have provided values which fall within Class I of the Buiding Research Establishment classification for sulphate-bearing soils and groundwaters (BRE Digest 250, 1981), indicating that the use of sulphate-resisting cement for buried foundations may not be required, but concentrations should be checked during site investigation. Shattered or highly weathered soft clayey zones should be removed and backfilled with suitable compacted material or concrete. For fresh to slightly weathered rock at depth, anticipated allowable bearing capacities of 1000 to possibly 2000 kPa may be used in design.

Table 1. Potential founding problems in the Magnesian Limestone

Geotechnical Consideration	Potential Foundation Problem
Wide range of measured engineering properties, particularly strength and deformability.	Difficulty in assessing representative values of bearing capacity and anticipated settlements.
High porosities and the presence of voids and cavities. Possible karstic conditions locally developed in rockhead.	Likely to reduce bearing capacity and cause excessive differential settlements below heavy structures. May require ground treatment or special foundation design before placing structures. May cause water inflow problems in excavations.
Local presence of highly weathered zones.	Likely to reduce bearing capacity. May preclude use of shallow foundations except for light structures.
Variable rockhead level and possible presence of infilled solution holes and channels.	Problems in achieving adequate bearing capacity at shallow depth. May result in excessive differential settlements unless deep foundations are used. Pile foundations of variable lengths may be required.
Local artesian groundwater conditions.	Likely to locally result in basal heave of excavations close to base of formation, or lead to stability problems of foundations in weathered rock if taken below water table.
Likelihood of previous or current subsidence of limestone associated with collapse of workings in the underlying Coal Measures. Also subsidence of superficial deposits associated with faulting/fracturing of underlying limestone.	May result in excessive differential settlements. May require treatment before placing foundations.

ii) Slope stability

No natural landslips have been recorded in the limestones. Because the dip of the bedding is low, slope stability problems in engineering works are likely to be confined to rockfalls and long-term degradation of rock faces rather than large scale instability. The most important factors influencing the stability of excavations are the presence of steeply inclined or vertical joints, highly weathered zones, brecciated limestones and weak, highly porous dolomites, which are particularly susceptible to weathering on exposure. Groundwater problems may also occur locally where mass permeabilities are high and the water table is at shallow depth. Where water ingress occurs in highly to completely weathered and brecciated rock or in fault zones, immediate support of excavations may be needed.

In general, the more resistant fresh to slightly weathered, bedded limestones can maintain stable slope angles of 70° to near-vertical, to heights in excess of 10 m, where no inclined joint surfaces are present or the cuttings are orientated such that discontinuity-bounded blocks are stable. Slopes of 1V:1H are recommended as a preliminary guide for excavations and cuttings in most of the limestone area, but regrading to 1V:2H may be necessary for highly weathered, brecciated or faulted zones and for long-term stability in beds particularly prone to degradation on exposure.

iii) Excavatibility and suitability as fill

Highly to completely weathered, brecciated and broken rock may be excavated by mechanical scraper or machine shovel; fresh to slightly weathered, jointed and fractured rock will require machine ripping. Massive limestone with widely spaced joints will need pneumatic breakers or minor blasting.

Fresh to slightly weathered rock is suitable as rockfill. Highly weathered and fractured rock is generally unsuitable as rockfill but may be used as a bulk fill material. The limestone should be assumed to be frost-susceptible for design purposes, and this should be taken into account for calculations of minimum pavement thicknesses.

2. Moderately Strong Sandstones

These are the sandstones of the Millstone Grit and Coal Measures groups. They are grey to greyish brown, moderately to well-jointed, moderately strong, variably micaceous, locally conglomeratic, medium to coarse-grained (in the Millstone Grit) and fine to medium-grained (in the Coal Measures) quartzose sandstones, with occasional siltstone and mudstone interbeds and coal partings. Bedding separation ranges from about 20 - 200 mm (very thinly to thinly bedded, or 'flaggy') to over 2 m (thickly bedded), and well-developed cross-bedding is common. Individual units range in thickness from thin impersistent bands within the mudrock lithologies to thick regionally persistent units such as the Rough Rock and the Thornhill Rock.

The depth and intensity of weathering is variable but where exposed, the complete weathering of the sandstones to a silty clayey sand or very sandy clay containing weathered sandstone fragments is often encountered to depths of 0.5 to 1.5 m, although residual sandy soils up to 5 m thickness are not uncommon in the Coal Measures. The change from residual soil to weathered rock is generally fairly distinct. Although the weathering grade decreases with depth, the thickness and character of the weathered zone may vary markedly in the vicinity of faults where locally steep bedding dips and fracturing have enhanced permeabilities. In these areas, 'pockets' of highly weathered sandstone rock have been encountered to depths in excess of 11 - 12 m below ground surface. The degree of weathering is a major factor in controlling engineering behaviour and summary geotechnical properties for 'fresh to slightly weathered' and 'moderately to completely weathered' sandstones of the Millstone Grit and Coal Measures are shown separately in Appendix I, Tables 2 - 3.

Standard penetration test (SPT) results vary widely according to weathering grade. A tentative correlation (Meigh, 1968) between SPT values and rock strength indicates that slightly weathered sandstone rock in the Leeds area falls into the 'medium strong' class with N-values generally between 100 - 250. N-values for moderately to completely weathered rock generally range from about 20 to 200, straddling the 'weak to moderately strong' classes. Where the residual soils developed over the sandstone outcrops are granular, the SPT values are generally indicative of medium dense to dense sands with values, and hence bearing capacities, improving with depth.

Soils developed over the sandstones are generally of low to intermediate plasticity. Some recorded data values do, however, indicate high plasticity materials and these may reflect the presence of mudstone layers or laminae within the weathered sandstone sequences or, more likely, the presence of Head deposits, which are difficult to distinguish from the *in situ* sandstone weathering profiles on borehole logs.

Insufficient data has been obtained to assess the *in situ* permeability of the sandstones but permeabilities can be expected to vary markedly across the outcrops depending on the degree of weathering, the size and spacing of joints and the presence of fault-zones. Where large or closely spaced discontinuities occur, moderate or high permeabilities (10^{-5} to 10^{-2} m/sec or higher) can be expected. Intact sandstone, with no or few joints or fractures, is generally only slightly to moderately permeable.

Uniaxial compressive strengths and point load indices for intact fresh to slightly weathered rock generally range from about 22 to 55 MN/m^2 and 1.0 to 2.5 MN/m^2, respectively, thus classifying the sandstones as 'moderately strong'. It is uncertain, however, whether all these test data relate to samples tested at their natural moisture content. If the samples were allowed to dry out prior to testing, anomalously high laboratory strengths may have been obtained which bear little relation to field values. Conversely, saturated test specimens may have yielded strength values up to 50 per cent lower than those for dried or partially saturated samples. However, tentative correlation of the recorded uniaxial strengths with SPT N-values (see above) suggests that the summary strength values presented in Appendix I, Tables 2 - 3 are generally representative of the *in situ* field strengths of intact sandstone in the study area. Rock strength decreases with increased weathering, and it should be remembered that tests on intact laboratory specimens do not represent the *in situ* strength of the rock mass, which is largely influenced by the frequency, orientation and nature (e.g. if infilled or 'clean') of the discontinuities.

Design Considerations

i) Foundations

In general discussion of the bearing capacity and settlement characteristics of the Upper Carboniferous rocks, consideration must be given to existing or potential areas of mining subsidence, because in some cases this may be an overriding factor when calculating settlements, and the presence of shallow mine workings may affect bearing capacity considerations. The potential hazard from shallow mine workings (usually within 30 - 40 m of rockhead), shafts and bell pits are discussed further in section 6, page 39. Adequate provision should be made in any proposed site investigation to ascertain not only the thickness and founding characteristics of the sandstones, but also the extent and depths of these workings, before consideration is given to the appropriate foundation design. The Millstone Grit and Coal Measures rocks are also extensively faulted and the presence of shattered rock in fault zones may give rise to adverse groundwater conditions in addition to differential settlements. Pretreatment of the ground may be necessary and site investigations should aim to delineate these problem areas as accurately as possible, prior to placing foundations.

A variable thickness of Head deposits is almost inevitably widespread, but patchily developed, over most of the sandstones. Although probably less than a metre thick in most areas, increased thicknesses may be expected at or near the foot of slopes. Such deposits are extremely variable in composition and can give rise to excessive differential settlements. Below or on slopes, they may also contain numerous shear surfaces and should be removed prior to placing foundations. Where the bedrock profile consists of (usually thin) alternating bands of sandstone and mudstone, the weathered zone may include bands of stiff clay, derived from decomposed mudstone, underlying less weathered and more resistant sandstone. Such a situation could have important implications for foundation design if the residual clay layer is not proved by penetration of the sandstone during site investigations.

Due to the wide range of rock conditions found in the Upper Carboniferous rocks, recent practice has been to use the results of *in situ* testing (eg. plate loading tests) to assess the bearing capacity and settlement characteristics at any particular site. For shallow foundation levels in the weathered zone, allowable bearing pressures of between 250 - 500 kPa have frequently been used, increasing in some cases to as much as 2000 kPa in thick layers of unweathered massive sandstone. Both driven piles and bored cast *in situ* piles have been used to carry foundation loads into the Upper Carboniferous rocks. Driven piles have disadvantages in that they tend to cause shattering and may 'hold up' on thin sandstone layers underlain by severely weathered shale or mudstone. Heave is also likely where driven piles are emplaced at close centres. Bored cast *in situ* piles are generally to be preferred as they can more readily penetrate the weathered zone and the materials passed through can be identified (in some cases, inspection of the base can also be carried out to confirm sound bedrock). It should be emphasised that the site investigation should be sufficiently detailed to give adequate knowledge of the rocks below the toes of piles. The allowable pile loading can often be equivalent to the maximum allowable stress in the concrete, namely 5 MPa of pile cross-sectional area using normal concrete. The penetration necessary to develop this capacity will vary with the rock conditions but where the founding medium consists of a sufficiently thick layer of sound sandstone, a penetration of about one pile diameter is usually satisfactory, reducing to about half a pile diameter where the sandstone is very hard. Pile loading tests should be carried out prior to the main work to confirm chosen loadings and penetrations. Where the number of piles involved in the construction may be insufficient to justify the expense of pile loading tests, it is prudent to use more conservative loadings. Where the weathered zone persists to appreciable depths or where there are steeply dipping strata (eg. near fault zones), it may be necessary to terminate piles in material which is outside the category of sound unweathered bedrock. In such cases it is probable that reduced loadings of about 3000 MPa of pile cross-sectional area can be used, again subject to confirmation by pile loading tests.

Data obtained for soil and groundwater sulphate concentrations in the study area nearly all fall into Class 1 of the BRE classification (Digest 250, 1981: *Concrete in sulphate-bearing soils and groundwaters*). However, sufficiently high sulphate concentrations have been recorded in the vicinity of colliery tips (Crutchlow, 1966; Meigh, 1968) to warrant special measures, such as the use of sulphate-resisting cement, to avoid concrete attack.

ii) Slope stability

No natural slope failures have been recorded in the sandstones. In excavations for construction works cut faces in massive to moderately-jointed, fresh to slightly weathered sandstone may remain stable at steep angles, but the presence of interbedded siltstones and mudstones will reduce stability and side slopes of 1V:2H have been suggested for preliminary design purposes. Perched groundwater tables may give rise to high hydrostatic pressures, causing heave at the base of excavations, and dewatering will be required where groundwater seepage is encountered. Excavations in fault zones may require immediate support due to the presence of shattered or brecciated rock and clay gouge.

iii) Excavatibility and suitability as fill

Weathered sandstone may be excavated by mechanical scraping or digging. Fresh or slightly weathered rock may require ripping and, in confined spaces, pneumatic tools for excavation. Blasting may be needed for major excavations in massive sandstone.

Sound sandstone rock is suitable for embankment fill if care is taken in selection and excavation. Although suitable as bulk fill, use as a high grade fill can be limited due to the variable amounts of clay and silt size particles, which may form the cementing medium of many of the sandstones, and to the common occurrence of argillaceous bands. For compaction purposes, the sandstones are generally classed as a graded granular soil.

3. Mudrocks

This unit encompasses those dominantly argillaceous formations with engineering characteristics bordering on those of a 'weak' rock or a 'strong' overconsolidated soil. It includes the mudstones, siltstones, coals and seatearths of the Upper Carboniferous (Coal Measures and Millstone Grit groups). The siltstones, which grade laterally and vertically into mudstones and sandstones, are not distinguished on the geological maps and are here included in this unit despite having geotechnical characteristics which straddle those of mudstones and moderately strong sandstones. Coal seams and seatearths (unbedded mudstone or siltstone rootlet beds generally underlying coals) form only a small proportion of the Upper Carboniferous strata and the lack of geotechnical data relating to these lithologies precludes a regional assessment of their engineering properties here. Coal seams may cause problems for construction because they are commonly highly permeable and thus weather and weaken rapidly on exposure. They also pose the potential hazard of spontaneous combustion if exposed at proposed foundation levels (see section 6).

The 'Mudrocks' unit consists predominantly of medium to dark grey, moderately fissured, weak to moderately strong mudstones, silty mudstones and siltstones. These are particularly susceptible to weathering processes and completely weathered firm to stiff, orange-brown and pale grey mottled clay soils commonly occur within 2 to 6 m of ground surface, followed by a gradual transition through less severely weathered material into unweathered bedrock. Zones of highly weathered material, comprising softened mudstone clasts in a silty clay matrix, may occur to depths of about 10 to 15 metres, and possibly deeper in fault zones. As the degree of weathering is a major factor in controlling engineering behaviour, summary geotechnical properties for 'fresh to slightly weathered' and 'moderately to completely weathered' mudrocks of the Millstone Grit and Coal Measures are presented separately in Appendix I, Tables 4 - 5. Summary geotechnical properties for the siltstones are shown in Appendix I, Tables 6 - 7.

Standard penetration test (SPT) results vary widely depending on weathering grade, but N-values for fresh to slightly weathered rock generally range from about 75 up to 250, approximating to weak to medium strong rock. For highly to completely weathered mudrocks, N-values of 40 or less are frequently recorded.

Plasticity data (Fig. 1) for the moderately to completely weathered mudstones cluster around the A-line in the manner typical of silty clays/clayey silts of low to high plasticity. For fresh to slightly weathered material, the few plasticity data plot as silty clays of generally low plasticity. These latter data may relate specifically to very silty mudstones which are less susceptible to breakdown on weathering to plastic clays than the less silty mudstones. The A-line plot for the siltstones (Fig. 2) indicates materials of generally low to medium plasticity. Consolidation data show the mudrocks to be of generally low to medium compressibility although high consolidation settlements have been recorded locally. In general, the effect of enhanced weathering results in increasing plasticity and moisture content and decreasing density and shear strength.

The few field permeability values for the mudstones, ranging from 10^{-4} to 10^{-6} m/sec, classify the rock mass as slightly to moderately permeable, but this may vary depending on weathering grade, fissuring and the presence of interbedded siltstones, sandstones and coal

Fig.1. Plasticity diagram for mudstones ('mudrocks') of the Millstone Grit and Coal Measures.

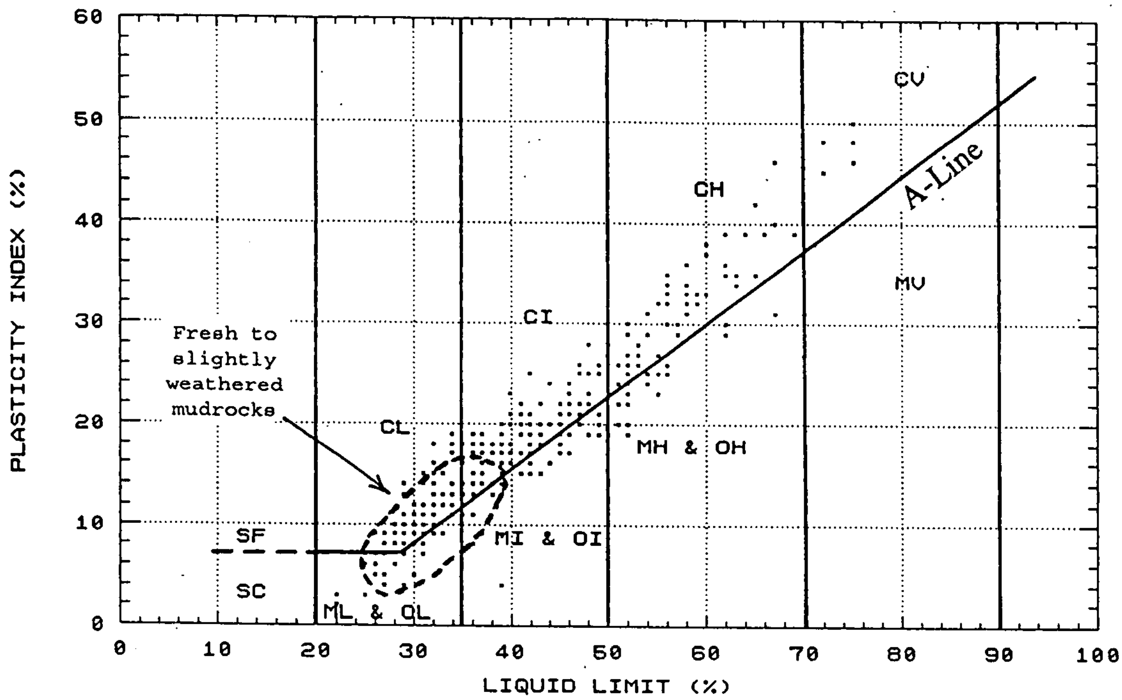
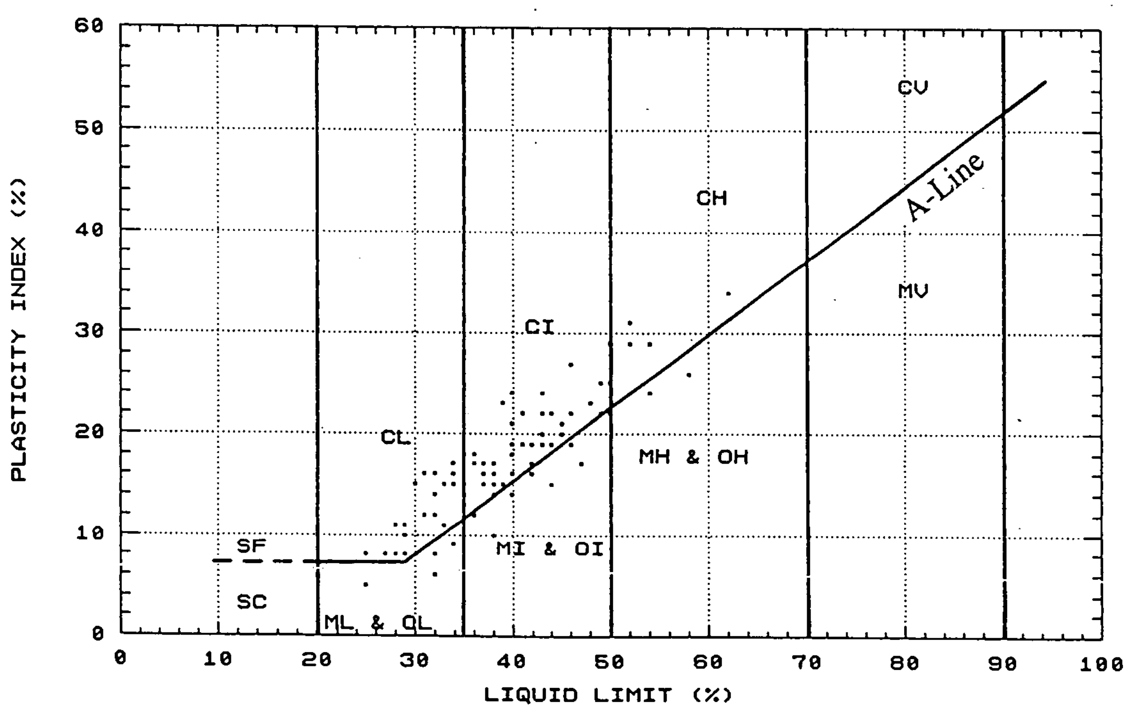


Fig. 2. Plasticity diagram for siltstones of the Millstone Grit and Coal Measures.



horizons. Waterlogging of the ground may occur locally where the mudrocks are weathered to clay and also where mudstone seatearths crop out.

Design Considerations

i) Foundations

Shallow mine workings present a potential foundation problem throughout much of the Coal Measures outcrop, and any site investigation should ascertain the presence, depth and extent of these. Once the workings have been accurately delineated, piled foundations set in sound rock below the worked level present the safest option for all but the lightest structures, although ground pre-treatment by grout injection has been successfully employed in some cases. Problems for development in areas of shallow mining are discussed further in section 6.

For shallow strip foundations within the weathered zone, allowable bearing pressures of 50 to 300kPa have been considered to be satisfactory, increasing to 500 to 1000 kPa at depths greater than 1.5 m in fresh to slightly weathered competent mudstone or sound siltstone. Working loads of about 600 kN are suggested for 0.5 m diameter piles set in sound mudstone, with bored cast *in situ* piles being preferred to driven piles. In most cases, penetration of three to four pile diameters into sound mudstone bedrock is necessary to develop sufficient bearing capacity.

When exposed to wet conditions, the mudstones swell, lose strength and rapidly deteriorate to clays. Mudstones with a low silt content appear to be most susceptible to this weakening process which may result in a marked reduction of bearing capacity and loss of side friction for piles. Such lithologies should be subject to slaking tests during site investigation and be protected from water ingress during excavation by cut-off drains, sheet piling or similar measures. Borings for piles should be cased through water bearing strata and engineers, aware of the problem, will often base pile design on end bearing values only. Fault-zones comprising shattered rock and clay gouge may present problems in terms of low bearing capacity and excessive differential settlements and the location of these zones should be part of any site investigation and taken account of in planning and design.

Sulphate concentrations recorded in this study fall into Class 1, and occasionally Class 2, of the BRE classification (BRE Digest 250, 1981). However, as noted previously, locally high groundwater sulphate concentrations have been recorded in the Upper Carboniferous rocks elsewhere, particularly in the vicinity of colliery tips, and special measures may be required at some locations to avoid attack on buried concrete.

ii) Slope stability

A few, generally shallow, dormant debris slides/flows and rotational slumps have been recognised during the geological mapping; the landslip deposits generally occuring on or near the middle-lower parts of slopes, in excess of c 8 - 10 degrees, in weathered mudstone below a coal seam and/or a capping sandstone. Minor, shallow slope movements (too small to be indicated as individual landslips on the geological maps, but identified from the presence of low arcuate scars above hummocky ground) are probable fairly common in the weathered mantle and/or Head deposits on such slopes. These movements are relatively isolated and pose no serious problems to development. They are discussed in more detail on pages 34 and 49.

Temporary excavations in fresh to moderately weathered mudrocks should be stable but slumping may occur in the highly weathered zone or the completely weathered clay mantle. The mudstones are highly susceptible to deterioration and softening when relieved of overburden pressure and in the presence of water. Siltstone and sandstone beds may give rise to water seepages and accelerated deterioration of cut faces by rain and heavy construction traffic may exacerbate these processes. To prevent deterioration in open excavations, the mudrock should be covered with a layer of blinding concrete as soon as formation level is reached; temporary cut slopes may need protection by spraying with bitumen or covering with plastic sheeting or tarpaulins. Side slopes of 1V:2H are recommended for preliminary design of excavations and cuttings but perched water tables may present considerable difficulties and in some cases regrading of cut slopes to 1V:4H with suitable drainage may be required.

iii) Excavatibility and use as fill

The mudrocks can be readily excavated by mechanical scraping or digging, but ripping or pneumatic breakers may be required at depth or for major excavations. Although successfully used as embankment fill, the material should be placed as soon as possible after excavation and subjected to minimum construction traffic when wet. Soaked and unsoaked CBR test results confirm the sensitivity of mudrock fill to compaction moisture contents. Remoulded undrained shear strengths of 150 to 350 kPa have been quoted in embankment design for material compacted at *in situ* moisture contents between 15 and 22 per cent, but these values may reduce markedly with an increase in moisture content of only 2 per cent of dry weight in wet conditions. Checks on 'field' moisture contents should therefore be maintained during construction. Plastic limit versus moisture content plots for *in situ* material indicate that near surface, highly weathered mudstones/clays may be classified as cohesive for compaction purposes. Less weathered mudstones, below about 6 to 7 m depth, are generally classified as dry cohesive.

5. Engineering Geology of the Superficial (Drift) Deposits

The superficial, or drift, deposits of the study have been divided into five geotechnical units. The relationship of these units to the geological divisions of the drift deposits are shown below:

Engineering Geological Units	Geological Deposit
1. Made ground	Made ground
2. Mixed soft-firm cohesive / loose-medium dense non-cohesive soils	i) Head ii) Alluvium
3. Mixed stiff cohesive / dense non-cohesive soils	Till (Boulder clay, moundy glacial deposits, glaciofluvial sand and gravel)
4. Non-cohesive sand and gravel deposits	Terrace deposits
5. Landslip	Landslip

Geotechnical units 2 and 3 (and in some areas unit 4) consist of variable and/or mixed deposits of clays and non-cohesive sands and gravels of contrasting engineering characteristics. However, interdigitation and impersistent layering and lensing of these lithologies makes it virtually impossible to refine their characteristics by geological mapping. The geotechnical units are, therefore, selected to reflect this variability in the superficial (drift) deposits as shown on the geological maps.

1. Made Ground

Made ground is ubiquitous in many parts of the Leeds urban area. Reflecting the industrial and residential development of the region, these areas, some of which have been subject to extensive landscaping, are mainly:

- Infilled beck valleys
- Made up ground levels on low lying, marshy or soft ground which was often prone to flooding
- Backfilled sludge lagoons
- Backfilled quarries, clay-pits and opencast sites
- Demolished areas of (predominantly terraced) housing stock
- Demolished large industrial sites

The distribution of existing made ground, generally over 3 m thick, is shown on Map 4 in the *BGS Report* WA/92/1. As an engineering geology unit, made ground, also known as fill or backfill, is variable in composition, geotechnical properties and thickness (1-2 m thicknesses are common but locally, in areas such as old quarries and opencast sites, thicknesses of 10 to over 20 m occur). In so far as records exist, the nature of the fill material and dates of re-soiling for the various made ground sites are presented in *BGS Report* WA/92/1. Infill materials at these sites may consist of chemical, mining and quarry waste, domestic refuse, and construction materials such as bricks, concrete and other rubble, in addition to bulk fill derived from the bedrock formations.

Although fill materials show a wide range of engineering properties, properly engineered fills, such as those typically used in the construction of highway embankments, may have excellent engineering properties. Recently-placed domestic refuse would be at the opposite end of the range. It is important that site investigations make it possible to initially classify the fill, in a qualitative manner, in terms of this wide range of behaviour. Classification should be carried out under the headings listed in Table 2. From a classification of this type, deductions can be made about the causes and possible magnitude of settlement of the fill subsequent to construction. Field loading tests should then be carried out to to enable quantitative estimates of settlement to be made with more confidence. On the basis of these estimations, it is useful to distinguish between three different situations related to a)small movements; b) significant movements; and c) very large movements.

Table 2. Qualitative classification of fills (from BRE Digest 274)

Classification	Description
Nature of material	Chemical composition. Organic content. Combustibility. Homogeneity.
Particle size distribution	Coarse soils, less than 35% finer than 0.06 mm; fine soils, more than 35% greater than 0.06 mm (BS 5930: 1981).
Degree of compaction	Largely a function of method of placement: thin layers and heavy compaction - high relative density: high lifts and no compaction - low relative density; end tipped into water - particularly loose condition; Fine grained material transported in suspension and left to settle out produces fill with high moisture content and low undrained shear strength (eg. silted up abandoned dock or tailings lagoon).
Depth	Boundary of filled area. Changes in depth.
Age	Time that has elapsed since placement; If a fill contains domestic refuse, the age of the tipped material may be particularly significant, since the content of domestic refuse has changed considerably over the years. During the last 40 years the ash content has decreased whilst the paper and rag content has increased. The proportion of metal and glass in domestic refuse has also increased. It may be that more recent refuse will be a much poorer foundation material than older refuse not only because there has been a shorter time for settlement to occur, but also because the content of material which can corrode or decompose is greater.
Water table	Does one exist within the fill? Do fluctuations in level occur? After opencast mining a water table may slowly re-establish itself within the fill.

a) <u>Small movements</u> (vertical compression of the fill subsequent to construction everywhere smaller than 0.5 per cent). This is likely with granular fill that has been placed under controlled conditions and received adequate compaction. Such a material forms a good founding medium and there should be few problems.

b) Significant movements (vertical compression of the fill subsequent to building estimated to have a maximum value between 0.5 and 2 per cent). A granular fill placed without compaction, with little organic matter and which has been in place for some years, may come into this category. In this situation special attention must be given to foundation design. If piling is considered to be uneconomic, basic alternatives are either to use some ground treatment technique to improve the load carrying properties of the fill or to design the foundations to withstand differential movements caused by settlement. Reinforced concrete rafts with edge beams have been used for two-storey dwellings, but where large differential settlements are likely, very substantial foundations may be needed and buildings should be kept small and simple in plan.

c) Very large movements (vertical compression of the fill subsequent to building estimated to exceed 2%). This category may include recently-placed domestic refuse with high organic content liable to decay and decomposition, and fine-grained materials which have been discharged into lagoons, as a suspension, to form highly compressible cohesive fill which might be susceptible to liquefaction. Severe settlement problems are likely and ground improvement techniques may have limited effect. With recently-placed domestic refuse methane emission may also be a major problem and such sites may be prohibitively expensive to develop.

Insufficient data are available to enable a detailed geotechnical assessment to be carried out for the made ground areas in this study, but the general problems and implications for development on made ground are discussed further on p. 35.

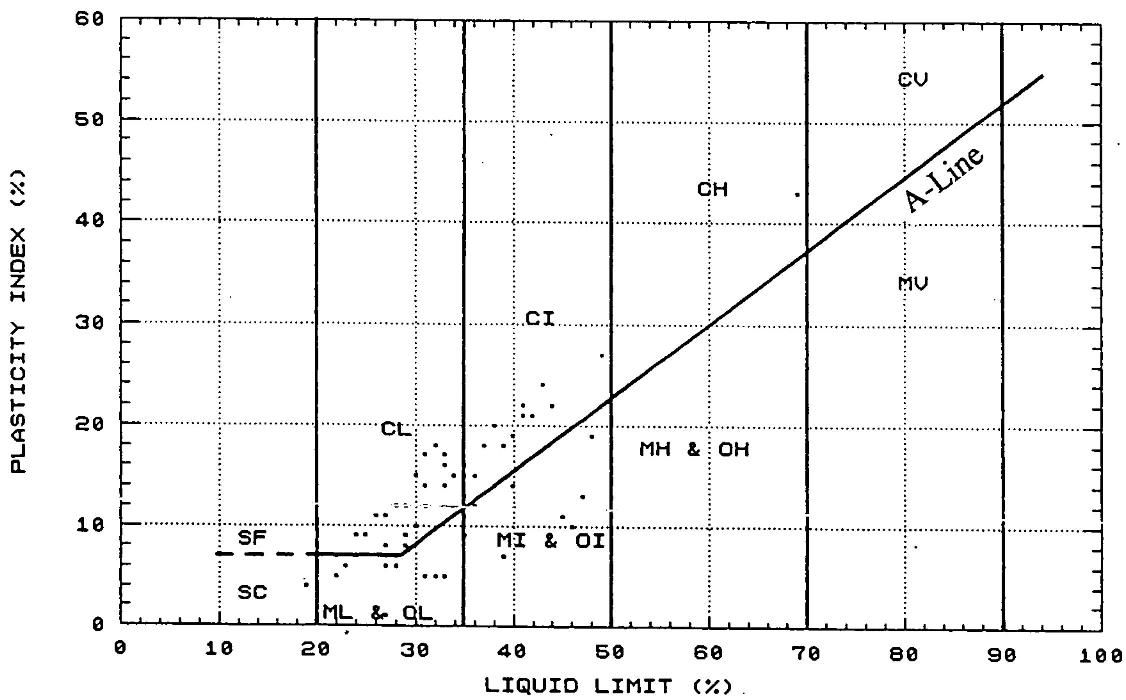
2. Mixed Soft-Firm Cohesive / Loose-Medium Dense Non-Cohesive Soils

i) Head

Head deposits are superficial materials derived from bedrock and other superficial deposits by the action of periglacial freeze-thaw processes and hillcreep, solifluction and gelifluction. Such deposits are variable but the most common lithology is that of a a soft to firm sandy, silty clay with stones; locally it may be a gravelly silty sand or clayey sandy gravel. The variable composition reflects the local derivation. Head commonly forms an extensive cover on the lower slopes of all the exposed bedrock formations, usually as a thin veneer less than a metre thick, but greater thicknesses may have accumulated at the foot of slopes and in hollows. Thicknesses ranging from less than 1 m to about 6 m (and occasionally more) are recorded in many site investigation reports, but the distinction between *in situ* soil (derived from completely weathered bedrock) and Head is generally unclear, the one grading into the other. On the geological maps, the distribution of Head generally greater than 1 m thick is indicated, but this is not fully representative. It should be assumed to occur elsewhere as a patchy veneer of varying thickness, which should be ascertained, and its geotechnical characteristics established, prior to the design of foundations or cut slopes.

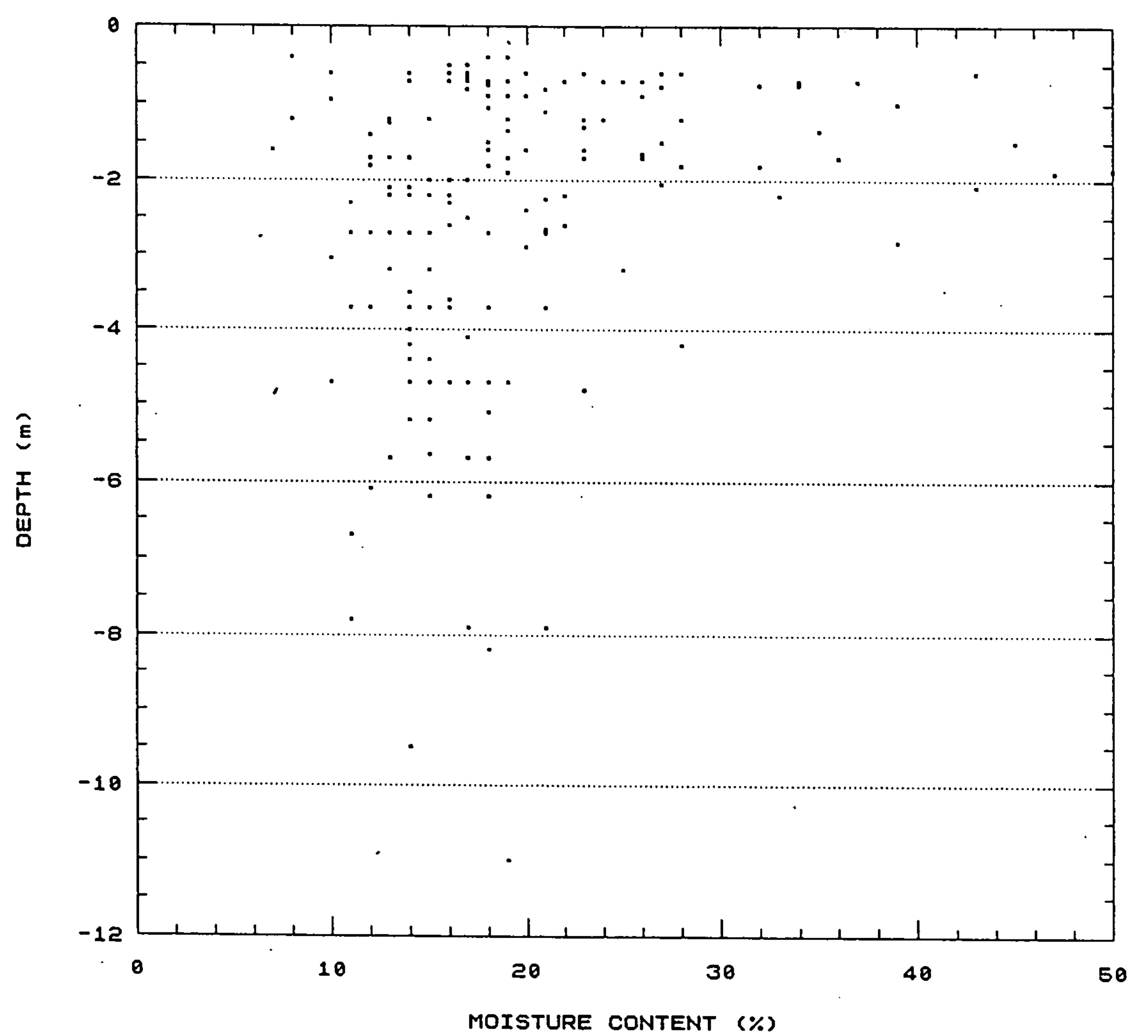
Summary geotechnical properties for all Head deposits are shown in Appendix I, Table 8. No distinction has been made between the Head developed over the Upper Carboniferous and Magnesian Limestone lithologies owing to the broad overlap of recorded parameter values and the substantial statistical bias towards data obtained for the Head developed over the Upper Carboniferous rocks, which crop out over the major part of the study area.

Fig. 3. Plasticity diagram for Head deposits



Plasticity data (Fig. 3) indicate that the Head deposits can be classified as sandy or silty (sometimes very silty) clays and clayey silts of low to intermediate plasticity. The absence of high plasticity clays is unusual as the weathered mudrocks typically degrade to highly plastic clays: clays of high to very high plasticity are recorded in Head deposits overlying Coal Measures rocks in the nearby Castleford-Pontefract area (*British Geological Survey Technical Report* WA/90/3). In the Leeds area, the predominance of low to medium plastic clays may reflect the extensive development of arenaceous beds in the Upper Carboniferous bedrock formations and their potential as a source material for Head deposits.

Fig. 4. Profile of moisture content versus depth for Head deposits



A profile of moisture content versus depth for the Head deposits (Fig. 4) indicates a sharp decrease in the range of recorded moisture contents from 10 - 50 per cent (from ground surface to about 3 m) to 10 - 20 per cent (which remains fairly constant from 3 m to the maximum recorded depth of Head below ground surface, about 8 to 10 m). This may partly reflect the difficulty of distinguishing Head from *in situ* residual soil which commonly shows a gradual decrease in moisture content with depth, reaching an average of about 16 per cent at the weathered bedrock surface. In contrast to the weathered *in situ* soils, Head deposits tend to show more variable and generally higher compressibilities, more variable rates of consolidation settlement, and generally low shear strengths.

Design considerations

i) Foundations

The presence of soft, highly compressible zones giving rise to low bearing capacities and excessive differential settlements, and the ponding of water in depressions (locally caused by mining subsidence) due to effectively impermeable clays, present the main problems for foundation design on Head deposits. In general, weathered bedrock lies within about 1.5 m of the ground surface and where feasible the Head should be removed prior to placing foundations or, where thicknesses are excessive, loads transferred to sound underlying bedrock by piling. For shallow foundations on dominantly clayey Head, the loading criteria shown in Table 3 may be used as a guide.

Table 3. Loading criteria for shallow foundations in Head deposits

Foundation Type	Depth (m)	Nett Safe Bearing Capacity (kPa)	Factor of Safety	Differential Settlement
0.5m STRIP	1.0	100	3	19 mm
0.5m STRIP	1.0	150	2	29 mm
RAFT	0.5	100	3	40 mm
RAFT	0.5	150	2	60 mm

The recorded sulphate concentrations indicate that no special precautions are normally needed to prevent sulphate attack on concrete. However, as for the Upper Carboniferous rocks, increased groundwater sulphate contents may be encountered in the vicinity of colliery waste tips and checks at specific sites should be undertaken during site investigations.

ii) Slope stability

Minor natural slope failures, often comprising areas of hummocky ground, may occur in the Head deposits. Such movements are too small and isolated to show on the geological maps. Even on low angle slopes, Head deposits may also contain relic shear surfaces (resulting from solifluction and multiple sliding movements during its formation under periglacial conditions). Shear strengths on these surfaces will be at or near residual values and this should be accounted for in slope design calculations. Cutting side slopes of 1V:2H are suggested as a guide for preliminary design in Head with no identified shear surfaces.

iii) Excavatibility and suitability as fill

Head deposits may be easily excavated with normal soft ground excavating plant. Problems may be encountered locally due to ponding of water in depressions where impermeable clayey Head is present. Head may be suitable for use as bulk fill, but locally may be too wet to achieve satisfactory compaction. No CBR data were obtained for Head deposits in the area but values may be expected to increase if compacted at optimum moisture contents (ranging from about 10 to 16 per cent) and to reduce considerably as moisture contents increase above optimum.

ii) Alluvium

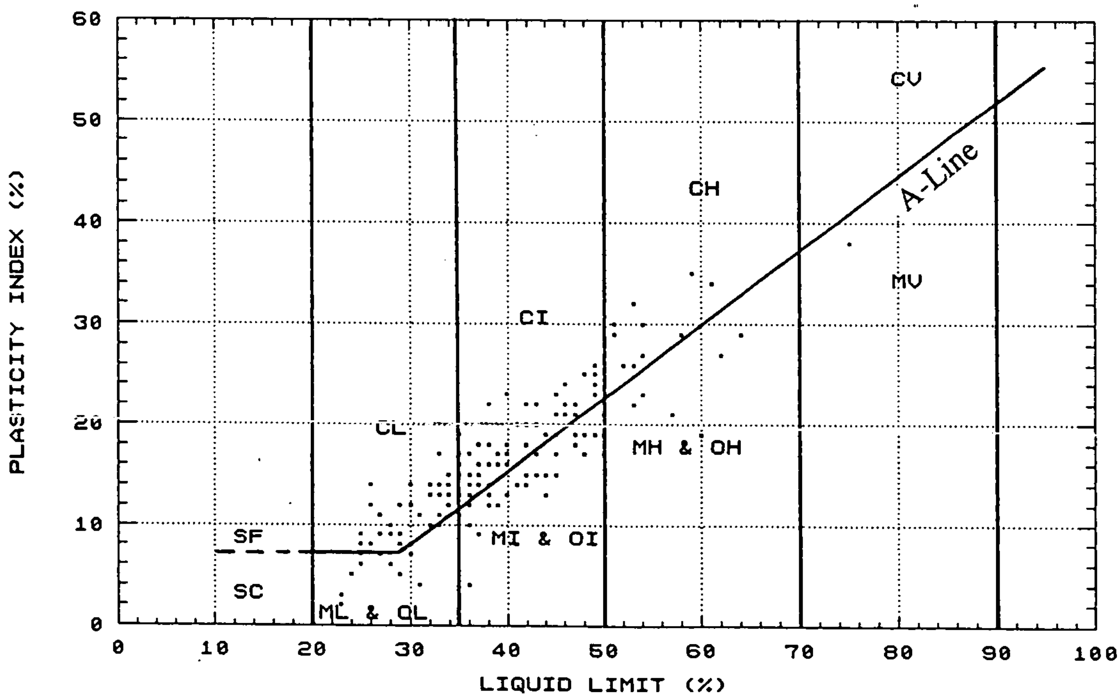
Alluvial deposits occur extensively on the valley-floor of the River Aire, and are also present in minor tributary valleys. Although extremely variable in terms of lithology on both a regional and local scale, and with highly variable geotechnical properties and engineering behaviour, the alluvial deposits may be broadly subdivided as follows:

'Upper Unit'	Brown-grey mottled, soft to firm silty CLAYS and clayey SILTS with local organic (peaty) horizons, and sand and gravel components becoming more common with depth.
'Lower Unit'	Medium dense, fine to coarse-grained SANDS and angular to rounded, fine to coarse-grained GRAVELS with variable clay content and local lenses of organic silt or brown, pebbly, sandy clay.

The depths and thicknesses of the above units vary across the area. In the Aire valley combined thicknesses of up to 10 m are commonly recorded in site investigation reports, but thicknesses of c. 14 m may occur locally (eg. in a meander loop of the River Aire near Rodley sewage works). At many locations the soft upper unit of the alluvium has been removed and replaced with fill. Summary geotechnical properties are presented separately for the two units in Appendix I, Tables 9 - 10, but lithologies (and engineering properties) may vary markedly at specific sites due to local layering and lensing.

Standard penetration test (SPT) N-values for the sands and gravels range widely from below 10 (loose) to over 100 (very dense), the great majority falling within the medium dense/dense range (N-values of 20 to 40) and showing an overall increase with depth. SPT N-values for the silts and clays are consistently below 40 (median 10) with no clear trend of increasing N-values with depth. These consistently low values reflect the low shear strengths of these materials.

Fig. 5. Plasticity diagram for alluvial silts and clays



The plasticity chart for the silts and clays (Fig. 5) indicates materials ranging from low to high plasticity, but zones of organic clay and peat (not recorded in the current database) will have very high plasticities. The silt/clay units may be expected to exhibit medium to very high compressibilities with very slow rates of consolidation settlement, particularly where soft organic clays and peat are present.

Design Considerations

i) Foundations

Low bearing capacities (undrained cohesion commonly around 20 to 40 kPa), high compressibilities, high groundwater tables and water uplift pressures, and the likelihood of excessive total and differential settlements pose problems for foundations in the alluvium. Where limited thicknesses occur, wholesale removal and replacement of the alluvium with suitable fill may be an economic option, but elsewhere alternative solutions are required. Of these, piling and raft foundations are most commonly used in the Leeds area. For heavy structures, bored or driven piles should be used to transfer loads to the basal gravel or into the underlying bedrock. Mini-piles bearing in the granular alluvial deposits should be suitable for light to moderately loaded structures. For piles in thicker alluvial deposits, consolidation of the clays may cause 'drag-down' of the loaded pile and this should be anticipated. Raft foundations have been successfully employed for light structures with large settlements accounted for in design.

Maximum embankment heights of about 4 m with 1V:2H side slopes have been suggested for construction on alluvium. Very low rates of consolidation settlement may be partly overcome by the use of lightweight embankment fill or staged surcharging, for both embankments and light structures. Monitoring of settlements during construction should be undertaken and the likelihood of differential settlement accounted for in pavement design.

Due to the artificial straightening of the River Aire channel, abandoned meanders occur at several locations. It is important that these features (which may contain soft, compressible organic clays and silts hidden beneath made ground) are identified during site investigations prior to planned development.

The recorded groundwater sulphate concentrations in Class 2 of the BRE classification indicate that care is needed in the selection of buried concrete.

ii) Slope stability

High groundwater levels mean that excavations in the alluvium are subject to severe water inflow problems and immediate support is normally required to maintain the stability of trench sides and cut faces. Running sands may also be encountered below the water table.

iii) Excavatibility and suitability as fill

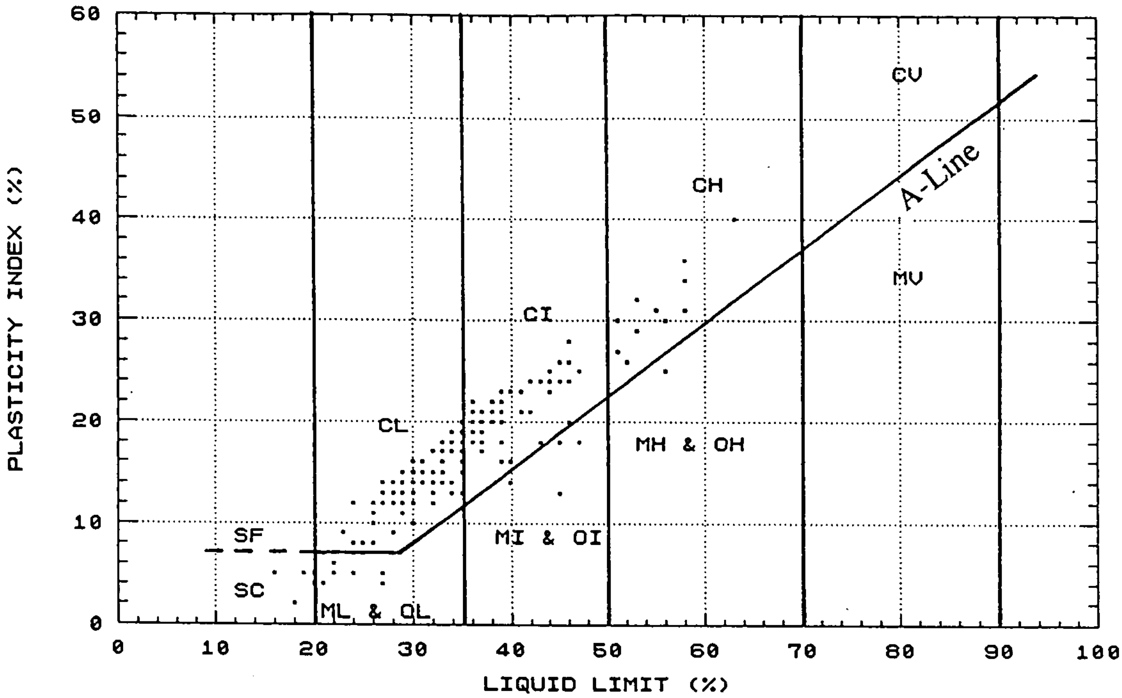
Alluvial deposits are readily excavated using normal soft ground excavating plant but severe water inflow problems may be encountered during working. These deposits are generally unsuitable as fill material.

3. Mixed Stiff Cohesive / Dense Non-cohesive Soils

This engineering geological unit comprises glacially derived material collectively termed 'till' on the geological maps. The till in the area is lithologically variable, and includes dominantly boulder clay, but with glaciofluvial sandy gravels and, at some localities, laminated silts and clays. Moundy glacial deposits (morainic drift) occur as low, elongated mounds over a small area in the Aire valley upstream from Leeds. No geotechnical data were obtained for these last deposits and no engineering assessment of them is made here. It is likely, however, that they were never subjected to substantial ice loading and, as such, their engineering behaviour may be more akin to gravelly clayey Head rather than an overconsolidated boulder clay.

Boulder clay, found mainly on the higher ground, typically comprises brown, stiff to very stiff, stony, sandy clay with clasts of sandstone, chert and ironstone. Limestone clasts are present where it lies close to the Magnesian Limestone outcrop. The deposit is often variable and lenses of sand, intercalations of laminated clays and interbedded sandy gravels may occur locally. Summary geotechnical data for the boulder clay (till) is shown in Appendix I, Table 11. SPT results mainly range from 15 to over 60 (median 31), with N-values increasing with depth. Depending on the sand/silt content, the till matrix consists of clays ranging from low to high plasticity (Fig. 6), with low compressibility and generally high shear strength (typical undrained cohesions of around 100 kPa and angle of shearing resistance 10°).

Fig. 6. Plasticity diagram for till (boulder clay)



Glaciofluvial sands and gravels may grade laterally and vertically into boulder clay. They range from slightly cohesive, clayey silty sands to highly permeable, dense/very dense non-cohesive sandy gravels with layers and lenses of silt and clay developed locally. Summary geotechnical data for these deposits are shown in Appendix I, Table 12. SPT N-values rarely fall below 30 and show a general, but erratic, increase with depth which possibly reflects variable clay contents and local silt/clay layering and lensing.

Insufficient data were obtained to tabulate summary geotechnical results for laminated glacial/glaciolacustrine silts and clays. Limited SPT results recorded N-values ranging from 14 to 40 with a median value of 25.

Design Considerations

i) Foundations

Till should present no major problems for shallow foundations provided lithological variations are determined during site investigations (particularly the presence of laminated silts and clays) and potential differential settlements are accounted for in design.

ii) Slope stability

Temporary cuts or excavations in 'homogeneous' boulder clay should remain stable, but the presence of layers or lenses of laminated clays may necessitate side support. Bands of silt and sandy gravel may result in perched water tables and seepage and, where exposed, excavations will require support. Cut slopes of 1V:2.5H have been recommended for long-term stability in relatively homogeneous boulder clay. Excavations in glaciofluvial sandy gravels will require immediate support and where water-bearing, measures to regulate water ingress will be required.

ii) Excavatibility and suitability as fill

Till may be easily machine-excavated but ponding of surface water in low permeability boulder clay may cause problems during working. The laminated clays may prove unsuitable for use as fill, as may boulder clay occurring near water-bearing beds of sand and gravel.

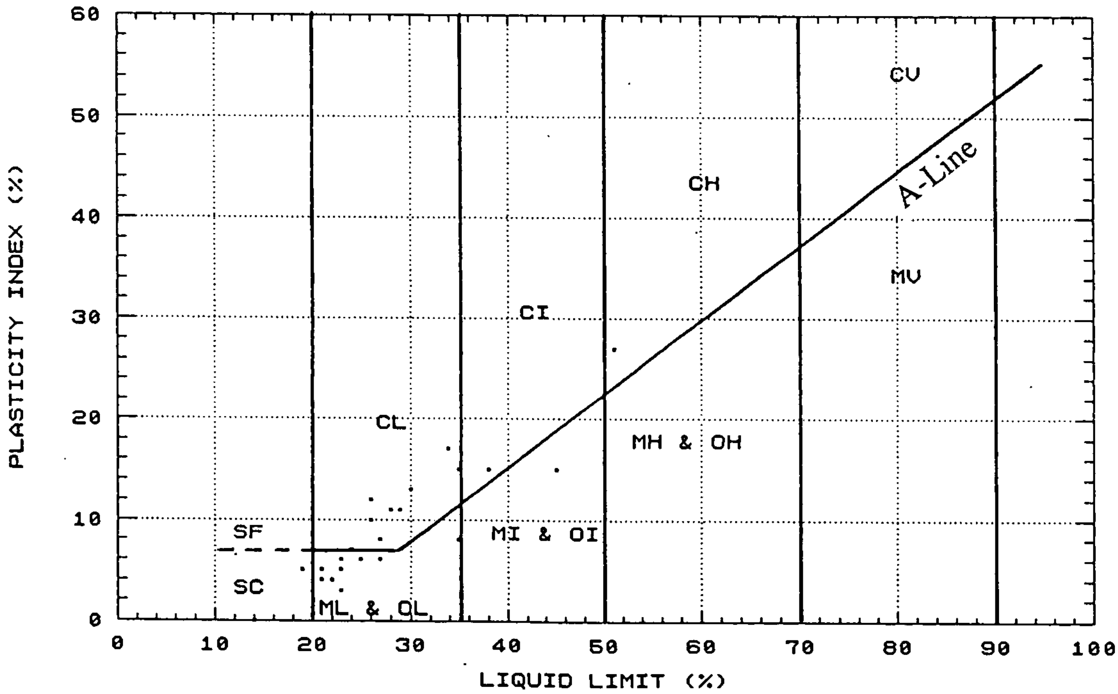
4. Non-cohesive Sand and Gravel Deposits

This unit comprises river terrace deposits, which occur as irregular but often elongated, impersistent low planar features along the Aire valley immediately upslope from the alluvium. The surface level of these deposits lies about 5 m above the alluvium (c 25 m above OD) and the surface climbs upslope imperceptibly to merge with hillwash and/or Head deposits on the lower Aire valley-slopes.

The terrace deposits consist mainly of medium dense, fine to coarse-grained sands and medium dense to dense, sandy, sub-rounded to rounded, fine to medium gravels with occasional cobbles. The gravel fraction is mainly of locally-derived Carboniferous sandstone with occasional ironstone, shale and limestone. Although these granular materials predominate, sandy clays and silts (in places, laminated or with included gravel) occur locally (eg. in the Hunslet area), typically as a surface veneer over the sands and gravels and/or at the margins of the terraces where they merge imperceptibly with weathered clayey slope deposits (hillwash/Head). In these marginal areas the terrace deposits are sufficiently clayey to have been worked for brick-making.

Summary geotechnical properties for the terrace deposits are shown in Appendix I, Table 13. For the granular materials, SPT N-values fall mainly between 10 to 50 (medium dense to dense) but loose deposits (N-values of 10 or less) are often found in the topmost 3 m below ground level. Limited plasticity data show the less dominant, fine-grained terrace deposits to consist mainly of low plasticity clays, silty clays and clayey silts, with some clays of intermediate plasticity (Fig. 7). These materials can be expected to show low to moderate compressibilities and moderate shear strengths (typical undrained cohesion of about 50 kPa, and angle of shearing resistance 9 degrees). Horizontal shear strengths may, however, be much lower in laminated silts and clays.

Fig. 7. Plasticity diagram for fine-grained terrace deposits



Design Considerations

i) Foundations

Provided lithological variations are identified, the terrace deposits should pose no major problem for most foundations. The deposits are well-drained, and shallow foundations should be above the water table. Care should be taken in some areas (eg. near Hunslet) to identify and ascertain the depth and lithological composition of infilled channels below the terrace deposits, particularly during site investigations for heavier structures. Recorded groundwater sulphate concentrations generally fall into Class 1 of the BRE classification, indicating that no special precautions are normally required to prevent attack on buried concrete. However, checks should be made at specific sites during ground investigations.

ii) Slope stability

Excavations and cut faces will require immediate support and casing will be required to prevent collapse of granular material into bores. Water ingress should not present problems unless borings or excavations into channel infill are taken below river level (as hydraulic continuity may be anticipated).

iii) Excavatibility and suitability as fill

The terrace deposits may be readily excavated by machine digging. The sands and gravels should be suitable as granular fill but the terrace clays and silts are probably unsuitable for fill purposes.

5. Landslip Deposits

The topography of the study area is generally low and subdued, and natural landslipping is not widespread. A few, generally shallow, dormant debris slides/flows and rotational slumps have been recognised during the geological mapping; the landslip deposits generally occuring on or near the middle-lower parts of slopes, in excess of c 8 - 10 degrees, in weathered mudstone below a coal seam and/or a capping sandstone (eg. near Meanwood Grove, GR 2815 3842). Minor, shallow slope movements (too small to be indicated as individual landslips on the geological maps, but identified from the presence of low arcuate scars above hummocky ground) are probable fairly common in the weathered mantle and/or Head deposits on such slopes (eg. at Farnley, north of Billey Lane, GR 2552 3201).

These movements are relatively isolated and pose no serious problems to development, however, where present, construction on these areas should be avoided wherever possible.

Remoulded clayey landslip debris is generally soft, poorly drained and may contain perched water tables. It also contains one or more shear surfaces, along which shear strengths are at or close to residual values. Construction activity (including excavations) on these deposits, even if the landslip is dormant and degraded, is likely to reactivate movement. If development on these areas is unavoidable, it is essential that site investigations are undertaken to adequately define the geometry and current stability of the landslip (and adjacent ground) in order that suitable stabilisation measures can be designed and implemented prior to construction. In some cases, small areas of landslip deposits may be removed completely and replaced with a free-draining granular fill. However, the mechanisms leading to initial failure should be ascertained and the likelihood of future potential movements in adjacent ground investigated.

6. Engineering Geological Implications for Planning and Development

The main problems and potential hazards to future planning and development in the Leeds area can be grouped under three broad categories:

A. MADE GROUND

B. MINING

C. NATURAL GROUND CONDITIONS

A. MADE GROUND

The distribution of the principal areas of made ground is shown on Map 4 in *BGS Report* WA/92/1 and, in so far as records exist, the report also catalogues the nature, maximum thickness and dates of re-soiling of the fill materials. The areas of existing made ground are mainly:

- Infilled beck valleys
- Made up ground levels on low lying, marshy or soft ground which was often prone to flooding
- Backfilled sludge lagoons
- Backfilled quarries, sand and gravel pits, clay-pits and opencast sites
- Demolished large industrial sites
- Demolished areas of (predominantly terraced) housing stock

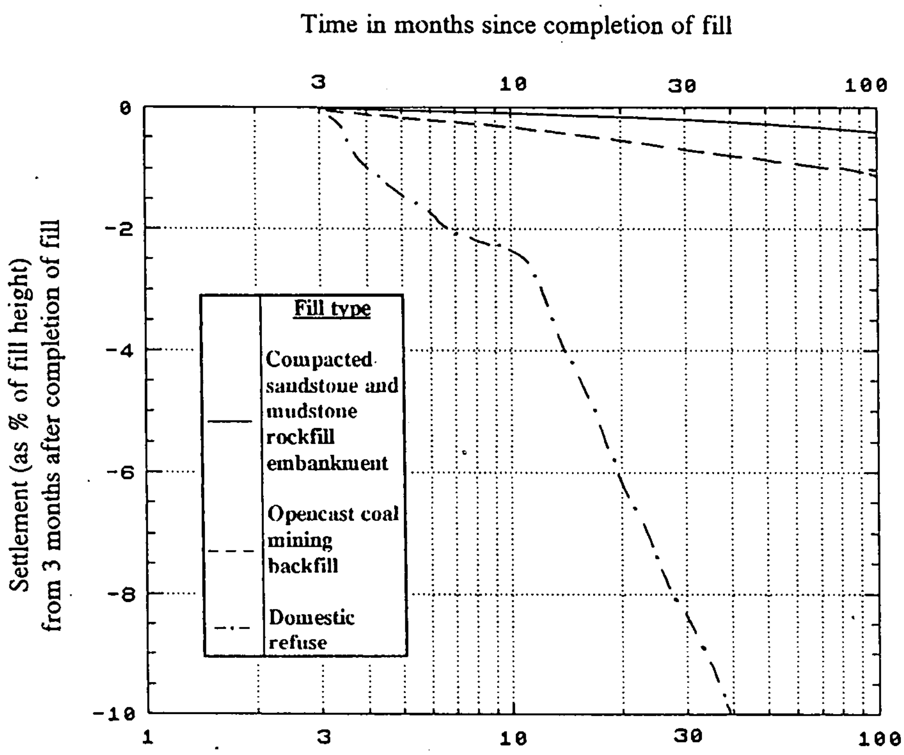
Serious problems can arise where construction takes place on filled sites, and all made ground should be treated as suspect because of the likelihood of extreme variability in the composition of the fill materials (some of which may be hazardous to health or harmful to the environment or building) and their compaction/settlement characteristics. Not all made ground will present problems, as some of the fill material will be inert and possibly well-compacted, but the following points should be borne in mind:

a) where fill has been placed above ground level, as in infilled becks and made up levels of low marshy ground, it may rest on soft alluvial clays or peats which may themselves undergo excessive differential consolidation settlement when loaded; this will be additional to that resulting from the compression of the fill itself. The stability of the ground underlying the fill may, therefore, need to be examined.

b) if structures are built on piles passing through the fill into underlying strata, negative skin friction caused by the fill settling under its own weight may be a major consideration in foundation design.

c) where the fill is deep, self-weight will often be the principle cause of long-term settlement. With granular fills and poorly compacted unsaturated fills of all types, the major compression occurs almost immediately and consequently most of the settlement due to self-weight occurs as the fill is placed. Nevertheless, significant further movements ('creep' settlement) do occur under conditions of <u>constant</u> effective stress and moisture content. With many fills the rate of creep compression decreases fairly rapidly with time and shows an approximately linear relationship when plotted against the logarithm of time elapsed since the deposit was formed (Fig. 8). It should be remembered that an increase in stress due to changes in applied load or moisture content may cause much greater movements than those shown in Fig. 8.

Fig. 8 Settlement rates of different types of fill (vertical compression plotted against $\log_{10}$time). [from BRE Digest 274]



d) compression of fills by building loads will be very variable depending on the nature of the fill, its particle size distribution, compactness, the existing stress level, the stress increment and the moisture content. Assuming the stress increments due to building loads do not bring the fill close to bearing capacity failure, settlements can be most simply calculated using a compressibility parameter related to one-dimensional compression. This parameter, the constrained modulus, is defined as $\Delta\rho_v$ / $\Delta\varepsilon_v$, where $\Delta\rho_v$ is an increment of vertical stress, and $\Delta\varepsilon_v$ is the increase in vertical strain produced by $\Delta\rho_v$. Some typical values of constrained modulus for different fill types are shown in Table 4. It should be realised that movements which occur during building construction are likely to be much less of a problem than those which occur after completion of the structure. The long-term creep component is therefore of particular significance (see 'c' above).

Table 4. Compressibility of fills (from BRE Digest 274)

Fill type	Compressibility	Typical value of constrained modulus (kN/m2)
Dense well-graded sand and gravel	very low	40 000
Dense well-graded sandstone rockfill	low	15 000
Loose well-graded sand and gravel	medium	4 000
Old urban fill	medium	4 000
Uncompacted stiff clay fill above the water table	medium	4 000
Loose well-graded sandstone rockfill	high	2 000
Poorly compacted colliery spoil	high	2 000
Old domestic refuse	high	1 000 - 2 000
Recent domestic refuse	very high	--

e) loose unsaturated fill materials are usually liable to collapse settlement following inundation with water. If this occurs after construction, a serious settlement problem may arise. It is believed that this is often a major cause of settlement problems in building development on restored opencast mining sites. Problems can also be caused by water penetrating into the fill from the surface through deep trench excavations for drains associated with building development.

f) when fine material is placed underwater, as in sludge lagoons, a soft cohesive fill is formed which is characterised by low permeability. Settlement is controlled by a consolidation process in which excess porewater pressures dissipate slowly as water is squeezed out of the voids in the fill. This type of fill may be susceptible to liquefaction, and often a firm but thin crust, overlying very soft material, may form over the surface of the lagoon deposit.

g) excessive differential settlements, leading to distortion and damage to buildings, are to be expected in highly variable poorer types of fill, or where the depth of fill changes rapidly. Where the made ground backfills a pit or quarry, it will invariably be less compacted than the surrounding natural ground, leading to differential settlement in foundations that straddle the edge of the filled area.

h) at sites in areas of demolished industrial buildings and housing, load-bearing walls may be present at or close to the surface of the fill. New foundations built across such walls and the surrounding fill are liable to severe differential settlements. Such sites may also contain basements, cellars, tunnels and service ducts which may be only partially filled with rubble and often remain as complete voids.

i) the problem of differential settlement may be compounded in cases where cavitation results from chemical or bacterial breakdown of the fill material.

j) fill comprising industrial waste may be potentially chemically active and capable of generating dangerous or combustible gases and toxic leachates.

k) domestic refuse will generate methane gas, which is highly combustible.

l) colliery spoil, which probably constitutes the greatest part of the made ground in this area, may be liable to spontaneous combustion, and tends to produce sulphate-rich acidic groundwater leachates causing corrosion problems in buried foundations. The leachates may also affect the quality of surface water.

B. MINING

1) Opencast sites

These sites may be considered to fall under the category of 'made ground', and the implications for planning and development are as discussed in section A.

2) Bell pits, mine shafts (and well shafts)

Early mining of coal (and ironstone) was carried out from bell pits, initiated from shafts about 1 metre in diameter and up to 10 metres deep. The working of coal from the base of the shaft created a bell-shaped void, with an unsupported roof to the coal seam. The excavated area of coal was roughly circular in plan and up to 6 metres in diameter. As examples, the Middleton Main Coal was worked in this fashion near Hunslet Carr and Halton (the latter still visible).

In many cases the bell pits were only partly backfilled and the present state of compaction is almost certainly very variable. The fill materials usually consist of silty clay and moderately to completely weathered mudstone fragments with traces of coal and/or ironstone nodules. Rotting timbers are sometimes encountered. Weathering of pyrite in the fill may result in high levels of sulphate and acid conditions, but the limited chemical tests obtained in this study have not substantiated this.

Although bell pits have usually been backfilled to a greater or lesser degree, mine shafts have often been left empty. When shafts are backfilled, a poor state of compaction nearly always exists in the highly variable fill materials, and voids are common. Even when a mine or well shaft has been capped within the last 40 to 50 years, voids may not have been filled and old caps may be in a poor condition (although a number of mine shafts have been capped to standards issued by British Coal over the last 50 years).

Collapse of shaft caps over empty shafts, collapse of the shaft fill leaving a void, and collapse of shaft linings into a void resulting in surrounding superficial deposits being drawn into the shaft are all potential threats to structural development. Building over shafts can result in severe differential settlement problems and, at worst, catastrophic ground failure is possible. Building over or close to shafts should be avoided, even though some structures have existed over such sites for centuries without damage (for example, Mount St. Mary's Church, Richmond Hill).

When located, shafts should be filled with pea gravel, grouted and capped. In bell pits, grouting may be unsuccessful in the ancient, generally cohesive backfill. Treatment by vibro-replacement to support light structures is possible, but some differential settlements should still be anticipated. If cost-effective, it may, in some cases, be worthwhile to opencast sites occupied by bell pits. Foundations can then be placed on underlying rock, or on overburden following its replacement in well-compacted layers.

In addition to adverse settlement characteristics, old shafts may also provide potential problems as pathways for noxious or explosive gases, or polluted groundwater.

It is essential that efforts are directed to locate the sites of old bell pits and mine/well shafts prior to construction and development. This is not always straightforward. Very few site descriptions given in site investigation reports for the Leeds area have noted the presence of open pits or hollow depressions in areas of old bell pit workings. This may be largely due to these 'tell-tale' features being obscured by centuries of agricultural, and other, activities. Approximate locations of shafts can, to some degree, be obtained from various editions of Ordnance Survey topographic maps, British Coal's shaft register, Leeds City Council (Architects Department), and the Inland Revenue Valuation Office. British Coal retains records of shafts properly filled and capped to prevent collapse. Where old bell pits or shafts are suspected, and no records exist, site investigations should consider and employ a combination of the following elements:

- Aerial photography - the study and analysis of which is often very successful in identifying anomalous tonal features indicative of old shafts and pits.

- Geophysical surveys (electromagnetic, magnetic and cross-hole seismic surveys and ground probing radar) - these may be very successful where shafts are empty, contain significant voids, or where backfill material is significantly different to the natural ground. Less success is likely where insufficient contrast exists between the natural ground and the infill material (this is particularly likely in the case of bell pits). Useful guidance on the geophysical techniques appropriate to the location of shafts and mineworkings, located at different depths and overlain by varying thicknesses of fill cover, is given in the Geological Society Working Party Report on Engineering Geophysics (Anon, 1988).

- Trial pitting/trenching - this should be carried out as a matter of course, both to confirm the presence of ground anomalies identified from aerial photography or geophysical methods. Where the fill cover is thin (<5 m) pitting and trenching may be employed as a relatively inexpensive alternative to a geophysical survey.

- Boreholes - boreholes are an inefficient and expensive method of locating shafts, but once located, boring is essential to prove depths of shafts and composition and consistency of the backfill. If shell and auger tools meet suspected obstructions within the fill, the holes should be completed by rotary open-holing until the full shaft depth, and the presence or otherwise of voids, are ascertained.

3) Shallow mine workings: Pillar-and-stall (or stoop and room) workings.

During the 15th and 16th centuries, the pillar-and-stall method of mining was developed to gain access to deeper reserves of coal. With this system, widely employed in the 18th and early 19th century, larger pillars are required to support the roof with increasing depth and thus it was self-limiting in terms of economic usage. In West Yorkshire the maximum feasible depth may have been about 30 metres (Giles, 1988). General dimensions of these workings (Bell, 1978) are summarised below:

- Workings penetrated only about 40 m from the shaft in the 15th century, increasing to about 200 m by the 17th century.
- Room size increased with time from, usually, less than 2 m wide in the 15th century to a range of 1.8 to 5.0 m in the 19th century.
- Extraction ratio ranged from 30 to 70 per cent, showing a general increase with time.

In Leeds, the depth to the roof in uncollapsed workings is rarely less than 6 to 10 m, but in the vicinity of Mount St. Mary's Primary and Secondary Schools and Church, workings were encountered at depths of only 3 m below ground surface (in the Beeston Coal) due to exceptionally good mining conditions. It is probable, therefore, that the whole of Richmond Hill, where this seam occurs, is honeycombed with these workings (Hollands, 1990). The old workings may be dry or flooded.

These shallow workings did not totally collapse when mining ceased. They thus pose serious problems for new construction work and are probably the single most common cause of structural damage in the Leeds area.

Piggot and Eynon (1978) noted three principal mechanisms of failure of shallow mine workings:

a) squeeze of floor or roof strata

b) pillar failure

c) collapse of roof beds spanning adjacent pillars

Mechanism (a) is a relatively short-term effect unless influenced by changes in groundwater conditions. Mechanism (b), collapse of pillars, is comparatively rare unless the pillars were 'robbed' at a later date. Pillar collapse may result from deterioration due to weathering, but is a rare occurence. Also, material from roof collapses provides lateral restraint which tends to enhance the strength of the pillars by 'buttressing'. Pillar collapse can be precipitated by longwall mining below old workings, but as much of the Leeds area lies within the abandoned coalfield, it is unlikely that further deep mining will occur. Pillar failure will tend to occur below about 30 to 50 metres depth where the uniaxial compressive strength of the coal is exceeded by the pressure of overburden. However, as most of the workings are over 100 years old, 'crushing out' of the pillars, which leads to surface settlements, will have ceased in all but a few cases.

Mechanism (c) is a major cause of ground instability problems as the collapse of roof beds spanning adjacent pillars may lead to upward propagation (migration) of a void. This can occur within a few months, or a long period of years after mining has ceased. When void migration occurs, the material involved in the fall 'bulks', which means that the migration is eventually arrested, but the bulked material never completely fills the void. At shallow depth, void migration may continue upwards to the ground surface leading to the possible formation of a crown hole. The height to which a void will migrate can, in general, be estimated by using the following expression (Bell, 1978):

$$H = t\left[\frac{\rho_1}{\rho} \Big/ \left(1 - \frac{\rho_1}{\rho}\right)\right]$$

where, H = height of migration

t = height of original void (usually the thickness of the seam worked)

ρ = bulk density of the roof rocks in place

ρ_1 = bulk density of the collapsed roof materials

Piggot and Eynon (1978) showed that the maximum height of the zone of collapse is commonly up to 6 times the height of the mine void but may, exceptionally, exceed tenfold. In the Leeds area, local engineering experience suggests that between 5 and 7 times the height of the original mine void is a realistic figure (Hollands, 1990). Site investigation procedures in areas where old workings within about 30 m of the ground surface are suspected should include:

- Rotary openholing using air flush to search for voids and collapsed roof rock, and to prove the thickness of coal seams.
- Trial pitting with a suitably powerful excavator if workings are at very shallow depth (less than 6 m).
- Selected cored boreholes are useful to identify the lithologies overlying the workings, and to assess their strength and stability by laboratory testing of core samples. They are also useful to assess stability of pillars.
- Geophysical methods (electrical resistivity soundings, gravity and cross-hole seismic surveys) may also be effective in locating and ascertaining the extent and dimensions of the old workings (Anon, 1988).

Once the old workings are identified and their extent and depth established, a number of options are available for site development and construction:

i) Avoid building over the affected area. This includes a peripheral area of influence, usually defined by projecting an angle of 25 degrees to the vertical from the periphery of the workings at the worked level to the ground surface.

ii) Opencast to extract the remaining coal. Foundations may then be placed on exposed rock at the base of the workings or on replaced and compacted overburden.

iii) Use reinforced raft foundations. Rafts are often the best option for light structures. If crown hole formation is a possibility, rafts sould be designed to span a void at least 2 to 3 m in diameter.

iv) Piling into sound rock below the worked coal seam is generally the best option for heavy structures. Bored, cast *in situ* piles are preferable to driven piles as the latter are prone to destabilise the workings and may be difficult to drive through Coal Measures rocks. Piles are at risk of being buckled or sheared by collapsing strata and appropriate pile diameters and suitable concrete, reinforced over the full pile length, should be used. Bored piles should be sleeved to avoid loss of concrete into voids and to reduce the effect of negative skin friction, should the strata overlying the workings subside.

v) The grouting of old workings and any migrating voids (including the peripheral zone defined by the 25 degree angle of influence) may be a suitable solution for ground improvement, particularly for expensive structures that can not tolerate even small differential settlements. A drawback to this method, however, is that it may be difficult to ensure that all the voids have been grouted. Also, delivery pressures, which are often necessarily high, can lead to ground heave, and a low viscosity grout (to ensure void penetration) may result in a high grout-take into the surrounding rocks if they are are fractured. Further, groundwater, if flowing through the workings, may be dammed by the grout and result in an increase in the head of water, high pore pressures, uplift forces and erosion. It has been suggested by K. Wardell and Partners (unpublished report, quoted in Hollands, 1990) that, unless grout-take has been high, the bearing capacity of the grouted zone is unlikely to exceed 150 kN/m^2. It may, therefore, be necessary to ensure that heavy 'point' loads (eg. from pad foundations) are not sited in or so close to the grouted zone as to induce high settlements or shear failure.

4) Longwall mining

The longwall method of mining, which involves total extraction of a panel between two parallel headings, enabled deeper seams to be mined after its introduction in the 19th century. Subsidence from longwall mining is usually rapid (within days of extraction) and residual subsidence is generally completed, typically, within 2 years of the cessation of coal extraction. Maximum subsidence, up to 80 per cent of the seam thickness, occurs when the width of the panel exceeds 1.4 times the depth of the seam (Healy and Head, 1989).

Longwall mining has ceased in the Leeds district. No further mining is, at present, contemplated and most of the past longwall mining took place at considerable depth in relation to typical foundation levels. In most areas, therefore, the presence of past longwall mining beneath a site is unlikely to compromise the integrity of future structures. In those areas where longwall mining occured at relatively shallow levels (less than about 50 metres below ground surface), site investigation techniques should be the same as for those suggested for pillar and stall workings. However, where the stowed waste ('goaf') is well compacted, it may be difficult to recognise the workings by rotary openholing. In such cases it will be necessary to selectively 'spot' core (ie. openholing to an estimated 1 to 3 m above the suspected level of the workings, followed by core drilling until the workings are fully penetrated).

Where shallow longwall workings exist, roadways may still remain open and there is the possible risk of void migration. Also, the 'goaf' is much weaker than the natural adjacent strata and heavy structures which stress the goaf may experience larger than anticipated settlements. End-bearing piles terminated just above the level of the workings could 'punch through' when loaded. If the repositioning of planned structures away from the affected area is not a viable option, grouting of open roadways should be a relatively simple and successful solution, but the cost of such work should be compared with the use of reinforced foundations (eg. rafts) within the localised area of risk. Where the goaf is likely to be stressed due to heavy foundation loads, allowances should be made in design to cater for additional settlements. Bored piles should be keyed into the floor rock (ie. at or below the base of the workings). Light structures are unlikely to be affected.

C. NATURAL GROUND CONDITIONS

Few serious problems should be experienced in natural ground, particularly for light to moderately loaded structures, provided adequate site investigation is undertaken. However particular difficulties may be encountered in the following situations:

1) Soft ground

Areas of soft ground are encountered in areas of alluvial clays and silts (ie. in the floodplain of the River Aire and its tributary valleys). Due to artificial straightening of the River Aire channel, abandoned meanders (which may contain soft, compressible organic clays and silts hidden beneath made ground) occur at several locations. It is important that these features are identified during site investigations prior to planned development. Poorly-drained, clayey Head deposits are also likely to give rise to soft conditions, but the patchy development and usually limited thickness of these deposits should not cause undue difficulty provided they are identified during site investigations and removed or appropriately accounted for in foundation design prior to construction.

Soft ground is characterised by low shear strengths and bearing capacities, is highly compressible and prone to excessive total and differential settlement when stressed. Where it is economic to do so, wholesale removal and replacement with suitable granular fill should be considered, otherwise special attention must be given to foundation design (eg. piling to sets in underlying gravels or sound bedrock for moderately to heavily loaded structures or, for lighter structures, reinforced concrete rafts with edge beams). For piled foundations in thick alluvial deposits 'drag-down' of the loaded pile by consolidation of the alluvial clays should be anticipated.

2) High groundwater levels in alluvial sands and gravels

Severe water inflow problems will be encountered in excavations in highly permeable alluvial sands and gravels and whenever possible, foundations and services should be placed well above the water table. Diaphram walling, grout injection and ground freezing are possible, but expensive, options for the exclusion of groundwater from excavations. Sheet piles are a less expensive alternative but may prove difficult to drive through dense gravels or gravels with included cobbles and/or boulders. All these techniques need to be applied through the full depth of the deposit. Drawdown of the water table using well point rings may be feasible but may be ineffective due to very high permeabilities. Drawdown may also cause settlement damage to existing nearby structures. Pumping tests should be carried out to assess *in-situ* permeabilities during site investigation.

3) Moisture susceptible mudrocks

Mudstones of the Coal Measures and Millstone Grit 'mudrock' lithologies are susceptible to rapid deterioration and softening when relieved of overburden pressure following excavation and in the presence of water. Increased moisture contents can result in a reduction of shear strength, marked reduction in bearing capacity and loss of side friction for piles. Auger drilling for cast-*in situ* piles under wet conditions may also remould the mudstone lining the borehole walls to a soft clay. The deterioration of mudstones has been observed in foundation excavations in the centre of Leeds, where the exposed rock has altered to a slurry (0.2 to 0.3 m thick) in a matter of days. In the Adel district, contractors have had to repeatedly deepen excavations that have deteriorated to a mud slurry between the time foundation level was reached and building inspectors arrived on site.

Suspect mudstones should be subject to slaking tests during site investigation and be protected from water ingress during excavation by cut-off drains, sheet piling or similar measures. Borings for piles should be cased through water-bearing strata and, often, engineers aware of the problem will undertake pile design based on end bearing values only. To prevent deterioration in open excavations, the mudstone should be covered with a layer of blinding concrete as soon as formation level is reached and the side slopes protected by spraying with bitumen or other methods.

Variations in the depth and grade of weathering give rise to differing soil thicknesses and foundation conditions in the mudrock lithologies, and variable bearing capacities are to be expected in the weathered zone. It is recommended that during site investigations for shallow foundations, trial pitting is carried out as a matter of routine in order to ascertain the variation in weathering profiles.

Mudrocks may also pose a problem to shallow foundations during prolonged periods of drought, when they are prone to drying, shrinkage and cracking. Foundations should therefore be sufficiently deep to prevent this.

4) Geological faults

Geological faults are planes about which adjacent blocks of rock strata have moved relative to each other. Movement may be vertical, horizontal or, more likely, a combination of the two. They may be reactivated by mining, occasionally causing earth tremors. Fault zones contain weakened, fractured and brecciated rock which promote deep weathering profiles and therefore varying bearing capacities. Lithologies with differing strengths and settlement characteristics may lie juxtaposed on either side of a fault. Foundations straddling this contact may therefore suffer differential settlement. This may not be a severe problem for light to moderately loaded structures, but heavy structural loads should be taken down to levels where differential settlements are of an acceptible magnitude.

Most structures cannot withstand the magnitude of movements initiated along faults by mining subsidence, and buildings should not be placed across faults unless the workings can be stabilised or it is certain that any existing subsidence is complete. It is therefore important that the precise positions of faults (and underground workings) are established prior to any future development.

5) Coal at foundation level

When coal is encountered, it should be removed to at least a metre depth below foundation level, and a suitable inert fill placed between the foundation and the coal as a precaution against spontaneous combustion. An alternative approach may be to opencast the site, which is advisable if previous partial extraction has left potentially hazardous ground conditions.

6) Karstic features in Magnesian Limestone

These may be a potential problem only in the limited limestone outcrops present in the north-east corner of the study area.

Karstic features are formed by the dissolution of the soluble limestone by surface and ground waters percolating along joints and fissures. This results in highly variable rockhead levels and hollows, depressions and channels in the limestone surface, commonly infilled with superficial deposits of variable thickness, lateral extent and composition. At depth, dissolution results in the formation of voids, cavities and enlarged open or partially filled channels following faults, fractures, joints and bedding planes. These features can lead to severe differential settlements below heavy structures and may preclude the use of shallow foundations except for light structures. Water inflow problems may also be encountered in excavations.

7) Landslips.

Natural landslipping does not present a significant problem to development in the area. A few, generally shallow, dormant debris slides/flows and rotational slumps have been recognised during the geological mapping; the landslip deposits generally occuring on or near the middle-lower parts of slopes, in excess of about 8 to 10 degrees, in weathered mudstone below a coal seam and/or a capping sandstone (eg. near Meanwood Grove [2815 3842]). Minor, shallow slope movements (too small to be indicated as individual landslips on the geological maps, but identified from the presence of low arcuate scars above hummocky ground) are probable fairly common in the weathered mantle and/or Head deposits on such slopes (eg. at Farnley, north of Billey Lane [2552 3201]).

Construction on these areas should be avoided wherever possible. If this is not feasible, it is essential that site investigations are undertaken to adequately define the geometry and current stability of the landslip (and adjacent ground) in order that suitable stabilisation measures can be designed and implemented prior to construction. In some cases, small areas of landslip deposits may be removed completely and replaced with a free-draining granular fill. However, the mechanisms leading to initial failure should be ascertained and the likelihood of future potential movements in adjacent ground investigated.

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Appendix I

- Summary geotechnical tables
- Annex A: Key tables, defining parameter 'classes' used in summary geotechnical tables.

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Buff, weathered, fine to coarse-grained, marly dolomitic LIMESTONE with local oolitic horizons. Thinly to massively bedded.	Variable rockhead level with possible karstic conditions. Highly weathered zones and solution cavities (infilled and open) may cause excessive differential settlements and inadequate bearing capacities for shallow foundations.

SUMMARY FORMAT

TEST PARAMETER	No. of Samples
No of Samples 50%	>5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
24 56 47 100	22 77 50 90	27 17 13 20 12 23	

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
27 17 13 20 12 23	7 36 31 41	7 21 18 24	7 18 10 20			3 7.8	4 1

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
5 88	5 8			

* Classes defined in Annex A

Table 1. Summary geotechnical data for Lower Magnesian Limestone (Cadeby Formation)

Table 1. Summary geotechnical data for Lower Magnesian Limestone (Cadeby Formation)

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Grey to greyish brown, moderately to well-jointed, moderately strong, variably micaceous and locally conglomeratic, fine to coarse grained SANDSTONES of the Millstone Grit and Coal Measures.	Moderately strong to strong rock. Generally good founding medium but interbedded mudstones will cause local variations in settlement characteristics Potential problems with underlying shallow coal workings.Ripping/blasting required for excavations.

SUMMARY FORMAT

TEST PARAMETER	No. of Samples
No of Samples 50%	>5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
11 150 64 200	64 42 20 75 0 99	222 52 28 81 14 100 0 100	

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
64 4 3 8 2 14				49 2.42 2.36 2.49 2.26 2.57		17 6.7 6.1 7.4	17 1 1 1

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
		61 35 25 54 17 64	53 1.1 0.8 1.9 0.3 2.5	122 1.55 0.8 2.3 0.4 3.2 0.3 3.7

* Classes defined in Annex A

Table 2. Summary geotechnical data for sandstones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

Table 2. Summary geotechnical data for sandstones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

ENGINEERING GEOLOGY DESCRIPTION
Greyish brown, moderately strong to moderately weak, silty micaceous SANDSTONES and medium dense to dense silty, clayey SANDS. (weathered sandstones of the Millstone Grit and Coal Measures)

ENGINEERING GEOLOGY COMMENTS
Variable depth of weathered zone. May be weathered to sandy soil within 5 m of ground surface. Zones of highly weathered rock may be present to depths in excess of 12 m below ground surface, particularly in fault zones.

SUMMARY FORMAT

TEST PARAMETER		
	No of Samples	No. of Samples
50%		>5
25%	75%	>10
(percentiles)		
10%	90%	>25
2.5%	97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
525	220	117	
64	73	31	
36 123	50 102	14 41	
19 176	31 133	0 46	
11 283	1 200	0 72	

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
670	182	182	165	348	57	219	223
15	33	20	13	2.06	1.75	6.7	1
12 19	28 38	18 22	9 18	2.00 2.13	1.66 1.82	6.1 7.1	1 1
9 24	28 44	16 25	6 22	1.91 2.19	1.52 1.92	5.6 7.5	1 1
6 30	22 58	14 30	2 29	1.81 2.25		4.5 8.2	1 2

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
25	25	36	36
7	25	36	32
4 20	19 34	21 54	8 48
3 23	15 40	12 64	1 70

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)
28	29			67	6
3	4			5	4
3 3	3 4			3 7	
3 3	3 4			2 8	

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
347	346	13	69	33
70	6	25	1	0.5
38 105	0 18	20 32	0.6 1.3	0.3 1.0
24 150	0 28		0.2 1.6	0.3 1.5
13 250	0 36			

* Classes defined in Annex A

Table 3. Summary geotechnical data for sandstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

Table 3. Summary geotechnical data for sandstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

Table 4. Summary geotechnical data for mudstones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Medium to dark grey, moderately fissured, weak to moderately strong, silty and sandy MUDSTONES of the Millstone Grit and Coal Measures.	Fresh to slightly weathered rock of low to intermediate plasticity and medium compressibility. Tends to deteriorate rapidly in presence of water. Suitable as fill if placed under controlled compaction conditions

SUMMARY FORMAT

TEST PARAMETER	No. of Samples
No of Samples 50%	>5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
89 156 94 202 71 260	26 50 40 75 12 102	9 54	

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
86 9 8 12 6 15	25 31 29 33 26 35	25 21 19 22 18 23	25 10 8 12 6 13	12 2.15 2.08 2.30	8 1.915 1.82 1.95	8 6.7 6.1 7.5	8 1

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
9 105 76 110	9 20 12 23	65 32		2 1.1

* Classes defined in Annex A

Table 4. Summary geotechnical data for mudsones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Orange brown and pale grey, moderately strong, fissured silty MUDSTONES and firm to stiff, silty CLAYS of low to high plasticity (weathered mudstones of the Millstone Grit and Coal Measures).	Depth of weathered mantle variable but often within 2 to 6 m of ground surface. Highly weathered zones may occur to depths in excess of 10-15 m. Heavy loads may need piling to sound bedrock. May be too wet to achieve optimum compaction.

SUMMARY FORMAT

TEST PARAMETER	
No of Samples	
50%	No. of Samples >5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
785	145	40	3
66	102	27	12
40 100	75 135	11 43	
27 146	50 175	0 61.5	
14 195	20 230		

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
1198	296	296	296	778	111	335	334
17	42	23	19	2.05	1.74	6.5	1
13 22	36 52	21 26	14 25	1.95 2.14	1.59 1.84	6 7	1 1
10 28	29 59	19 29	9 33	1.86 2.21	1.52 1.93	5.5 7.5	1 1
8 34	26 70	16 33	4 42	1.78 2.28	1.35 2.00	4.5 8.0	1 1

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
22	22	22	22
29	55	11	2
18 40	46 64	5 17	0 6
15 47	41 67	4 24	0 11

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)
27	27	17	17	47	12
3	3	2.00	11	3	5.5
3 3	3 4	1.96 2.00	9 11	2 7	3 23.5
3 3	3 4			1 10	

* Classes defined in Annex A

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
748	748	5		6
76	8	7		0.7
53 110	0 15	1 13		0.5 0.9
34 139	0 22			
18 190	0 30			

Table 5. Summary geotechnical data for mudstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

Table 5. Summary geotechnical data for mudstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Grey to greyish brown, moderately strong, variably clayey SILTSTONES of the Millstone Grit and Coal Measures.	Rock of low plasticity and low compressibility. Grades laterally and vertically into mudstones and sandstones. Generally good founding medium but interbedded mudstone may cause local variations in strength and bearing capacity.

SUMMARY FORMAT

TEST PARAMETER		
	No of Samples 50%	No. of Samples >5
25%	75% (percentiles)	>10
10%	90%	>25
2.5%	97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
4 136	32 91 55 115 40 130	39 28 14 46 6 54	

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
13 9 6 12				4 2.4		2 7.6	2 1

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
		5 28	3 0.6	24 1.3 0.8 1.75 0.1 2.5

* Classes defined in Annex A

Table 6. Summary geotechnical data for siltstones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

Table 6. Summary geotechnical data for siltstones of the Millstone Grit and Coal Measures (fresh to slightly weathered).

ENGINEERING GEOLOGY DESCRIPTION	ENGINEERING GEOLOGY COMMENTS
Greyish brown, moderately strong to moderately weak clayey SILTSTONES and firm clayey SILTS and silty CLAYS (weathered siltstones of the Millstone Grit and Coal Measures).	Variable depth of weathered zone. May be weathered to silty clay soil within top 1 to 5 m of ground surface. Variable strength and bearing capacity associated with depth and grade of weathering.

SUMMARY FORMAT

TEST PARAMETER	No. of Samples
No of Samples	
50%	>5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD MEASUREMENTS

SPT N-VALUE	RPT (mm	RQD (%)	If (mm)
178	57	48	
83	110	0	
49 134	75 150	0 34	
32 164	43 225	0 0	
20 225			

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
260	88	88	88	178	28	72	72
17	38	22	17	2.11	1.81	6.7	1
14 20	34 44	19 24	14 21	2.03 2.18	1.68 1.88	6.15 7.3	1 1
11 23	29 50	17 27	9 25	1.92 2.23	1.56 1.93	5.3 7.8	1 1
9 30				1.75 2.35			

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
6	6	7	7
14	44	20	22

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	MDD (Mg/m3)	OMC (%)	CBR unsoaked (%)	CBR soaked (%)
32	32			8	4
3	4			8	7
3 3	3 4			3.5 9	
2 3	3 4				

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)	UCS (MPa)	PT. LOAD diam. (MPa)	PT. LOAD axial (MPa)
152	152	2	9	29
90	0	16.5	0.4	1.3
55 134	0 9		0.3 0.7	0.8 2.4
42 190	0 20			0.3 3.5
20 308	0 30			

* Classes defined in Annex A

Table 7. Summary geotechnical data for siltstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

Table 7. Summary geotechnical data for siltstones of the Millstone Grit and Coal Measures (moderately to completely weathered).

Table 8. Summary geotechnical data for Head deposits.

ENGINEERING GEOLOGY DESCRIPTION
Soft to firm, sandy silty CLAY with stones (typically). Locally, may be a gravelly silty SAND or sandy GRAVEL.

SUMMARY FORMAT

TEST PARAMETER	
No.of Samples	No. of Samples
50%	>5
25% 75% (percentiles)	>10
10% 90%	>25
2.5% 97.5%	>100

FIELD TESTS

SPT N-VALUE
34
22.5
15 35
7 51

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
164	61	65	61	121	13	44	48
17	33	20	12	2.08	1.57	6.4	1
14 22	28 39	17 22	8 17	1.98 2.14	1.53 1.67	5.85 6.7	1 1
12 28	24 44	15 27	5 21	1.86 2.20		5.1 7.3	1 1
10 43				1.78 2.26			

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
12	12	13	13
8.5	26.5	42	20
6 16	19.5 36	39 50	11 25

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)
6	6		
3	3		
3 3	3 4		

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)
130	130
55	5
36 84	0 14
116.5 20	0 20
9 175	0 28

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Variable lithology and thickness gives rise to variations in geotechnical behaviour. Generally low bearing capacities and variable rates of settlement. May contain relict shear surfaces where developed on slopes > 3° - 7°, with shear strengths approaching residual values.

Table 9. Summary geotechnical data for alluvial silts and clays.

ENGINEERING GEOLOGY DESCRIPTION
Very soft to firm, occasionally laminated, sandy, silty and organic CLAYS and clayey, sandy SILTS, with impersistent layers or 'pockets' of PEAT.

SUMMARY FORMAT

TEST PARAMETER		
	No. of Samples	No. of Samples
	50%	>5
25%	75%	>10
(percentiles)		
10%	90%	>25
2.5%	97.5%	>100

FIELD TESTS

SPT N-VALUE
63
10
6 24
4 27

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
464	152	152	152	322	30	107	112
26	38	23	15	1.95	1.415	6.8	1
19 33	33 46	20 27	12 21	1.85 2.10	1.36 1.57	6.5 7.3	1 1
15 39	28 53	18 30	8 26	1.74 2.21	1.30 1.68	5.7 8	1 2
12 55	25 62	16 36	4 34	1.63 2.28		4.4 8.3	1 2

PARTICLE SIZE DISTRIBUTION

	CLAY + SILT (%)	SAND (%)	GRAVEL (%)
	5	5	5
	22	39	29

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)
45	43		
3	3		
3 3	3 4		
3 4	2 4		

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)
305	305
40	0
21 65	0 6
15 104	0 12
9 217	0 22

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Highly compressible soft silts, clays and peat, with low shear strengths and very low bearing capacities. Generally unsuitable founding medium due to large differential settlements. High groundwater levels pose severe water inflow problems for excavations.

Table 10. Summary geotechnical data for alluvial sands and gravels.

ENGINEERING GEOLOGY DESCRIPTION
Medium dense to dense, fine to coarse-grained SANDS and angular to rounded, fine to coarse GRAVELS with variable clay content and local lenses of organic silt or brown pebbly sandy clay.

SUMMARY FORMAT

TEST PARAMETER		No. of Samples
	No. of Samples	
50%		>5
25%	75%	>10
(percentiles)		
10%	90%	>25
2.5%	97.5%	>100

FIELD TESTS

	SPT N-VALUE
No. of Samples	507
50%	30
25% / 75%	22 / 42
10% / 90%	15 / 71
2.5% / 97.5%	7 / 114

INDEX AND CHEMICAL TESTS

	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
No. of Samples	127	10	10	10	25	5	57	56
50%	13	26	20	7	2.06	1.87	7.2	1
25% / 75%	11 / 17	25 / 30	18 / 21	5 / 12	1.96 / 2.19		6.8 / 7.4	1 / 1
10% / 90%	8 / 26				1.92 / 2.27		5.3 / 7.7	1 / 2
2.5% / 97.5%	6 / 38							

PARTICLE SIZE DISTRIBUTION

	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
No. of Samples	78	78	125	125
50%	2	5	28	63
25% / 75%	0 / 3	1 / 11	15 / 38	47 / 81
10% / 90%	0 / 6	0 / 20	11 / 53	34 / 88
2.5% / 97.5%			4 / 68	4 / 94

CONSOLIDATION AND COMPACTION TESTS

	Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)
No. of Samples	3	3		
50%	3	4		

STRENGTH TESTS

	Cu (kPa)	ϕu (degrees)
No. of Samples	25	25
50%	30	18
25% / 75%	20 / 55	12 / 27
10% / 90%	5 / 88	0 / 30

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Medium dense/dense basal alluvial gravels of sufficient thickness are often suitable as sets for piles, provided site investigations have proved the absence of clay, silt or organic lenses. Casing is required for bores to combat severe water inflows.

Table 11. Summary geotechnical data for till (boulder clay).

ENGINEERING GEOLOGY DESCRIPTION
Brown, stiff to very stiff, stony, sandy CLAY (boulder clay) with clasts of sandstone, chert, ironstone and limestone* (*where it lies close to the Magnesian Limestone outcrop).

SUMMARY FORMAT

TEST PARAMETER		No. of Samples
	No.of Samples	>5
	50%	
25%	75% (percentiles)	>10
10%	90%	>25
2.5%	97.5%	>100

FIELD TESTS

SPT N-VALUE
74
31
15 51
6 66

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
527	158	175	158	208	123	75	76
16	34	17	16	2.10	1.92	6.6	1
13 19	29 39	16 19	13 21	2.03 2.17	1.8 1.98	5.9 7.3	1 1
12 23	24 46	15 23	8 26	1.95 2.23	1.74 2.02	5.3 7.6	1 1
11 28	20 58	14 27	5 31	1.81 2.30	1.65 2.08		

PARTICLE SIZE DISTRIBUTION

CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)
23	23	25	25
15	34	35	11
9 19	19 43	28 50	5 12
		22 73	1 29

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)
19	19	37	8
3	3	9	4
3 3	3 3	7 17	
		5 21	

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)
255	255
93	5
64 147	0 9
35 210	0 14
16 290	0 21

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Boulder clay is often variable with lenses of sand, bands of laminated clays and interbedded sandy gravels. Matrix clays of low to medium plasticity, and low compressibility and permeability. Few problems for shallow foundations provided lithological variations are established during site investigations.

Table 12. Summary geotechnical data for till (glaciofluvial sands and gravels).

ENGINEERING GEOLOGY DESCRIPTION
Glaciofluvial sands and gravels ranging from slightly cohesive, clayey silty SANDS to highly permeable, medium dense to very dense sandy GRAVELS with layers and lenses of silt and clay developed locally.

SUMMARY FORMAT

TEST PARAMETER		
	No.of Samples	No. of Samples
50%		>5
25% (percentiles)	75%	>10
10%	90%	>25
2.5%	97.5%	>100

FIELD TESTS

SPT N-VALUE	
	44
44	
32	54
22	80

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
25						10	10
15						6.85	1
9 16						6.7 7.0	1 1
8 20							

PARTICLE SIZE DISTRIBUTION

	CLAY + SILT (%)	SAND (%)	GRAVEL (%)
	24	24	25
	10	37	51
	4 22	21 53	30 73
			0 83

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Generally dense to very dense sandy gravels should provide few problems for most foundations, provided lithological variations are established during site investigations. Excavations will require immediate support and where water-bearing, measures to regulate water inflows will be needed.

Table 13. Summary geotechnical data for river terrace deposits.

ENGINEERING GEOLOGY DESCRIPTION
River terrace deposits comprising medium dense fine to coarse SANDS and medium dense to dense fine to medium GRAVELS, with occasional cobbles. Granular materials predominate but sandy CLAYS and SILTS (sometimes laminated) occur locally, usually at terrace margins.

SUMMARY FORMAT

TEST PARAMETER		
	No.of Samples	No. of Samples
	50%	>5
25%	75%	>10
(percentiles)		
10%	90%	>25
2.5%	97.5%	>100

FIELD TESTS

SPT N-VALUE	
	131
22	
12	32
9	47
4	95

INDEX AND CHEMICAL TESTS

MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	BULK DENSITY (Mg/m3)	DRY DENSITY (Mg/m3)	pH	SO3 class*
149	25	25	25	93	7	57	56
15	27	17	8	2.10	1.81	6.8	1
12 18	23 30	17 20	5 13	2.03 2.17		6.3 7.3	1 1
9 23	21 38	14 24	4 15	1.89 2.24		6.1 7.7	1 1
6 30						4.8 8.0	

PARTICLE SIZE DISTRIBUTION

	CLAY + SILT (%)	SAND (%)	GRAVEL (%)
	30	30	30
	13	28	58
	4 16	22 42	43 70
	1 29	9 54	27 8

CONSOLIDATION AND COMPACTION TESTS

Mv class*	Cv class*	CBR unsoaked (%)	CBR soaked (%)

STRENGTH TESTS

Cu (kPa)	ϕu (degrees)
94	94
51.5	9
28 90	0 17
19 140	0 24

*Classes defined in Annex A

ENGINEERING GEOLOGY COMMENTS
Granular terrace deposits should prove a suitable founding medium for most light to medium structures, provided lithological variations are established during site investigations. In some areas, these deposits may infill buried channels which must be delineated prior to foundation design. Excavations/bores require immediate support/casing.

Annex A

Key Tables, defining parameter classes used for summary geotechnical test data presented in Appendix II, Tables 1 to 13.

TABLE A1. Sulphates in soils and Groundwater

Class	Total SO_3 (%)	SO_3 in 2 : 1 Soil : Water extract (g/l)	In Groundwater (g/l)
1	<0.2	<1.0	<0.3
2	0.2 - 0.5	1.0 - 1.9	0.3 - 1.2
3	0.5 - 1.0	1.9 - 3.1	1.2 - 2.5
4	1.0 - 2.0	3.1 - 5.6	2.5 - 5.0
5	>2	>5.6	>5

From BRE Digest 250 (1981)

TABLE A2. Scale of Point Load Strengths

Term	Point Load Strength KN/m²
Extremely Strong	>12000
Very Strong	6000 - 12000
Strong	3000 - 6000
Moderately Strong	750 - 3000
Moderately Weak	300 - 750
Weak	75 - 300
Very Weak	<75

After Anon (1972)

TABLE A3 (a). Coefficient of Volume Compressibility, Mv.

Class	Description of Compressibility	Mv (m²/MN)	Examples
5	Very High	>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	Fluvioglacial clays, Lacustrine clays
2	Low	0.05 - 0.1	Boulder clays
1	Very Low	<0.05	Heavily overconsolidated 'boulder clays', Stiff weathered rocks

After Head (1982)

TABLE A3 (b). Coefficient of Consolidation, Cv.

Class	Cv m²/year	Plasticity Index Range	Soil Type
1	<0.1	>25	Clays Montmorillonite
2	0.1 - 1		High plasticity
3	1 - 10	25 - 15	Medium plasticity
4	10 - 100	15 or less	Low plasticity
5	>100		Silts

After Lambe and Whitman (1979)

A4. California Bearing Ratio (CBR) Tests

CBR test results given in summary tables (Tables 1 to 13, in Appendix II) show values for soils compacted at natural moisture content.

Appendix II

Glossary of geotechnical tests quoted in the database and their applications

Appendix II

Glossary of geotechnical tests quoted in the database and their applications

1. FIELD TESTS

The Standard Penetration Test (SPT)

The standard penetration test (SPT) is a dynamic test carried out at intervals during the drilling of a borehole. A standard 50 mm diameter split barrel sampler is driven into the soil at the bottom of the hole for a distance of 450 mm by the blows of a standard weight (65 kg), falling through a standard distance (0.76 m). The number of blows (N) required to drive the last 300 mm is recorded. Test details are given in BS 5930:1981.

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 penetration test (CPT).

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

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

Relative Density/Consistency	SPT	RPT
Very loose/very soft	0 - 5	
Loose/soft	5 - 10	
Medium dense/firm	10 - 30	
Dense/stiff	30 - 50	
Very dense/hard	50	< 300
Rock/Heavily overconsolidated soils		200 - < 100

Permeability

The permeability of a soil is its capacity to allow water to flow through it. It may be measured on laboratory samples or from boreholes. Laboratory tests do not take into account the effect of structural discontinuities in the soil or rock mass *in situ* and may not, therefore, give a true indication of the permeability of the ground *en masse*. Pumping tests using boreholes give more representative permeability values.

In field permeability tests, water may be 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 BS 5930:1981. Sections of borehole may be isolated by sealing with 'packers' in order to measure water flows over selected depth intervals. The data obtained from either method enable the coefficient of permeability (k, metres/second) to be calculated.

The permeability is used to predict the inflow of water during excavation or tunnelling and to design groundwater control schemes to deal with it. This parameter 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.

Rock Quality Designation (RQD)

RQD is a quantitative index based upon core recovery by diamond drilling. The RQD is defined as the percentage of core recovered in intact pieces of 100 mm (10 cm) or more in length over the total length of the borehole. An RQD value would normally be established for each core run (of say 2 metres). Hence:

$$\text{RQD (\%)} = 100 \times \frac{\text{length of core in pieces} < 100\text{ mm}}{\text{length of core run}}$$

Deere (1964) proposed the following relationship between the numerical value of RQD and the engineering quality of the rock:

RQD	Rock Quality
25%	very poor
25 - 50%	poor
50 - 75%	fair
75 - 90%	good
90 - 100%	very good

Fracture spacing index (I_f)

The fracture spacing index gives a direct measure of fracture spacing, and refers to the average size of cored material within a recognisable unit, measured in millimetres. The 'unit' may be a geological or lithological unit, or a unit length with a similar number of fractures. When few fractures traverse the core, the index is the unit length divided by the number of fractures within the unit. If the core is very broken, the index is the average diameter of a number of separate rock fragments. For example, if a one metre long unit of core is cut by four fractures, the fracture spacing index (I_f) is 1000 mm / 4 = 250 mm. For a half metre length of core fractured into pieces with an average size of approximately 3 mm, the index for that 500 mm long unit will be the average diameter of the core fragments = 3mm.

2. LABORATORY TESTS

2.1 Index and Chemical Tests

Moisture Content

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

$$\text{Moisture content, } m = \frac{\text{weight of water}}{\text{weight of solids}} \times 100\%$$

The standard test procedure is given in BS 1377:1975. The moisture content is a basic soil property and influences soil behaviour with regard to compaction, plasticity, consolidation and shear strength characteristics.

Atterberg or Consistency Limits (Plasticity Tests)

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

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

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

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

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

$$Ip = LL - PL$$

The test procedures are given in BS 1377: 1975.

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

Density

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

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

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

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

Density measurements are simple if an undisturbed specimen of known, or easily measured, volume is obtained. If this is not possible in the field, the sand replacement method is used to determine the volume of a hole from which the soil sample is excavated by filling with a measured quantity of dry, uniformly graded sand of known density. Density measurements are usually expressed as Mg/m^3 and full test details are given in BS 1377:1975.

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

Specific Gravity

The specific gravity of a soil is the ratio of the weight of dry solids to the weight of an equal volume of water (i.e. the weight of water displaced by the solids). It is, therefore a dimensionless parameter. Full test details are given in BS 1377:1975. Specific gravity is a basic soil property and represents an average for the particles of different minerals present in a soil sample. The parameter is used to enable calculation of other basic soil properties. For example: specific gravity (G), moisture content (m), voids ratio (e) and degree of saturation (S) are given by the useful relationship:

$$Gm = Se$$

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 through each sieve is calculated. A grading curve of percentage passing against sieve size is plotted.

The fines passing 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, dried and the contained solids weighed or, alternatively, hydrometer readings of the soil-water suspension are recorded at specific time intervals. The size distribution can then be calculated using Stokes' Law which relates settling time to particle size. The entire grading curve for coarse and fine material can then be plotted. Full details are given in BS 1377:1975.

Particle size distribution is used for classifying soil in engineering terms (BS 5930:1981) which in turn provides an indication of soil behaviour with regard to permeability, susceptibility to frost heave or liquefaction, and some indication of strength properties. Particle size analysis does not, however, indicate structure and will not distinguish between a sandy clay and a laminated sand and clay which may behave very differently *in situ*, but may show similar particle size distribution in a bulk test sample.

pH

In the pH test, 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 for at least 12 hours. A glass electrode connected to a pH meter is then placed in the stirred mixture and the 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 BS 1377:1975.

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

Sulphate

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

The sulphate content of groundwater or an aqueous soil extract is determined by passing the water through a column of strongly acidic, cationic exchange resin activated with hydrochloric acid. The groundwater or soil-water washings are collected and titrated against standardised sodium hydroxide solution, using a suitable indicator. From the amount of sodium hydroxide used during titration the quantity of dissolved sulphates can be determined and expressed in terms of SO_3 content, as grams per litre or as parts per hundred thousand. Full test details are given in BS 1377:1975.

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

2.2. Strength Tests

Triaxial Compression Test

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

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

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

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

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

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

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

Uniaxial Compression Test

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

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

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

Point Load Test

The point load test provides a rapid, economical and reasonably accurate index of rock strength and strength anisotropy, the principles of which have been described by Broch and Franklin (1972) and Bieniawski (1975). It can be conducted in the laboratory or in the field. The test involves loading a sample of rock to failure between two standard-shaped steel cones mounted in a portable hydraulically-operated press. The load at failure is determined from appropriately calibrated pressure gauges and the Point Load Strength (Is) calculated from the relationship:

$Is = P/D^2$, where D is the initial distance between the loading cones holding the sample; and P is the maximum pressure recorded on the gauge (ie. the failure load).

The test is usually carried out on pieces of rock core placed both diametrically and axially between the loading cones to obtain an indication of strength anisotropy. For diametral tests the diameter:length ratio (D/L) of the core should be >1.4; for axial tests the D/L ratio should be 1.1 (where D is the axial distance between the cones). The test may also be performed, less accurately, on irregular lumps with D/L ratios of 1.0 - 1.4.

It is usual practice to correct the point load strength (Is) to a standard 50 mm reference diameter using a set of standard curves to give the Point Load Index, $Is_{(50)}$. This index is essentially equivalent to the point load strength multiplied by correction factors for shape and size. The Anisotropy Index (Ia) of the rock may be expressed as:

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

2.3 Consolidation and Compaction Tests

Consolidation Test

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

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

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

Compaction

The compaction test determines the moisture content (the 'optimum') at which a soil may be compacted to its maximum dry density. A quantity of soil (5 kg) is compacted in a standard (Proctor) mould using a standard (2.5 kg) or heavy (4.5) rammer 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 are used to determine the optimum moisture conditions at which to place a given soil as general or embankment fill.

California Bearing Ratio (CBR)

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

$$\text{CBR} = \frac{\text{Measured force}}{\text{'Standard force'}} \times 100\%$$

There are, however, various ways of preparing samples for the test. The samples may be either undisturbed or remoulded. Remoulded samples may be compressed into a standard CBR (or Proctor) mould under a static load, or dynamically compacted into it, at the required moisture content, either to achieve a specific density or by using a standard compactive effort. Undisturbed samples may be taken on site in a CBR mould, either from natural ground or from recompacted soil such as an embankment or a road sub-base. Specimens may be tested in the mould as prepared (or as received) or after soaking in water for several days.

For soaked CBR tests on remoulded soil at maximum compaction, for example, the test normally involves a series of samples which are compacted in a 152 mm diameter mould at moisture contents around the optimum. 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.

The CBR value of recompacted soil is very sensitive to variations in moisture content and dry density. Some typical laboratory CBR values for British soils compacted at natural moisture content are indicated below:

Type of soil	Range of PI (%)	Range of CBR* (%)
Clay	40 - 70	1 - 3
Silty clay	about 30	3 - 5
Sandy clay	10 - 20	4 - 7
Silt	0	1 - 2
Sand (poorly graded)	NP	10 - 20
Sand (well graded)	NP	15 - 40
Sandy gravel (well graded)	NP	20 - 60

Lower values relate to water table depth < 600 mm below formation level. Upper values to water table > 600 mm below formation level. (From TRRL Road Note 29)

In the field test, the plunger is jacked into the ground against the reaction of a heavy lorry. Field values are usually lower than laboratory values and the results of these *in situ* tests are not directly comparable with laboratory test results. The laboratory test in the CBR mould is recognised as the standard test. 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.