THE HIGHWAYS AGENCY

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THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF
THE ENVIRONMENT FOR NORTHERN IRELAND

Earthworks - Design and Preparation of Contract Documents

Summary: This Amendment deletes the specific formula for earthworks quantities measurement, clarifies the relationship between earthworks quantities measurement pre-tender and post-contract, and corrects the references to the latest contract documents consequent on the publication of the Manual of Contract Documents for Highway Works dated December 1991.

AMENDMENT NO.1 (APRIL 1995)

Replacement, Additional and Deleted Pages

Page No.		Date
Front sheet		
Contents page		April 1995
Replace	Chapter 1 pages 1-4 incl.	April 1995
Add	Chapter 1 pages 5, 6	April 1995
Replace	Chapter 3 pages 7, 8	April 1995
Delete	Chapter 3 pages 9, 10	April 1995
Replace	Chapter 14 pages 1, 2	April 1995
Delete	Annex B pages i, ii	April 1995

The replacement sheets supersede those dated June 1991. All superseded and deleted pages should be archived as appropriate.

Summary

1. As noted in the amended Chapter 1, paragraph 1.13, the `Designer' has been retained throughout for consistency within the amended document. The equivalent current term is the Design Organisation (DO).

2. "Paragraph(s)" (with an upper case `P') has been used throughout this amendment for consistency with the original document. At a future revision "paragraph" (with a lower case `p') will be used throughout.

Implementation

The amended Advice Note should be used forthwith for all schemes currently under preparation, unless, in the opinion of the Overseeing Organisation, this would result in significant additional expense or delay progress. Design Organisations should verify the use of this Advice Note with the Overseeing Organisation.

VOLUME 4 GEOTECHNICS AND DRAINAGE

SECTION 1 EARTHWORKS

PART 1

HA 44/91

DESIGN AND PREPARATION OF CONTRACT DOCUMENTS

Contents

Chapter

1. INTRODUCTION

General

1.1 The guidance given in this Departmental Advice Note covers two areas. The first area is specifically directed to preparing contract documents under the Manual of Contract Documents for Highway Works (MCHW) dated December 1991, and its amendments. See Paragraphs 1.4 to 1.16 below. Table 1/1 below shows where advice on each SHW Series 600 Clause and relevant Appendix is given in this Advice Note. The second area covers general advice on design and assessment of earthworks.

1.2 The following Departmental Standard and Advice Note have been reviewed and are now superseded by this Advice Note:-

i. HD 6/80 Specification for Dynamic Compaction;

ii. HA 11/80 Dynamic Compaction of Earthworks.

Scope

1.3 The guidance given on design of earthworks, and preparation of that part of contract documentation for highway construction relating to earthworks, is applicable to all Department of Transport Trunk Roads including Motorway projects. This Advice Note may also be considered as good practice on other schemes involving major earthworks.

Definitions and Abbreviations

1.4 The following definitions and abbreviations are used and shall apply in this Advice Note.

1.5 Ground Investigation (GI) is the examination of a site required to provide geotechnical data which are representative of the ground conditions and relevant to the scheme considered. It forms part of the overall Site Investigation (SI).

1.6 Specification for Road and Bridge Works (SRBW) is the Department of Transport Specification for Road and Bridge Works published in 1976 and is now superseded.

1.7 Specification for Highway Works (SHW) is the Department of Transport Specification for Highway Works published as Volume 1 of the Manual of Contract Documents for Highway Works (MCHW1).

1.8 Notes for Guidance (NG) are the Notes for Guidance on the Specification for Highway Works (NGSHW) published as Volume 2 of the Manual of Contract Documents for Highway Works (MCHW 2).

1.9 Method of Measurement is the Department of Transport Method of Measurement for Highway Works (MMHW) published as Section 1 of Volume 4 of the Manual of Contract Documents for Highway Works (MCHW 4.1) (Bills of Quantities for Highway Works).

1.10 This Advice Note was published in June 1991 on the basis that the, then current, Highway Construction Details (HCD) are the Department of Transport Highway Construction Details (Third Edition) published in 1987. However, the current HCD published as Volurme 3 of the Manual of Contract Documents for Highway Works (MCHW 3) should be used. These do not differ significantly on earthworks and drainage matters.

1.11 Regional Geotechnical Engineer (GE) is the Regional Office (Transport) Geotechnical Engineer with delegated responsibility for the geotechnical aspects of the scheme.

1.12 Geotechnical Liaison Engineer (GLE) is the Designer's nominee responsible for geotechnical matters relating to the scheme.

1.13 The Designer is the Consulting Engineer or Agent Authority responsible for the earthworks design. The 'Designer' is now termed the 'Design Organisation' (DO) but, pending a future complete revision of this Advice Note, the former term is retained pro tem for consistency within the amended document.

1.14 The Engineer is the Engineer as defined in the Conditions of Contract.

1.15 The Project Manager is appointed by the Regional Office Director (Transport) to be responsible for the day-to-day management of the scheme on behalf of Dtp.

1.16 In this Advice Note it has been assumed that the Conditions of Contract (CoC) are the Institution of Civil Engineers (ICE) Conditions of Contract for use in connection with Works of Civil Engineering Construction, Fifth Edition, as amended by the Model Contract Document (MCD) for Highway Works Contracts issued by the Department of Transport as Volume 0 of the MCHW (MCHW 0.1) for use with the December 1991 edition of the SHW. The actual CoC will be those promulgated by the SD series of Departmental standards and the relevant MCD stated therein.

Stages of Earthworks Design

General

1.17 The use of the Department's Standards, Advice Notes, Specifications etc are all part of the Certification procedures. For these to work satisfactorily there needs to be good liaison between the Project Manager, Designer, GE and GLE. It must be noted that the Project Manager has a clear responsibility for the technical aspects and the management of the scheme whilst the GE is responsible for receipt of Certification and may be required to be involved in the geotechnical aspects. The GE must keep the Project Manager fully informed of all geotechnical matters which might affect the cost and progress of the scheme. Equally, the Project Manager must inform the GE of any geotechnical work required to progress the scheme.

Project Stages

1.18 The stages of a highway scheme, and the appropriate geotechnical works, are given as a guide in Table 1/2. The geotechnical complexity of any scheme may mean some latitude is required if unnecessary study and abortive costs are to be avoided. On certain schemes, alternative lines may each require a site investigation to properly determine the choice of route.

1.19 For some schemes, at the Main Ground Investigation stage, it may be that either the bridge types or actual locations may not be fully confirmed so that the Main Ground Investigation may need to be split into two parts. The first GI will give sufficient information to enable the general geology to be identified for the highway design with nominal bridge sites located. The second stage would complete the Main Investigation with further boreholes located in the proposed bridge foundation areas (BD 2, DMRB 1.1), so that the design of the structures can be confirmed and a submission made for technical approval.

Guide to Manual

1.20 At scheme preparation and contract preparation stages, the Designer should classify the materials in order to enable him to make the appreciation of earthworks balance to which reference is made at Paragraphs 3.45, 3.46 and 4.33 to 4.38 below. To assist with this classification, reference should be made to MCHW 1, Series 600 and MCHW 2 Series NG600 and to Table 1/1 below. It is emphasised that the actual classification of soils in the Contract should now follow the guidance in Paragraph 2.22 of SA3 (MCHW 0.3.3) which superseded paragraph 3.5 of this document.

1.21 It should be remembered that this Advice Note deals only with design and preparation of contract documents for earthworks. In an actual contract when remeasuring for final account, the earthworks balance tabulation should be based on the actual classification of soils made by the Engineer or by the Contractor, as appropriate and construed in accordance with the Contract. This will generally depend on the content of Appendix 6/1.

1.22 Volumes 1 to 4 of the MCHW dated December 1991, published by HMSO, were promulgated by SD 1, SD 2 and SD 3 (MCHW 0.2.1, MCHW 0.2.2 and MCHW 0.2.3), the August 1993 amendments were promulgated by SD 6 (MCHW 0.2.5).

Implementation

1.23 The amended Advice Note should be used forthwith for all schemes currently under preparation, unless, in the opinion of the Overseeing Organisation, this would result in significant additional expense or delay progress. Design Organisations should verify the use of this Advice Note with the Overseeing Organisation.

TABLE 1/1

SPECIFICATION CLAUSE		RELEVANT APPENDIX	CHAPTER IN HA 44/91
Clause	Title		
601	Classification, Definitions and Uses of Earthworks Materials	6/1, 6/2, 6/3, 6/9	3, 4, 5
602	General Requirements	6/1, 6/2, 6/3, 6/7, 6/8	3, 4
603	Forming of Cuttings and Cutting Slopes	6/3, 6/10	7
604	Excavation for Foundations		7
605	Special Requirements for Class 3 Material	6/1, 6/4	5
606	Watercourses	6/3	12
607	Explosives and Blasting Used for Excavation	6/3	7
608	Construction of Fills	6/1, 6/3, 6/9,6/12,6/13	4, 5, 8,
609	Geotextiles used to Separate Earthworks Materials	6/5	5
610	Fill to Structures	6/6	11
611	Fill above Structural Concrete Foundations	6/6	4
612	Compaction of Fills	6/3, 6/9	4
613	Sub-formation and Capping	6/7	10
614	Cement Stabilisation to Form Capping		10
615	Lime Stabilisation to Form Capping	6/7	10
616	Preparation and Surface Treatment of Formation	6/7	10
617	Use of Sub-formation or Formation by Construction Plant		10
618	Topsoiling, Grass Seeding and Turfing	6/8	12
619	Earthwork Environmental Bunds	6/9	4, 8, 12
620	Landscape Areas	6/9	4, 8, 12
621	Strengthened Embankments	6/9	5, 8, 11,
622	Earthworks for Reinforced Earth and Anchored Earth Structures	6/1, 6/3, 6/9	11
623	Earthworks for Corrugated Steel Buried Structures		11
624	Ground Anchorages	6/10	11
625	Crib Walling	6/10	11
626	Gabions	6/10	11

TABLE 1/1 (Continued)

SPECIFICATION CLAUSE		RELEVANT APPENDIX	CHAPTER IN HA 44/91
Clause	Title		
627	Swallow Holes & other Naturally Occurring Cavities	6/11	9
628	Disused Mine Workings	6/11	9
629	Instrumentation and Monitoring	6/12	4
630	Ground Improvement	6/13	9
631	Earthworks Materials Tests		4
632	Determination of Moisture Condition Value (MCV) of Earthworks Materials		4
633	Determination of Undrained Shear Strength of Remoulded Cohesive Material		4
634	Determination of Saturation Moisture Content (SMC) of Chalk		5
635	10% Fines Value and Other Tests of Particle Soundness		4
636	Determination of Effective Angle of Internal Friction (Φ') and Effective Cohesion (c') of Earthworks Materials		4
637	Determination of Resistivity to Assess Corrosivity of Soil, Rock or Earthworks Materials		11
638	Determination of Redox Potential to Assess Corrosivity of Earthworks Materials for Reinforced Earth and Anchored Earth Structures		11
639	Determination of Coefficient of Friction and Adhesion between Fill and Reinforcing Elements or Anchor Elements for Reinforced Earth and Anchored Earth Structures		11
640	Determination of Permeability of Earthworks Materials		4, 10
641	Determination of Available Lime Content of Lime for Lime Stabilised Capping		10
642	Determination of the Constrained Soil Modulus (M*) of Earthworks Materials		11
Table 6/1	Acceptable Earthworks Materials: Classification and Compaction Requirements	6/1	3, 4
Table 6/2	Grading Requirements for Acceptable Earthworks Materials	6/1	2, 3, 4
Table 6/3	Limits of Material Properties of Fill for Use with Metal Components in Reinforced Earth and Anchored Earth Structures for Class 6H, 6I, 6J, 7C and 7D Materials.		11
Table 6/4	Method Compaction for Earthworks Materials: Plant and Methods	6/3	4
Table 6/5	Grass Seed Mixture per 50kg	6/8	12

NOTES:

1. See also MCHW 2, Series NG000, Table NG0/1, page 7, List of sub-clauses which refer to Contract-specific requirements described in Numbered Appendices 6/1 to 6/13 inclusive.

TABLE 1/2

Stages of a highway scheme and associated
geotechnical work

Geotechnical Work	Scheme Stage	Scheme Report Stage
(a) Procedural Statement-1 (b) Preliminary Sources Study(Desk Study) and preliminary investigation if required. (c) Procedural Statement-2	Admission to Road Programme Pre Public Consultation Consultation & Route Announcement	Preliminary Report
(d) Main Investigation (e) Geotechnical Certificate 1 Consultation & Route Announcement	Pre Public Inquiry Publication of Draft Orders Objections Period & Inquiry	Order Publication Report Factual Report Interpretative Report
(f) Supplementary Procedural Statement-2 (if required) (g) Supplementary Investigation (if required) (h) Supplementary Geotechnical Certificate 1 (if required) (i) Geotechnical Certificate 2	Pre Contract Tender (Post order making) Works Commitment Stage, Construction Stage Post-Construction Maintenance Period	Earthworks Design Report Notes to Resident Engineer
		Geotechnical Feedback Report

2. GROUND INVESTIGATION

Ground Investigation

2.1 Advice on carrying out GI is given in Advice Note HA 34/87 `Ground Investigation Procedure' which should be referred to for further details. Using the Preliminary Source Study (PSS) as a basis, the Ground Investigation will need to be planned in considerable detail to ensure that samples of sufficient size, type and number are obtained from each soil or rock type. This is necessary to enable the requisite tests to be carried out. Tests will need to be selected bearing in mind the information that is required for the design of embankment foundations, the design of slopes, for the selection of appropriate acceptance criteria for fill and to assess the subgrade properties. It is essential that this latter subject is not left until the preparation of contract documentation but is considered at the early stages of GI and is developed throughout the design process. The GI should also provide sufficient information for Tenderers in order to price the Contract and for the Contractor to decide on types of construction plant and programming. Long-term monitoring of ground water data should form part of each investigation. Further advice is given in Chapter 4.

2.2 Designers should be aware that tests required by SHW may not necessarily be standard tests described by British Standards or other references used as a standard. The most obvious example of this are the gradings set out in Table 6/2 of the SHW where sieve sizes greater than 75mm have been quoted. The BS 1377 tests for determination of the particle size distribution of a soil sample does not cater for this. The Designer will, therefore, have to ensure that tests he specifies in the GI embody the extra requirements of the SHW.

2.3 Consideration should also be given to materials whose characteristics can alter on exposure or during remoulding. For instance, there is considerable evidence available to suggest that pyritic shales exhibit increased acidity and sulphate content on exposure or when placed in embankments. Testing at the GI stage may not indicate this and in these cases the Standard BS 1377 test for sulphate content is not appropriate, and specialist advice should be sought. Stabilization of materials may be an option for capping layers and so the GI should include adequate sampling and testing for sulphates, organics and Atterberg Limits.

2.4 In the engineering discipline, the definitions and distinctions between the terms `soil' and `rock' are usually based on measures of particle size and either hardness, durability, inertness, or any combination of these. The accepted values for these criteria are governed by the intended use of the material. Furthermore the use of `soil' or `rock' can have implications for administering and costing projects. The result has been a number of definitions for each specific field of work. In the `Specification and Method of Measurement for Ground Investigation', soil and rock are defined, in paragraph 2.2, in terms of either the type of excavation tool, or drill bit required, or size of boulder encountered, in a ground investigation. The purpose in defining material in this way is to form a basis for payment for the ground investigation and for administering the contract. This definition is not used in other applications such as classifying fill materials. The SHW, appropriately, prefers to use `material' and `classes of material' so avoiding any confusion of terms. Where the word `rock' is used, in Clauses 603.5, 603.6 and 604.1, it refers to insitu rock or to argillaceous rock and is defined for use in selected fills. There is no such term as `rock in excavation' in the SHW and for excavation purposes `Hard Material', as defined in the MMHW (see Paragraph 3.18), should be assessed from the investigation drilling, rate and type of recovery, insitu tests, trial pits and exposures. As far as this Advice Note is concerned, rock, soil, or just material are used as general engineering terms and no strict contractual definition should be inferred. This reflects the differing terminology found in earthworks design, assessment, and documentation, all of which are subjects covered by this Note. (See also Chapter 3 on rock and paragraphs 4.51 and 4.52).

2.5 The extent of `Topsoil', as defined in SHW Table 6/1, should be determined from the GI and tests. (See also Chapter 12).

Sampling and Testing

2.6 The properties of engineering soils and rocks vary considerably not only from one geology to another but within the same geology and it should not be assumed, without full supporting evidence, that the behaviour of materials in apparently identical geological environments will be the same. Consequently representative and thorough sampling techniques and accurate testing, both insitu and in the laboratory, are required. `Site Investigation Practice: Assessing BS 5930' includes many useful references.

2.7 Difficulties arise because no two soil or rock samples will give exactly the same test results although they may well perform in a similar manner from an engineering point of view. Classification systems have evolved to sort materials into groups which all behave in a similar manner under particular conditions. The limits that are applied in sorting materials into the different groups or classifications may be somewhat arbitrary. However, the system does provide a unified approach to assessing the characteristics of a mass of material rather than attempting to assess the characteristics of innumerable individual samples.

2.8 The classification set out in SHW Table 6/1 has been developed to suit the requirements of handling earthworks materials to construct embankments using defined compaction methods or procedures to produce a defined or pre-determined end product. The limits used may or may not be the same as those used for other classification systems. For example, there is a conflict with the British Soils Classification Systems (BSCS) in BS 5930. Following the theme set in Paragraph 2.2 above, soil descriptions provided in ground investigation documents to BS 5930 may not accord with descriptions resulting from classifications in accordance with SHW Table 6/1. Designers should ensure that where descriptions and classifications of materials are given the system used is clearly described.

2.9 One of the objects of an investigation is to provide soil and rock test results to investigate the engineering properties of the materials which can be used in the design. There are a number of factors which must be considered to produce as representative results as is possible.

2.10 Borehole and trial pit location. The location of boreholes and trial pits is crucial. All the information from the PSS must be used as a guide to identify those locations which will provide the maximum amount of information about the surface and sub-surface geology. Critical areas identified in the PSS, such as soft ground, and areas associated with large structures, such as bridge foundations, will require particular attention.

2.11 Sample Location. Sample locations should be chosen to include those materials on which the design depends. Carefully planned sampling of this sort linking geological knowledge with the requirements of design is the main factor in determining sample location. Locations should also be chosen that include any expected variations in the properties of a material.

2.12 Sample disturbance. Methods of sampling should be chosen to produce samples which are only disturbed by the amount allowable in the required testing and to suit the extent to which the results are expected to be representative of the in-situ conditions.

2.13 Sample size. At normal rates of sampling any one sample many represent a mass of soil in a cutting at the ratio of typically 1:100,000 by weight. However, ultimately the size of sample will depend on the amount of material required for testing and the complexity of the geology.

2.14 Testing errors. Research has shown that considerable errors can be introduced at the testing stage even under controlled laboratory conditions. This is well described by Sherwood in TRRL Report LR 339.

2.15 It should be appreciated that the sampling procedure will have an effect on the reliance that can be placed on the soil properties being measured - see Table 2/1.

2.16 The number of tests carried out must be sufficient to draw conclusions from; 20-30 test results from a particular material would normally be required. This will obviously depend on the uniformity of the material in each area and how critical the results are to the design. The types of test will depend on the material property to be measured. The criteria to be used for determining the limits of the material property for acceptability depend on a number of factors given below.

2.17 The choice of material property and limits to be applied depend on:-

i. the material type;

ii. ease of application of test during the earthworks contract - speed of result required;

TABLE 2/1

Applications of the various qualities of soil samples derived
from different sampling procedures

<table>
<tr><th colspan="2">QUALITY CLASS</th><th>TYPICAL SAMPLING PROCEDURE</th><th>PROPERTIES EXAMINED</th><th>APPLICATION</th></tr>
<tr><td colspan="2">1</td><td>Piston thin walled sampler with water balance</td><td>Remoulded properties
Fabric
Water Content
Density and porosity
Compressibility
Effective strength parameters
Permeability
Coefficient of consolidation</td><td>Laboratory data on insitu soils.
Classification</td></tr>
<tr><td colspan="2">2</td><td>Pressed or driven thin or thick walled sampler with water balance</td><td>Remoulded properties
Fabric
Water Content
Density and porosity
Compressibility
Effective strength parameters
Total strength parameters</td><td>Laboratory data on insitu insensitive soils.
Classification</td></tr>
<tr><td rowspan="2">3</td><td>3A 100% recovery. Continuous</td><td rowspan="2">Pressed or driven thin or thick walled samplers. Water balance in highly permeable soils.</td><td rowspan="2">Remoulded properties
Fabric</td><td rowspan="2">Fabric examination & laboratory data on remoulded soils.
Classification</td></tr>
<tr><td>3B 90% recovery Consecutive</td></tr>
<tr><td colspan="2">4</td><td>Bulk and jar samples</td><td>Remoulded properties</td><td>Laboratory data on remoulded soils.
Sequence of strata.
Classification</td></tr>
<tr><td colspan="2">5</td><td>Washings. Disturbed samples from percussion boring in non-cohesive soils</td><td>None</td><td>Approximate sequence of strata only</td></tr>
</table>

(after Rowe (1972))

iii. reproducibility of test results;

iv. the amount of acceptable material available, earthworks balance;

v. the adaptability required - a wet period may reduce the quantity of acceptable on-site material and result in increased importation of off-site material.

vi. Engineering requirements:-

Stability of fill material.

Trafficability and compaction (CBR).

Control of settlements.

2.18 The criteria available for determining acceptability for the above are:-

i. upper and lower limit moisture content - applicable to all soils;

ii. optimum mc (determined from compaction test) + x% and -y%, limits mainly applicable to non-cohesive soils;

iii. LL. PL. and PI. Upper limits applicable to cohesive soils;

iv undrained shear strength (in-situ or remoulded) lower and sometimes upper limit applied to cohesive soils;

v. moisture content as a factor of PL, for application to most cohesive materials; one factor times PL for an upper limit and another factor times PL for a lower limit;

vi. Moisture Condition Value which can be found for a wide range of cohesive soils and certain granular soils.

2.19 Note that the choice of criteria will have a bearing on the investigation eg trial pits may be required for in-situ vane tests, large samples are required for remoulded shear strengths and the Moisture Condition test.

2.20 The choice of criteria should not be made until the results of a number of different classification tests have been analyzed. The limits for acceptability will probably be chosen in the light of the amount of acceptable material required and the test results obtained. Both these processes are very much aided by plotting and tabulating the results. The choice of limits and consequential volume of acceptable material can be termed `prediction'. This process can be aided by carrying out simple statistical analysis. The presentation of data in this form will assist in assessing the effects of varying limits on volumes of acceptable and unacceptable material should this become necessary at any stage of the project.

2.21 Consideration should be given to test data that show erratic variations from what might be expected as they may indicate possible problem areas.

2.22 Large quantities of data can best be handled by computers. There is considerable benefit to be gained by using computers at an early stage, preferably during the GI, to store the relevant data for retrieval and analysis later.

Planning the Earthworks Design

2.23 The design of a highway scheme develops with time, but the initial feasibility stage is often critical in establishing the probable horizontal and vertical alignment. As the scheme develops and more information becomes available from the investigations of the site the line may be varied to a lesser or greater degree. The line of the highway is still, however, broadly fixed and the materials that are available for the earthworks can be deduced. In arriving at the final earthworks design a number of factors should be carefully considered in order to produce a safe and economic scheme.

2.24 The approximate quantities of material required for the fill areas, including pavement construction, should be matched by the quantities of acceptable material likely to arise from the excavations. Due allowance should be made, except for measurement purposes, for bulking or shrinking and compaction. It may be possible to change the road line slightly either to avoid undesirable materials and areas or to increase the available quantity of a particular class of material (see Paragraphs 4.1 to 4.32). A perfect earthworks balance will, however, rarely be obtained in practice.

2.25 The basic physical properties of the insitu materials are fundamental to the initial earthworks design and will determine the shape of the cuttings and embankments. The designer may use interpretative information to assist in his design. The relative location of each cut and fill area, the approximate quantities in each and the distribution of the various materials within or below these earthworks all affect the design. There are also external factors which will effect the earthworks, such as land-take restrictions in urban areas, the visual intrusion of the earthworks into the surrounding rural countryside, limitations on the access along the site caused by railways, roads, rivers etc. All these must be considered in the light of how they will affect excavating the material from the cuttings and placing it in the desired sequence into the embankments, and they will be more fully discussed later in this document. It is prudent at the PSS stage (Desk Study) to examine similar soils in any nearby project for initial assessment of the earthworks design. The local authority's view on the use of a particular soil is also likely to be of value.

3. SPECIFICATION AND METHOD OF MEASUREMENT FOR HIGHWAY WORKS

General

3.1 The information given in this Chapter complements the SHW 600 Series Notes for Guidance and provides assistance on completing the associated numbered Appendices. The object of the Specification is to provide the Contractor with sufficient detail that he may provide materials and work in such a manner that the end product is that required by the DTp. The 600 Series Specification is a mixture of `end product' and `method' specifications. The Notes for Guidance are to assist the Designer in preparing the Specification and this Advice Note provides technical details and background to assist in this preparation.

3.2 The Method of Measurement (MoM) includes the Item Coverages which are incorporated in the Contract by the provisions of the Preamble to the Bill of Quantities. The MoM sets out the basis on which payment is made and the ground rules for preparing the Bills of Quantities. It is essential that amendments to existing Specification clauses or new clauses are followed up with appropriate amendments to the MoM. The Method of Measurement for Highway Works (MMHW) defines the coverage of items in Bills of Quantities and their Method of Measurement for use in construction DTp highways.

Additional information for the Notes for Guidance

3.3 The following guide indicates where advice may be obtained for a particular note in the Notes for Guidance.

NG600.1 Design of earthworks; refer to Paragraphs 2.6 and 2.22 and Chapters 7 and 8. Selection of limits for soil properties and preparation of Appendix 6/1; refer to Chapter 4.

NG600.4 Examples of Appendices are given in Annex A.

NG601.7 Limits for acceptability; refer to Chapter 4.

NG605.3 and 4 Designation of Class 3 material, compaction of chalk and times of the year when chalk earthworks may not be carried out; refer to Paragraphs 5.8 to 5.20.

NG613.3 Advice on subgrade assessment and appropriate capping materials is given in Chapter 10.

NG618.2 and 4 See advice given in Chapter 12.

NG621.2 Properties of Geotextiles and Geomeshes are given in Paragraphs 5.72 to 5.89.

NG640.1 Permeability tests for earthworks materials are described in Chapter 4 and 9.

Advice on the Specification

3.4 Table 1/1 Advice Note Guide shows where advice on each SHW Clause may be found within this Advice Note and also cross refers with the relevant Specification Appendix.

Testing for Classification and Acceptability

3.5 The SRBW did not set down who was responsible for testing for the purposes of classification and determining acceptability. The SHW Clause 602.1 is a little more helpful in directing the reader to Appendix 6/1 which in turn indicates that the Designer is to state, amongst other things, `Special Requirements for determining acceptability, who classifies and where, and whether trial pitting is required'. It is therefore recommended that generally the Contractor should be given responsibility for classification and determining acceptability based on the Designers limits and criteria including the appropriate sampling and testing for compliance. In some circumstances, however, it is possible that the Engineer may be more appropriate to take this responsibility.

3.6 If the Contractor is given the responsibility for determining the acceptability of all the materials used on site, he will need to carry out the required sampling and testing to prove to himself and to the Engineer that the correct materials are being used. Therefore the Engineer must specify what tests the Contractor shall carry out and give a frequency of testing for each test together with any other relevant information. Providing the contract makes it absolutely clear what and how much testing the Contractor is expected to carry out, then payment is allowed under Clause 36 of the CoC. For this reason sampling and testing has not been included in MMHW item coverage and no separate BoQ items are included for them. Suggested frequencies of testing are given in Paragraphs 3.8 to 3.12 but the Designer would be well advised to ensure that his estimate of the quantity of testing given in Appendix 6/1 is generous. Whatever sampling and testing is carried out by the Contractor, this does not affect the Engineer's own tests, although the frequency of sampling and testing by the Engineer's staff should not be as high as the Contractor's. Whoever is made responsible for the sampling and testing, all the relevant information should be included in Appendix 6/1.

3.7 If in Appendix 6/1 the Engineer undertakes the responsibility for classification and determining acceptability, then this should not detract from the Contractor's overall responsibility to provide acceptable material as defined in the Contract.

3.8 If testing is the responsibility of the Engineer then the Contract should include details of all the apparatus and equipment required to equip the Engineer's site laboratory, as well as the usual services. If testing is the Contractor's responsibility, then in addition to the Engineer's laboratory, a list of tests together with the source reference for each test (BS, ASTM, SHW etc) which the Contractor is expected to carry out must be included in Appendix 6/1. Details about the submission of test results to the Engineer must also be set out. It is already the Department's policy that all permanent testing laboratories should be accredited by the National Measurement Accreditation Service (NAMAS) as part of their Quality Control policies: accreditation of site laboratories will follow at a later date. No mention should be made as to where the Contractor should carry out his testing ie his own site laboratory, independent testing laboratories etc, this should be left for him to organise.

3.9 In order for the Contractor to be able to include for testing in his rates, the frequency of each test should also be stated.

3.10 The tests fall into two categories:-

i. tests involving taking representative samples from materials and carrying out the test either on the site, or in a site or other laboratory;

ii. insitu tests, such as plate bearing tests, and tests for controlling method compaction.

3.11 Compliance sampling and testing should be carried out at excavation for on-site materials unless the material is likely to change between excavation and deposition, in which case further sampling and testing should be carried out at deposition. Imported materials should be sampled and tested for compliance at deposition and preferably at source. Appendix 6/1 should state the locations of all compliance sampling and testing. The onus is placed on the Contractor for maintaining the acceptability of material.

3.12 The scale of testing depends very much on the size of the contract, uniformity of material and how critical the results are to the design, and so the frequency of testing given in Table 3/1 is given only as a guide.

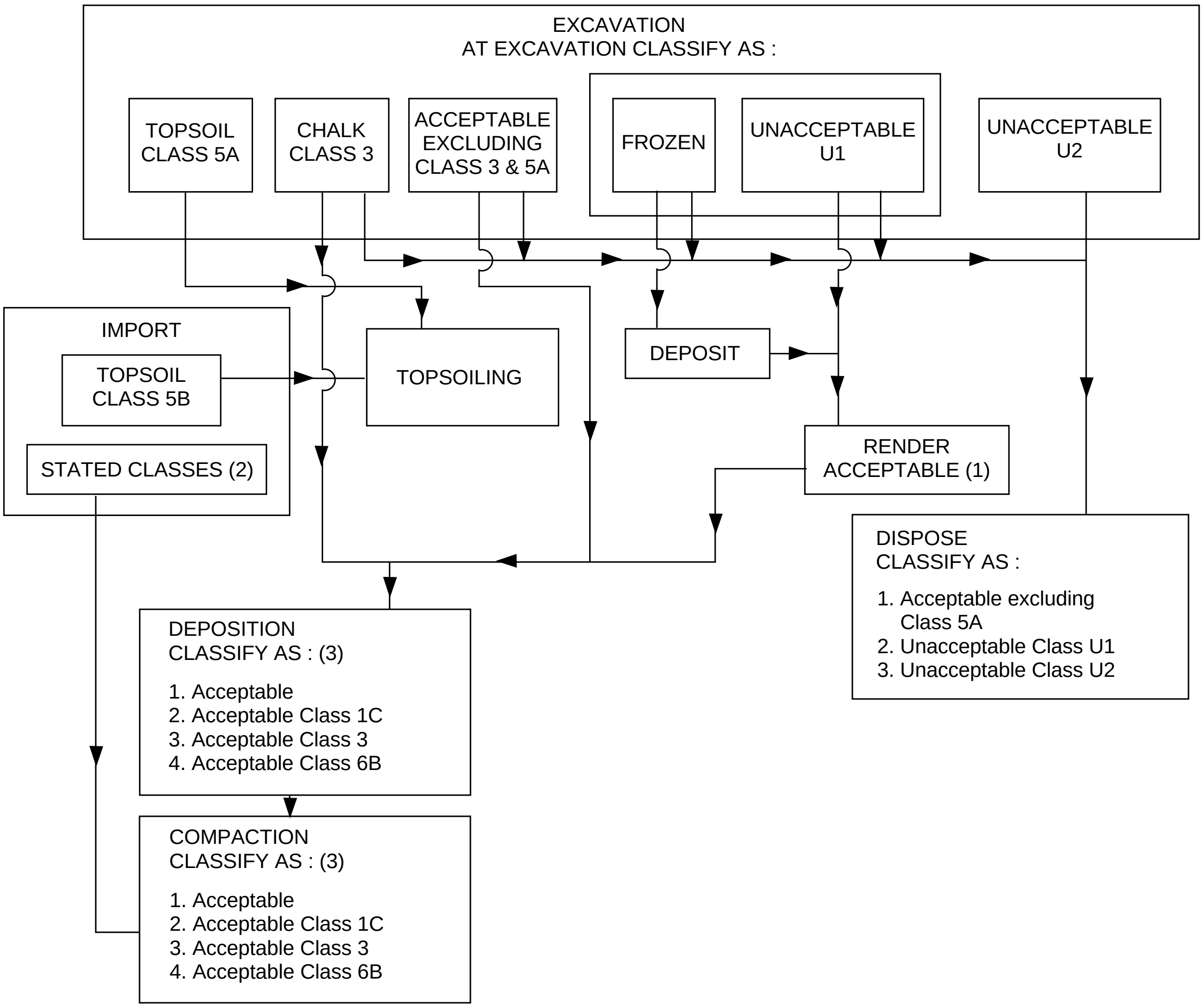
3.13 For the purposes of measurement, the MMHW requires material in excavation to be classified as topsoil Class 5A, chalk Class 3, other acceptable materials or unacceptable material Classes U1 and U2.

3.14 For measurement purposes, the MMHW requires material at deposition and compaction to be classified as acceptable, Class 1C, 3 and 6B, although Class 1C and 6B should only be measured separately when they are specifically stated in the Contract to be placed in a particular location. For practical purposes on site the Engineer will need to subdivide existing Classes further to ensure that the correct acceptability criteria and compaction method is applied to the material and also where zoning is required the correct Class of material is placed in that zone. Measurement will then follow as appropriate. Figure 3/1 of this Advice Note shows the classification requirements for measurement purposes during the various earthworks operations.

TABLE 3/1

Suggested frequency for classification and acceptability testing

Material Class	Requirement	Suggested Frequency Per Source
1 General Granular Fill	Grading/Uniformity Coeff	Twice a week
	mc/MCV	1-2 tests per 1000m^3 of material up to a max 5 per day
	SMC of Chalk	Twice a week
2 General Cohesive Fill	Grading	Twice a week
	mc/MCV/PL/Shear strength	1-2 tests per 1000m^3 of material up to max of 5 per day
	SMC	Twice a week
	Bulk Density (PFA)	1-2 tests per 1000m^3 of material up to max of 5 per day.
3 General Chalk Fill	mc	1-2 tests per 1000m^3 of material up to max 5 per day
	SMC	Daily
4 Landscape Fill	Grading/mc/MCV	Daily
5 Topsoil	Grading	Daily
6 Selected Granular Fill	Grading/Uniformity Coeff	1 test per 400 tonnes of material
	PI/LL	Daily
	10% Fines Value/SMC	Weekly
	omc/mc/MCV	1 test per 400 tonnes of material
	Organic Matter/ Total Sulphate Content	As required or weekly
	pH/Chloride Ion Content	As required or weekly
	Resistivity	As required
	Undrained Shear Parameters	As required
7 Selected Cohesive Fill	Grading/mc/MCV	1 test per 400 tonnes of material
	SMC	Twice a week
	PI/LL	As required or daily
	Organic Matter/ Total Sulphate Content	As required or weekly
	pH/Chloride Ion Content	As required or weekly
	Resistivity	As required
	Undrained and Drained parameters	As required
	Permeability	As required
	Coeff of Friction/Adhesion	As required
8 Miscellaneous Fill	mc/MCV	Daily
9 Stabilised Materials	Pulverisation	1 test per lane width per 200m length
	mc/MCV	1 test per lane width per 200m length
	Bearing Ratio	1 test per lane width per 200m length



NOTES :

(1) If the Contractor opts to render unacceptable material acceptable for use in the Works, then he will be paid as though he had disposed and then imported the class of material he rendered acceptable. If the contract requires material to be so treated then payment will be at rates in the BoQ.

(2) For Import, only Stated Classes have to be classified (Group I, Feature 2)

(3) Deposition and Compaction of Class 1C and 6B materials shall be measured separately only where Class 1C or 6B material as such is specifically stated by the Contract to be required to be placed and compacted in a particular location.

Figure 6/1 Classification during earthworks operations.

Advice on the Method of Measurement

3.15 Part I of the MMHW provides definitions for `Hard Material' and `Existing Ground Level' and Part IV, Section 6: Earthworks defines `Earthworks Outline', `Sub-soil level', `Surcharge' and further amplifies `Existing Ground Level'.

3.16 It is required that the Bill of Quantities is divided into parts. Earthworks as described in this Advice Note will fall within both the Roadworks and Structures parts of the Bill.

3.17 The earthworks items fall under the MMHW headings of:-
Excavation
Excavation in Hard Material
Deposition of Fill
Disposal of Material
Imported Fill
Compaction of Fill

3.18 The MMHW defines `Hard Material' as follows.

i. Material which requires the use of blasting, breakers or splitters for its removal but excluding individual masses less than 0.20 cubic metres.

ii. Those strata or deposits so designated in the Contract.

3.19 Hard Material is paid for as an `extra over' item which means that an increased rate is paid over normal excavation rates. The Designer must make an assessment of where and how much hard material will occur in excavations and indicate this in the contract drawings (see Paragraphs 3.39 and 3.40). He will also need to assess whether the hard material, on excavation, will be acceptable. Many hard materials may be suitable for use as one of the selected granular materials. The Designer will have to assess whether to designate any hard material for use as selected fills and whether to make allowance for rendering unacceptable material into acceptable material or leave both decisions to the Contractor. Whichever approach is adopted should be made clear in the contract documents.

3.20 It is recommended that no more classification than is required for measurement purposes is set out in the Contract. For excavation purposes this will be:-

1. Acceptable Topsoil Class 5A
2. Acceptable Chalk Class 3
3. Acceptable Material excluding Classes 3 and 5A
4. Unacceptable U1
5. Unacceptable U2

3.21 These are the MMHW Group II Features. The associated Group III Features are:-

1. Cutting and other excavation
2. Structural foundations
4. New watercourses
5. Enlarged watercourses
6. Intercepting ditches
7. Clearing abandoned watercourses
8. Removal of surcharge

3.22 Feature 2 will usually apply only to the Structures Bill. Feature 2 is further subdivided by Group IV Features:-

1. 0 metres to 3 metres in depth
2. 0 metres to 6 metres in depth and so on in steps of 3 metres.

3.23 For deposition purposes the drawings must show the locations of selected materials (see Paragraphs 3.39 and 3.40). However, measurement will fall under the following MMHW Group II Features except that the deposition of Class 1C and 6B materials should be separately measured only when Class 1C or 6B material as such is specially stated by the Contract as required to be placed in a particular location.

1. Acceptable material
2. Acceptable material Class 1C
3. Acceptable material Class 3
4. Acceptable material Class 6B

3.24 These will have to be sub-divided into the following MMHW Group III Features where appropriate.

1. Embankments and other areas of fill
2. Strengthened embankments
3. Reinforced earth structures
4. Anchored earth structures
5. Landscape areas
6. Environmental bunds
7. Fill to structures
8. Fill above structural concrete foundations
9. Fill on sub-base material, roadbase and capping
10. Fill on bridges (under footways, verges and central reserves)

3.25 It is worth noting the acceptable material excavated for structural foundation may not be suitable for backfilling. It is generally accepted that all this material is billed as being deposited as `acceptable material' in Group III Features 1 or 5.

3.26 The drawings must make clear the dividing line between `fill to structures' and `fill above structural concrete foundations'.

3.27 Unless selected materials can unquestionably be won from the site it is recommended that they are treated as imported materials. For imported materials the MMHW allows the following Group I Features.

1. Imported acceptable material
2. Other stated classes of imported acceptable fill
3. Imported topsoil Class 5B

Imported topsoil Class 5B shall not be identified by any Group II Features.

3.28 Figure 3/1 shows the classes which fall into the MMHW categories. It can be seen that for import only Stated Classes require classification; imported acceptable material does not require classification. All imported materials must however, be specified as Group I Features 1, 2 or 3. It is recommended that Group I Feature 2 be used when Class 6, 7 and 8 materials and material to SHW Clause 512 are to be imported. Where Class 1, 2 and 3 materials are to be imported, either Group I Feature 1 or Group I Feature 2 may be used, depending on whether it is necessary to distinguish between the classes or not. Further discussion on the classes of materials is given in Chapter 4.

3.29 However, for the purpose of assessing which materials may be considered for import the following information may be useful:-

3.30 If PFA is required as general fill then Class 2E must be stated.

3.31 Where chalk does not fall into the categories of Class 3 or unacceptable it may be included in Classes 1A, 1B, 1C, 2A, 4, 6A, 6B, 6E, 6F, 6H, 6I, 6J, 6P, 7A and 8.

3.32 Except in exceptional circumstances, Class 4 material should not be imported. Limits should be set so that sufficient site arising material is available. If there is a shortfall then Classes 1, 2 or 3 material should be specified and used.

3.33 Class 5A cannot be imported.

3.34 Class 6E, 7E and 7F materials for stabilisation are not intended for import as a design option. Wherever possible stabilisation should be allowed for as an option in the Contract.

3.35 Where there is an obvious excess of acceptable site material the Designer should make every effort to reduce the quantity of imported selected fills by means of designs using lesser quality but acceptable materials. An example would be to design a slightly more costly structure which could tolerate acceptable site arising material rather than expensive imported granular backfill material. Where selected materials arise on site the Designer should not indicate their location but he must be able to justify his design and may have to back up his decision in the event of a dispute. It can be seen that there is no necessity for the Designer to break down acceptable material into the various classes in the excavation but nevertheless he should be satisfied that the materials available meet the design requirements.

3.36 Sufficient detail must be given on the drawings for the Contractor to assess how much of each class of fill material is required and to determine where it is to be placed. The Designer will need to determine quantities of each class for his own purposes. The drawings will need to show where each class of material is to be placed, but the quantities shown will only be those required by the MMHW (see Paragraphs 3.23 to 3.26).

3.37 Compaction follows the same measurement rules as Deposition: only Classes 1C, 3 and 6B are specifically stated, the remainder being included in the term `acceptable material'. As stated in Section 6, Para 43 of the MMHW, Class 1C and 6B are only measured where they are specifically required in the Contract to be placed at a particular location.

3.38 In his prices, the Contractor is to include the requirements of SHW Clauses 602.15 and 16. However items need to be included in the BoQ for any requirements under SHW Clause 602.17. If the Contractor elects to render unacceptable material into acceptable material (SHW Clause 602.7) then he should be paid as though the material concerned was disposed of and a similar quantity imported. If the Engineer wishes to carry out a similar operation then, under SHW Clause 602.17, the necessary requirements shall be set out in SHW Appendix 6/1 and paid for by an item in the BoQ.

Preparation of Contract Drawings and Earthworks Quantities

3.39 For a scheme which requires the excavation of cuttings and construction of embankments the following information will need to be shown.

i. Areas where topsoiling is required and the specified thickness and whether the slope is greater than 10 to the horizontal or at 10 or less.

ii. Areas as above for grass seeding, turfing or hydraulic mulch grass seeding.

iii. Areas where any previous pavement is to be perforated or broken up and left insitu.

iv. Areas where any previous road pavement is to be removed and to be measured as `hard material'.

v. Areas where any special treatment is required such as:-

- benching;
- special embankment foundations - dig out and starter layers, backfill, selected fills etc;
- lengths of strengthened or reinforced embankment shoulders;
- surcharge;
- cut/fill transition zones.

vi. Earthworks limits and batter slopes required.

vii. Landscape areas and extent of amenity bunds.

viii. Noise measuring stations.

ix. Extent of earthworks to structures.

x. Extent of earthworks to side roads and interchanges.

xi. Extent of earthworks to main carriageway.

xii. Quantities of excavation, deposition and hard material, with a clear statement as to what is, or if necessary, what is not designated as hard material for payment purposes.

xiii. Details of special treatments (see (v)).

xiv. Exploratory holes in plan and section showing groundwater strikes, standpipe/observation well levels, extent of hard material, defined as either strata or within limits defined by levels, shear strength, MCV and Atterberg Limit test parameters.

3.40 It is suggested that the above information should be shown on the drawings in the following format:

A. 1:25000 scale drawing showing geological plan with route superimposed.

B. 1:2500 scale plan drawings and 1:2500 horizontal, 1:250 vertical scale longitudinal sections, showing exploratory holes, items (xiv) above formation level on sections, centre line on plans and simplified geology at exploratory hole locations. Larger scales such as 1:1250 horizontally and 1:125 vertically will be necessary for clarity where there is a preponderance of information.

C. 1:2500 horizontal, 1:250 vertical scale longitudinal sections showing the earthworks and detailing quantities of excavation and deposition; item (xii) above.

D. 1:500 scale plan drawings showing items (i), (ii), (iii), (iv), (v), (vi), (vii), (viii), (x), (xi) above. More than one volume of drawings will be required to accommodate all these items.

E. 1:500 scale plan drawings showing item (ix) above.

F. 1:100 or 1:500 plan drawings showing standard and special details; item (xiii) above.

Interpretative information should not be included in the contract documentation.

3.41 The earthworks to structures drawings (E) should indicate the extent of filling to each structure and fill above structural concrete foundations in order that the quantity and location of each class of material may be determined. The actual quantities should be included on the earthworks drawings' longitudinal sections (C) along with all the other quantities.

3.42 It is convenient to divide the earthworks into sections corresponding with the following:-

Cuttings   if large volumes involved, each
Fills   cutting or embankment may be subdivided
Structures
Interchanges
Side roads
Landscape areas
Accommodation works
Others including watercourses and drainage lagoons.

3.43 These sections should reflect the separate treatment of earthworks in the various Bills under the headings of Roadworks and Structures as described in Paragraphs 3.16 and 3.17. Thus the structural earthworks associated with a bridge spanning a cutting will be kept separate from the cutting earthworks quantities. The following information in terms of earthworks quantities should be provided where appropriate for each section:-

Excavate Topsoil (Class 5A)
Excavate Acceptable Material Class 3
Excavate Acceptable Material other than Class 3 and 5A
Extra Over Excavation for Excavation in Hard Material
Excavate Unacceptable Material Class U1
Excavate Unacceptable Material Class U2
Deposit Acceptable Material Other than Class 3
Deposit Acceptable Material Class 1C*
Deposit Acceptable Material Class 3
Deposit Acceptable Material Class 6B*

* See MCHW 4.1 Chapter IV Series 600 paragraph 31.

3.44 This information can conveniently be displayed on the Drawings using a table of quantities for each earthwork. For billing purposes, quantities will need to be entered on the earthworks schedules which includes totals for the whole Contract for each group of materials. Proformas for completion are given in MCHW 4.2, Chapter IV, Series 600, pages 6 and 7.

3.45 Where there are major obstacles to the free passage of earthmoving machinery through the site (eg rivers, railways, canals, major roads) then the earthworks need to balance between them by minimising import and/or export of material, using the principle of making the most economic use of materials available on site; this results in sub-divisions of the quantities of embankments and cuttings which extend over these haulage barriers. This aspect needs early consideration by the Department and by the Designer, if yet appointed, when the proposed works are being split into a series of discrete schemes. It should be remembered that what constitutes an 'obstacle' is subjective and depends on the amount of money which it is economic to expend to overcome it. Although responsibility for such decisions remains with the Department and the Designer up till contract award, it should also be remembered that the Contractor then assumes ultimate responsibility in most cases and will usually make the final decision as to whether soil for earthworks will be hauled past an 'obstruction', unless there is an express restriction in the Contract to the contrary (usually in Appendix 1/13).

3.46 After an initial assessment of cut and fill balance, the Designer should then ensure that the earthworks quantities balance arithmetically in the bill of quantities within each contract within the scheme. This will best be done by use of MCHW 4.2, Chapter IV, Series 600, pages 7 and 6, "Typical Structures Earthworks Schedules" (sic) and "Typical Roadworks Earthworks Schedule", in that order. Where there is a shortfall in material for structures, the deposition and compaction of acceptable materials from elsewhere on site shall be measured with the relevant structure(s). In the case of surplus material arising from structures the deposition, compaction or disposal, as appropriate, shall be measured with roadworks. See also Paragraphs 4.33 to 4.38 concerning earthworks balance.

material from structures deposition, compaction or disposal, as appropriate, shall be measured with roadworks.

4. USE OF MATERIALS AND CONSTRUCTION

Assessing Acceptability Criteria

4.1 The main objective of acceptability assessment is to enable the scheme to be constructed to a satisfactory standard of design and longevity for the minimum cost. This requires the maximum use of on-site or locally available materials, and setting specifications for imported materials which are adequate for performance but not unnecessarily restrictive thereby incurring increased cost. The change in name from `suitability' to `acceptability' is intended to emphasize this desire to maximise the use of on-site materials, and also to demonstrate that although some material is not suitable for general or selected fills, it may be acceptable for other uses eg landscaping.

4.2 Clause 601.1 SHW defines acceptable material as that which meets the requirements of Table 6/1 SHW and Appendix 6/1 SHW, and both Table and Appendix have been arranged so as to allow the Designer to best utilise the available materials on each individual scheme. Table 6/1 SHW divides both on-site and imported materials into 9 principal classes, which are further sub-divided for compaction purposes or because of particular properties or applications. It also lists the criteria and relevant tests whereby a material may be identified as belonging to a particular class, together with the limiting values for some of those criteria which define whether the material is acceptable or not. Appendix 6/1 SHW is the means whereby the Designer can state values for those acceptability limits not already given in Table 6/1 SHW for the remaining criteria tests. This Appendix gives the Designer flexibility by allowing him to vary the acceptability limits of any fill material that would be acceptable in some circumstances and locations and not in others which may prevail on a site. Figure 4/1 sets out a recommended sequence for the use of Table 6/1 SHW and Appendix 6/1 SHW in the classification of materials and the setting of the acceptability limits for each property mentioned.

4.3 It is vital that site investigations should be thorough and carried out sufficiently in advance of the design stage to enable a proper appraisal to be made of the materials that will be encountered. The ground investigation should give comprehensive information regarding the insitu ground conditions, particularly concerning those conditions governing acceptability criteria as mentioned in Table 6/1 SHW. From the exploratory hole logs, sample descriptions, trial pit logs, exposures, and the results of those tests for which acceptability limits are already specified in the Table, a judgement can be made regarding which of the sub-divisions each material or stratum falls within, and whether it can be regarded as acceptable or not. The Designer now assesses which of the acceptability criteria he wishes to use for each material, eg MCV or moisture content, <u>since all of them may not be applicable at the same time</u>, and after consideration of the insitu properties of he materials, establishes desirable limits for those criteria he has chosen. In arriving at these criteria he must take into account the type of material ie cohesive or granular, and whether the tests are applicable to it, and also that the criteria will be used during construction to check compliance and acceptability and the tests should be relatively simple and robust. Table 4/1 gives a suggested list of basic tests which should help the Designer to assess the classification and acceptability of the materials. Additional tests may be required for specific and selected materials such as Saturated Moisture Content for chalk or 10% Fines.

4.4 It should be noted that material classifications made during the GI to the British Soil Classification System (BS 5930) must be reviewed against the requirements of SHW Tables 6/1 and 6/2 since they are not always compatible.

4.5 An important aspect of classification for the SHW and of soils in general is the Particle Size Distribution (PSD). Poor sampling techniques particularly with granular soils can mask the true profile, and particularly, percussive boring techniques are unlikely to provide samples of sufficient quality and size for accurate PSDs (see Chpater 2). A knowledge of the soils to be expected will enable the Designer to assess that the test results are those which might be expected for a particular soil type. As a guide engineering properties are mainly determined by the finest 25% of material in a soil sample. The coarser 75% does however contribute to the soil compressibility potential. Soil permeability, its ability to transmit water, is largely determined by the finest 10% to 15% of material. It can be seen, therefore, that a knowledge of the geological make-up of the soils and careful interpretation of the GI data is essential if a decision on the proper use of materials is to be made.

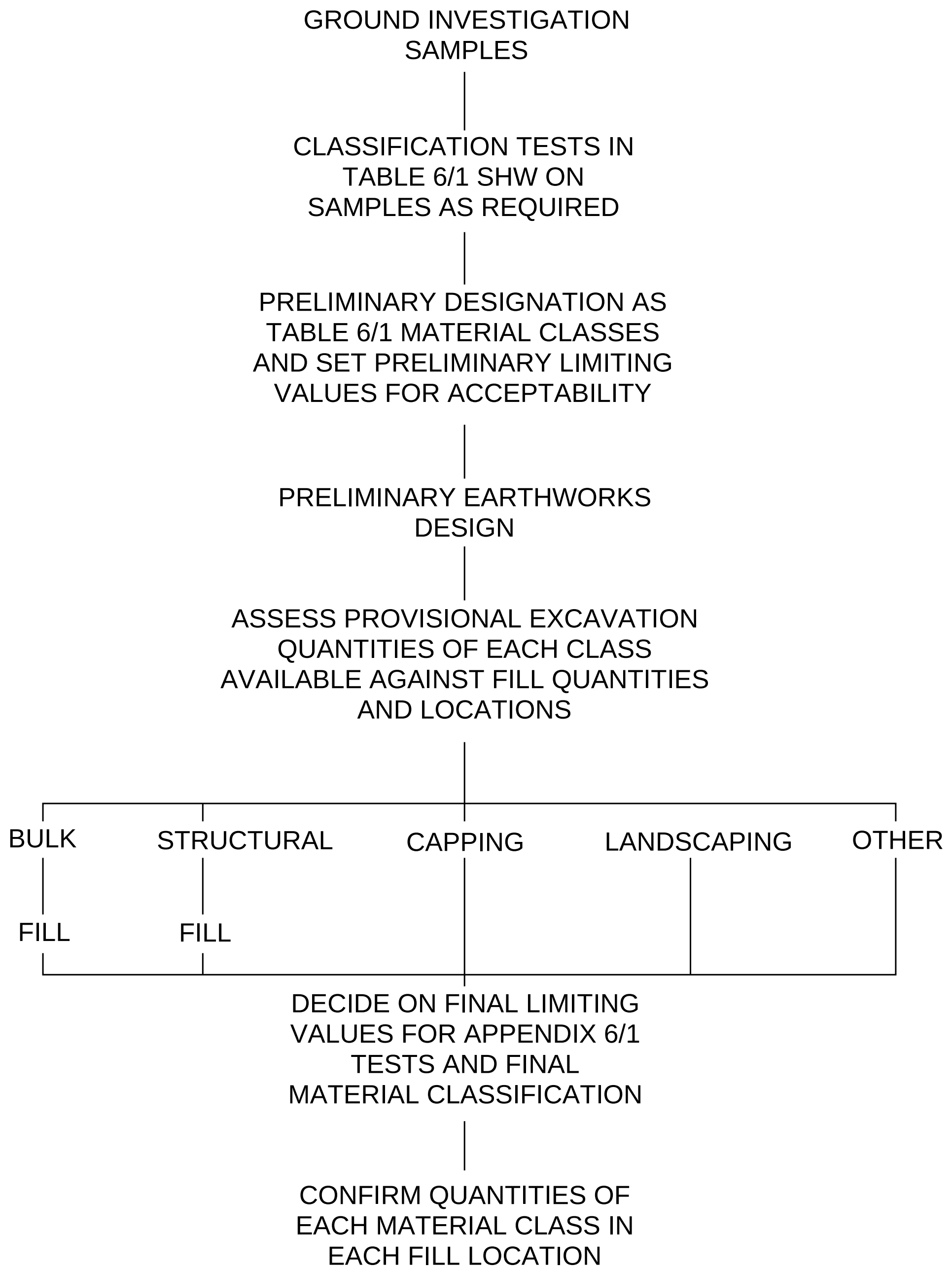


Figure 4/1 Earthworks classification assessment

TABLE 4/1

Classification and acceptability tests

TEST	APPLICABLE MATERIAL TYPE	PURPOSE	REFERENCE
Moisture content	All	Classification Stabilisation	BS 1377/BS 812
Atterburg Limits	Cohesive	Classification	BS 1377
Particle Size Distribution	All	Acceptability Classification	BS 1377*
MCV	Cohesive and/or some Granular	Acceptability Trafficability	Clause 632 SHW TRRL LR 1034 TRRL RR 130 TRRL RR 90 BS 1377
Maximum Density and Optimum Moisture Content	Mainly Granular	Acceptability Compatibility	BS 1377/BS 812
CBR	All except coarse Granular	Trafficability Stabilisation Classification	BS 1377 BS 1924 BS 1377
Triaxial (quick)	Cohesive	Acceptability Trafficability	BS 1377
Chemical Tests	All	Acceptability	BS 1377
Relationship Testing+	All	Acceptability	-

* BS 1377 Part 2 should be expanded to include all sieve sizes quoted in Table 6/2 SHW. See 2.2.

+ Testing soils at various moisture contents to study the change in soil properties.

NOTE: Further reference should be made to Paragraph 4.8 regarding a combination of a number of these tests to determine acceptability limits.

4.6 The preliminary earthworks design can now be completed with appropriate side slopes. The approximate quantities of the required fill volumes for each embankment or other fill area can then be assessed against the available excavated materials for all the various requirements such as general fill, selected fill and capping materials. The Designer may vary the limiting values for acceptability in order to maximise the use of the materials available. For instance a wet cohesive material may not be acceptable for use in a shallow embankment, but may be acceptable if placed within the lower areas of a larger embankment. Therefore a further sub-division of Class 2A would be made in Appendix 6/1 SHW to allow for this and the restrictions under which the wetter material could be placed would also be defined. However, care should be taken in the assessment of available quantities since a material could quality for more than one classification.

Assessing Acceptability Limits

4.7 When establishing the limiting values of the acceptability criteria the Designer should consider a number of factors which could influence his choice of values and therefore his choice of fill:-

i. Earthworks balance, haul distances and ease of transportation within the site.

ii. Availability of imported material from local borrow pits and ease of transportation to the site.

iii. Ability of the material to withstand site and construction traffic. Information regarding the effect of earthmoving plant on soil conditions can be found in TRRL LR 1034, SR 522 and RR 130.

iv. Ability of material to compact to a satisfactory density (grading and natural moisture content).

v. Moisture content and susceptibility to moisture.

vi. Frost susceptibility and rate of water supply through underlying soil.

vii. Possibility of improvement by groundwater lowering, drying.

viii. Chemical nature of material and affect on adjacent material, structures, pipes etc.

ix. Availability of material for landscaping, environmental bunds etc.

x. Location of material within the fill area. It may be possible to use poorer material in the core of an embankment.

4.8 A suitable method of arriving at the desirable limits for acceptability is by relationship testing. In relationship testing, material properties such as MCV, CBR, undrained shear strength and density are studied at varying moisture contents and the relationships found are used as a basis for various decisions. Relationship tests are used to:-

i. provide information on soil characteristics at a range of moisture contents,

ii. indicate susceptibility to a reduction in strength due to wetting and increased strength due to drying,

iii. help determine wet and dry acceptability limits for the optimum use of available materials,

iv. enable acceptability assessments to be carried out by different methods and approaches, and correlations to be made between them,

v. aid site control testing where sometimes only limited data, ie moisture content or MCV, are immediately available.

vi. help the Contractor assess plant requirements and assess haul distances.

4.9 Typically two or three relationship test packages are required for each material type. If the material is variable then relationships should be determined on the two `extremes' and on the `average' material. It is recommended that the emphasis on the spread of test points should be to the `wet' side of optimum to demonstrate the more critical behaviour of the material, ie CBR 2%, MCV 6 and shear strength 30kN/m^2. Additional test points should always be included if the initial spread is inadequate.

4.10 A recommended test package is shown in Table 4/2.

4.11 The use of relationship testing allows the Designer to relate various properties to each other by means of the varying moisture contents. For cohesive soils, after considering various matters such as trafficability (TRRL RR 130), stability and settlement, the Designer must decide upon a minimum shear strength or MCV. The upper limit is usually dependent upon the ability of the likely compaction plant to compact a very stiff or hard clay, but a flexible approach is recommended to avoid having to dispose of dry material from site which could be used as a satisfactory fill material. The relationship curves then allow these design values to be related to other properties, such as CBR and MCV, and enable a decision to be made on the most appropriate method of assessment and site control testing. For granular soils the dry density/moisture content relationship is probably the most relevant with the moisture content range which achieves 95% of the standard maximum density to BS 1377: Part 4 (2.5kg rammer method) being a reasonable yardstick to avoid later significant settlements within the fill.

4.12 The values that the Designer will enter for the acceptable limits in Appendix 6/1 SHW should reflect the results obtained from the comprehensive testing carried out on the ground investigation samples. No fixed figures can be given for the majority of these values, which will obviously vary for different soils within the same material class, but the following comments should be considered by the Designer when preparing Appendix 6/1 SHW.

4.13 Class 1 - General Granular Fill - Classes 1A and 1B both contain moisture content and MCV acceptability properties. Moisture content is usually recommended as the best option for specifying a granular material by means of the dry density/moisture content relationship from the material sample compaction curves and a permitted moisture content range may be deduced which achieves 95% of the standard maximum density to BS 1377: Part 4 (2.5kg rammer method). Typically an acceptable range could be a lower limit of 1% to 2% below optimum moisture content and an upper limit of 1% to 2% above optimum moisture content. This does not totally preclude the MCV being related to those moisture contents and used as the limiting acceptability values if it is demonstrated, by relationship testing, to be a responsive test on the particular granular materials encountered. However it is strongly recommended that either moisture content or MCV be used but not both. Either test can be carried out on site for compliance testing and they are both simple although moisture content determination is relatively slow compared to MCV. Further advice on the suitability and use of the Moisture Condition Apparatus with granular materials may be found on TRRL RR 90. In Scotland reference should be made to SDD SH 7/83.

4.14 Class 2 - General Cohesive Fill - Classes 2A, B, C and D all contain moisture content, MCV and undrained shear strength acceptability properties. The ground investigation samples should provide relationship curves for each material as described before. Typical acceptable strength limits for such cohesive materials could be a lower limit of 30-50kN/m^2 and an upper limit of 150-200kN/m^2. The lower limit would depend on factors such as trafficability or stability, and the upper limit on the ability of the material to be compacted successfully using the relevant method from Table 6/4 SHW. From these figures, equivalent values may be established for the other properties such as moisture content or MCV and a decision made as to the most appropriate method of defining acceptability for that material and construction compliance testing. Similarly lower MCVs of between 7 and 9 and upper MCVs of between 13 and 15 could be the starting limits from which the equivalent moisture content, CBR and shear strength figures are deduced. The Designer should exercise his judgement and experience in deciding which properties he uses.

4.15 If Class 2A material is composed of chalk then the SMC should not exceed 26%. This is more fully discussed in Paragraphs 5.8 to 5.20.

4.16 Class 2E, (PFA cohesive material) is discussed in Paragraphs 5.58 to 5.66, but typical values for the bulk density limits could be in the 1.3-1.65Mg/m^3 range.

4.17 The Specification considers separate classes of acceptable material; however the mixing of acceptable granular and acceptable cohesive materials can lead to a combined material which is unacceptable for the purpose intended. It is recommended, therefore, that variable materials, such as mixed sands and clays in glacial tills, should be constructed in separate layers across the full width of the embankment ensuring each layer contains acceptable material. The upper surface of impermeable layers should be cambered to shed water transversely. If it is unavoidable that the top layers of an embankment are constructed of material

TABLE 4/2

Relationship testing - sample test package

TEST	COMMENT
MCV/Moisture Content Calibration (5 points)	As Clause 632 SHW. The changes in moisture content should be carried out by wetting and drying from the natural state
Dry Density/Moisture Content	BS 1377: Part 4, 2.5kg rammer method; 4.5kg rammer method may also effect of greater compactive effort and vibrating hammer method for appropriate granular materials
CBR/Moisture Content	BS 1377: Part 4 (Determination of CBR) on compaction sample from above. Normally unsoaked but may be soaked under appropriate surcharge for special studies. Normal surcharge is usually $10kN/m^2$. CBR test on compaction sample from dry density test if CBR mould is used
Shear Strength/Moisture Content	Hand vane and/or penetrometer tests may be adequate on the MCV and CBR samples. Remoulded triaxial samples may also be used at each moisture content for undrained testing. Vane and triaxial tests may not provide collaborative data and site specific correlations between these tests should be checked
Atterberg Limits Natural Moisture Content/Particle Size Distribution	For each test package. All tests to BS 1377

susceptible to erosion then protection from water ingress should be provided.

4.18 Class 3 - General Chalk Fill - The designation of chalk fill as Class 3 material is discussed in Paragraphs 5.8 to 5.20. However the limiting values for SMC would have a lower limit of 26% an upper limit of approximately 38%. The predicted moisture content of chalk fill is 0.85 x SMC in summer conditions and so an upper limit for moisture content would be approximately 33%. Chalk however is a notoriously variable material and these figures should be treated with caution.

4.19 Class 4 - Landscape Fill - The fill for landscape areas can usually be drawn from a far greater spectrum of materials than other acceptable fills. It is not normally intended that the material should be able to support foundations, safety or noise fencing, vehicle parking etc, and the limiting values of acceptability for this class should reflect this. The basic requirement is that the material should be able to be transported and placed, be stable in its design profile, and be able to receive sufficient compaction to avoid subsequent excessive internal settlement.

4.20 It is recommended that for cohesive material the MCV test is used, with values of 6 or more, whereas for granular material the maximum moisture content could be in the range of 1.4 to 1.6 times the optimum value to give 90% of maximum dry density to BS 1377: Part 4 (2.5kg rammer method). The Designer can widen or narrow these ranges of values depending on availability of better material or for any particular material he may wish to use solely as landscape fill. The grading requirements would be similarly variable.

4.21 Class 6 - Selected Granular Fill - Class 6D, H, I, J, N and P all contain moisture content and MCV acceptability properties. As discussed for Class 1 granular materials, moisture content limits are more satisfactory than MCV, and should be set at the range of moisture contents which achieve 95% of standard maximum density to BS 1377: Part 4 (2.5kg rammer method) for Classes 6D, H, I and J, and to BS 1377: Part 4 (vibrating hammer method) for Classes N and P.

4.22 Classes 6I, J, N and P all refer to structural backfills and the properties listed in Table 6/1 SHW will have been tested and defined as stated. These values will have been used in the design of the relevant structures, and the Designer should investigate what percentage variation in these properties would cause an unacceptable change in the Factors of Safety of each structure. It is these limits which should be included in Appendix 6/1 SHW for properties such as Φ' and c', the coefficient of friction and the adhesion for fill/elements. Further information is given in Technical Memorandum (Bridges) BE 3/78 (Amended 1987) Reinforced Earth Retaining Walls and Bridge Abutments for Embankments, and BD 30/87 Backfilled Retaining Walls and Bridge Abutments.

4.23 Where permeability figures for Classes 6N an P are required, the actual permeability of the material should be measured using a constant head or constant hydraulic gradient permeability test method. The precise test depends upon the maximum particle size of the material, and full details of the procedures and equipment are given in BS 1377: Part 5 and Part 6. For testing in the triaxial cell, vertical permeability should be measured. If imported material is to be used, a minimum value for the coefficient of permeability of 5 x 10-5m/sec is recommended.

4.24 Class 6F2. The 10% Fines minimum value for materials found on-site should be set 10% below that value measured from samples of acceptable material taken during the ground investigation and rounded up although a minimum of 50kN is recommended. Where imported fill is to be used a minimum value of 50kN is recommended.

4.25 Class 6K, 6M and 6Q. It is recommended that the moisture content be specified in preference to the MCV. However if it is necessary to quote values for MCV then they should be the equivalent figures for the moisture content values, ie the MCVs at optimum mc - 2%, and at optimum mc + 1%.

4.26 Class 7 - Selected Cohesive Fill - Class 7A. The compaction requirement should be established from the ground investigation samples using BS 1377: Part 4 (2.5kg rammer method). The limiting values of c, Φ and c', Φ' should be set by the Designer to correspond to those values which would significantly affect the design of the structure, and the recommended limiting MCVs or moisture content values set to correspond to these at the limiting levels of the specified compaction. If the material is chalk, the SMC values are recommended to be 20% and 26%, which includes part of chalk Classes A and B of TRRL LR 806.

4.27 Class 7B. PFA is more fully discussed in Paragraphs 5.58 to 5.66, but limiting values of bulk density should be in range 1.3-1.65Mg/m^3. As stated for Class 7A, the limiting values of c, Φ and c', Φ' and coefficient of friction and adhesion (fill/element) should be established by the Designer. A reasonable permeability figure would be between 1x10-8m/sec and 1x10-6m/sec, although figures for the actual material should be obtained from the supplier.

4.28 Classes 7C and D. The values of c', Φ' and the coefficient of friction and adhesion (fill/element) should be established by the Designer from the structural design. The upper limits of moisture content for Class 7C can be based on these values of c' and Φ', and the lower limit on the equivalent MCV of 13 to 14. The limiting values for moisture content or MCV for Class 7D should be those which equate to the 95% of maximum dry density to BS 1377: Part 4 (2.5kg rammer method).

4.29 Class 7E. This material should be subjected to tests to determine what range of moisture contents, and therefore MCV, in its untreated state would give 95% of maximum standard dry density to BS 1377: Part 4 (2.5kg rammer method) in its treated condition. This will obviously be affected by the percentage of added lime, and it may be advisable to carry out a series of tests to allow for different percentages of lime content.

4.30 Class 8 - Miscellaneous Fill - Since this material is composed of either Class 1, Class 2 or Class 3 the moisture content or MCV limiting values should be established in the same way as each respective material class. It is noted that the MCV test is not generally recommended for use with granular material although it may be used with a few specific granular materials, see Paragraph 4.13. Compaction requirements have been left unspecified in Table 6/1 SHW, but should match the requirements for the relevant material class, although circumstances on site may dictate the use of alternative methods and plant.

4.31 Class 9 - Stabilised Materials - In general, for Class 9A the limiting values of moisture content could be based on achieving 95% of maximum dry density using 2.5kg rammer method of BS 1924; for Class 9C the limiting values of moisture content are to be based on achieving the end product specification; for Classes 9B and 9D the limiting values are based on achieving 95% of maximum dry density to BS 1377: Part 4 (2.5kg rammer method). The minimum bearing ratio is recommended to be 15% to be achieved at the end of appropriate curing periods for Classes 9A, 9B, 9C, and 9D. In any event the specified minimum bearing ratio must be achieved before subsequent construction layers are placed and compacted. The maximum MCV requirement for Class 9D material should be set at the minimum moisture content which would normally allow 95% of maximum dry density to BS 1377: Part 4 (2.5kg rammer method); the minimum MCV should be based on achieving a bearing ratio of 15%, taking account of lime content and curing period.

4.32 When preparing Appendix 6/1 for the scheme, the Designer should use the same format as Table 6/1 SHW, but only list the classes of material that he intends to use, and for these classes, only list those properties required for acceptability testing including those for which limits are already given in Table 6/1 SHW. Therefore Appendix 6/1 will give a complete list of acceptability criteria and limits for each material to be used in that scheme. If different or alternative materials are later specified the full details of properties should also be given in this form. Those criteria and material classes not required should not be included. A sample Appendix 6/1 is included in Annex A to this Advice Note.

Earthworks Balance

4.33 On an ideal scheme the quantity of fill required should match the quantity of acceptable excavated material, and whenever possible schemes should be so arranged that there is a near balance between cut and fill. However in arriving at the final earthworks figures and deciding upon the classes of fill for each embankment as shown on the earthworks' drawings, the following factors should be considered and allowed for where appropriate.

4.34 The distribution of the different classes of material in the cuttings; the availability of those materials for fill in the embankments and the sequence in which they will become available and be required.

4.35 The various restrictions which could apply to the excavation, haul and deposition of fill material, such as the presence across the site of an obstruction (eg railway, river, wet ground, main road or motorway); the ability of the insitu material to withstand the passage of the earthworks plant hauling the fill without the requirement of an expensive haul road; possible restrictions in land availability which could require steeper side slopes to some embankments and cuttings and which in turn could necessitate a particular type of material being specified for that fill material.

4.36 Where imported materials are required, locally available materials should be investigated in the first instance. The highway maintaining authority must be consulted regarding possible access roads to the site, and their distribution along the site could be critical in some cases. Department of Transport Circular 3/87 contains information regarding Planning Authorities and borrow pits etc.

4.37 Landscape areas within the construction site can sometimes be used as a convenient reservoir for acceptable material. They can be increased or decreased as construction proceeds by providing on-site locations for placing excess fill, or by reducing the extent and nature of the landscaping if the amount of acceptable fill material is less than anticipated. Therefore the contouring of a landscape area should not be too rigidly specified to allow the site engineers some latitude in the amounts of material put into it, but of course the basic function of the area must always be of primary importance which is usually to protect or preserve an existing feature. Off-site landscape areas are normally shown in contract documents as part of the site to ensure all provisions apply to them. They usually require planning permission and therefore should be considered permanent features during construction and should not be used to temporarily stockpile material.

4.38 When considering all these factors, the temptation to anticipate how the Contractor will programme the earthworks and select fill material must be strictly avoided. What may seem to be an obvious sequence of work to the Designer when preparing the earthwork documents may not be so attractive to a Contractor, who could well be subject to financial and commercial restraints and pressures which the Designer could not possibly foresee. The Designer should therefore concentrate on the finished earthworks and let the Contractor decide how to achieve it within his overall programme.

Compaction

4.39 Soil compaction is the process whereby soil particles are constrained to pack more closely together through a reduction in air voids leading to reduced internal settlement, higher strength and stability of the compacted material.

4.40 Two basic types of compaction specification can be applied.

4.41 Method. The method of compaction is specified in terms of plant, method of operation, thickness of layer and number of passes. This type of compaction specification is detailed in Table 6/4 of the SHW and is the type specified for most of the material classes in Table 6/1 of the SHW. The compactive effort specified in Table 6/4 for Methods 1 and 2 is designed to produce a maximum air-voids content of 10% assuming a conservative moisture content which is dry of the average field moisture content for the relevant material.

4.42 Method 3 should give approx 95% of BS 1377: Part 4 (2.5kg rammer method), maximum dry density assuming a conservative moisture content; Method 4 should produce a maximum 10% air voids at high moisture content (chalk); Method 5 is based on continental experience which gives satisfactory performance when this compactive effort is used on coarse granular material; Method 6 should produce a maximum 5% air voids at a lower limit of moisture content for sub-base compaction, and Method 7 should produce a maximum 5% air voids at a MCV of 12. However this does not preclude provision being made for some insitu density testing to ensure that the compactive effort is sufficient. Where a material lies across a boundary between two of the different methods of compaction described in Table 6/4, then the method which requires the higher compactive effort should be specified.

4.43 End Product. The state of compaction to be achieved is specified, leaving the choice of method by which it is achieved to the Contractor. Even so, it may be advisable in some instances in Appendix 6/3 of SHW to restrict the thickness of each compacted layer so that effective control can be maintained on site. This control will necessitate the determination of the insitu bulk density and moisture content for the complete depth of the compacted layer or, failing this, for the lower 150mm of the compacted layer, together with any other material property defined in Table 6/1 of SHW.

4.44 Vibrating rollers are now extensively used and in urban areas excessive vibration may cause damage to adjacent buildings. In contracts where this is thought likely TRRL RR 53 should be referred to for advice.

Instrumentation

4.45 The use of instrumentation as a means of gathering geotechnical information for highway-related earthworks may be divided into two main areas of operation.

i. Monitoring of construction. This is the most frequent application for instrumentation and is used where uncontrolled filling, excavation or other operation would invalidate design assumptions or reduce factors of safety to potentially dangerous levels.

ii. Performance evaluation. The object here is to provide data on the performance of the earthworks or structure. This is used to assess compliance with design assumptions and to predict long term behaviour.

4.46 There are a great number of applications for instrumentation and many different types of instrument but the most common measurements within the highway context are included in the following.

i. Vertical and horizontal movement. Measurement of movement is commonly carried out by the use of settlement pins or gauges, profile gauges, extensometers, strain meters, inclinometers, photogrammetric methods etc.

ii. Measurement of pore water pressures, by various types of piezometer such as open standpipe, Casagrande, electrical, pneumatic etc.

iii. Earth pressures by various cell types.

iv. Load measurement, by load cells, strain-gauged load cells, vibrating wire load cells etc.

4.47 Instrumentation can be expensive, so when considering its use on a highway scheme the geotechnical problems must be properly defined to establish the extent of the required data. The type and quantity of instrumentation proposed can then be justified and the frequency of readings properly set. The physical location of the instruments is obviously vital, and areas of uncertainty must first be established in the design process. A decision can then be made as to which of these areas is critical and what information is required. It should be remembered that simple solutions are often the best, for example a row of pegs along the toe of an embankment can provide a reliable indication of impending slope failure at low cost.

4.48 It is essential that provision is made for proper calibration of the instrumentation before installation and for checks to be made on read-out units, probes etc during the course of construction. Adequate reference points should be located outside the area of influence of the construction. Sufficient time must be allowed for datum readings to be taken prior to commencement of construction.

4.49 Damage to instrumentation is almost inevitable and back-up instrumentation may be required in areas where it may be vital to keep a constant flow of information. However in areas where instrumentation is vital to the control of construction, an Additional Specification Clause should be included in the contract stating that, if the instrumentation is damaged and the data is affected or is discontinued, then no further work in that area shall be allowed until the instrumentation is replaced or repaired, and is again operating satisfactorily. When locating piezometer read-out cabinets, inclinometer ducts etc, potential vandalism must be considered, and appropriate measures taken to either physically protect instrument housings or locate them in an inconspicuous position. Whilst access to the instruments may be simple during construction, access in the long-term, after a road or motorway is open, may be much more difficult, and thought should be given to this problem.

4.50 At the time of recording, the instrumentation readings must be related to values which the Designer has specified will correspond to two levels of awareness; a trigger value to alert and allow corrective measures to be taken, and a critical value beyond which the construction is at risk. It is essential that the Engineer designates one or more individuals to co-ordinate the reading and analysis of the results.

Rock Assessment and Fill

4.51 See also Chapter 3 on measurement of hard materials. Rock assessment can be based on the difficulty of excavation and handling the materials in cuttings. Some soft rocks may be extremely difficult to excavate when in massive forms in cuttings and hard rocks in thin bands interlayered with soft material may be excavated easily. Whilst the GI will provide some pointers the best assessment will come from previous experience of a comparable material in a similar geotechnical and design situation.

4.52 Rock fill is a term which will not be found in SHW or MMHW, but, for all intents and purposes, SHW Classes 1C, 6B and 6C are rock fill. Such materials need to be hard and durable. The gradings differ for each of these classes but soundness is determined by the soak Ten-Per-Cent Fines Test (TPF) where a fines value of 50kN needs to be achieved. Other classes of material may be produced by crushing rock and it should be noted that a number of other classes also require a TPF value to be met.

5. INFORMATION ON SOME SPECIFIC MATERIALS

General

5.1 Some general information is given below for a wide range of soils and rocks, to help Designers with their classification and assessment of materials. In some cases, for example chalk and PFA materials, the use and treatment of a particular material is described in some detail. More specific information on assessing material types, their classification and acceptability is given in Chapter 4.

Argillaceous Rock

5.2 Argillaceous rocks have been defined by the SHW principally to prohibit their use in most of the SHW Class 6 `Selected Granular Materials' and is not used in the strict geological sense. The definition includes shales, mudstones, slates and unburnt colliery shale (Minestone). These materials have a tendency to break down during weathering and are unlikely to have the long-term durability required for Class 6 materials: they also exhibit properties which can make them chemically aggressive to structures. Shales may also exhibit the same problems as those described for unburnt colliery shale in Paragraphs 5.27 to 5.30 and Paragraph 12.14.

Soft Rocks

5.3 The term `soft' applied to rocks is not particularly meaningful except in so far that it tends to describe those materials which are likely to break down during excavation, placing, compaction or most importantly in service. Rocks under this heading are likely to be sedimentary deposits. But some metamorphic rocks, such as schists, and some igneous rocks are also very susceptible to weathering. This does not preclude soft rocks from being used. Nearly all materials will fit into the SHW classification system, and soft rocks, especially those of a basically granular nature, may be used as one of the selected granular fills; the class of material and the design parameters depending on the degree of degradation of the material.

Sherwood Sandstone (includes Bunter Sandstone)

5.4 Sherwood Sandstone consists mainly of softish rock composed of cemented particles of sand which break down during compaction and beneath construction traffic. It is uniformly graded and when freshly excavated has been used successfully as a capping layer on embankments. When trafficked the percentage passing the 75 micron sieve can increase from approximately 10% to 20%, making it frost susceptible. It is not recommended for use as capping in cutting unless good drainage can be assured. Although very weather susceptible, degrading to a lower permeability and CBR, it has successfully been used as a fill to structures. Close control of both the upper and lower moisture content limits is advisable to ensure good compaction.

Mercian Mudstone (includes Keuper Marl)

5.5 Mercian Mudstone is largely comprised of argillaceous rocks and outcrops widely in the midlands and West of England. Their bearing capacity can be difficult to assess due to a variable depth of weathering and interbedding with sandstones. They are often highly fissured and the resultant percolation of water leads to softening around the fissures. Near the ground surface the whole mass may be softened by weathering. The classification of `Keuper Marl' into four weathering zones, together with typical index properties, effective stress parameters and other properties can be found in CIRIA Report 47. Details of allowable bearing capacities can also be found in BS 8004:1986; Foundations; Section 2. The 1976 Rankine Lecture `The Triassic Rocks' by A C Meigh, is also a valuable source of information on the geology of Mercian Mudstone.

5.6 Mercian Mudstone generally contains sulphates, and tests should be carried out as part of the ground investigation to discover their concentration. They may be sufficiently high to require protection measures to adjacent steel or concrete.

5.7 In places, rocksalt within the Keuper Marl has been mined. CIRIA/PSA publication `Construction over abandoned mine workings' discusses methods of dealing with problems associated with mined areas. Man-made or natural disturbances of the groundwater regime can dissolve the salt beds resulting in subsidence (Howell and Jenkins, 1976).

Chalk

Description

5.8 Chalk is usually a soft, white, porous, jointed limestone of greatly varying strength. However in the lower layers of the Lower Chalk there is a transition zone from the underlying clays which consists of a greyish or buff coloured chalk marl. The marl can be found either in separate layers, or homogeneously distributed in varying proportions with the chalk. In some cases the marl can form up to 40% of such a mixture. The upper layers of Lower Chalk, together with the Middle and Upper Chalk, are much purer. Some relatively thin layers of very hard chalk are sometimes found at intervals throughout the Chalk and in the Middle and Upper Chalk, bands of flint nodules are also found.

Moisture Content

5.9 Chalks have a natural moisture content varying from 8% to 36%. During earthworks operations the excavation and compaction processes break down the natural structure of the chalk releasing some of this water and generating fine material. If the amount of fines and released water, added to any free water found in the joints, is high enough, then the result is a temporarily unstable fill material which may remain unstable for a period varying from days to weeks, depending on the rate of drying. Therefore any classification of chalk for acceptability as a fill material must be based on a prediction of moisture content and degree of crushing. It has been found that stable conditions are likely at moisture contents below 23%, and these stable conditions can be maintained above this moisture content by reducing the degree of crushing, ie by reducing the compaction, and selection of appropriate earthworks plant.

Ground Investigation

5.10 On sites where chalk is present, the Ground Investigation should include trial pits or large diameter shafts to obtain suitable samples of the chalk, both for classification purposes and to inspect the structure of the chalk. Closed circuit television cameras may also be lowered down the shafts for an insitu inspection of the degree of fissuring and flint content. Where there are deposits of other materials overlaying the chalk, the interface between them can be sharply undulating and solution features can allow extensive infilling of the overlaying deposits into the chalk. Consequently, additional boreholes may be required at critical locations such as structural foundations.

Chalk Classification

5.11 The classification test for chalk requires the measurement of the saturated moisture content (SMC) as described in Clause 634 SHW and the Chalk Crushing Value (CCV) as described in BS 1377: Part 4. Using the results from these tests, the chalk samples can be classified from A to D by employing the Chalk Classification Chart in Figure 5/1 which is based on the chart from TRRL Laboratory Report LR 806. The percentages of each class of chalk likely to be present within each cut area and also for the whole scheme can then be estimated. The earthworks classification of the material in Appendix 6/1 of SHW will depend on these percentages. It should be noted that the chalk classification must not be quoted in the Contract, being for the Designer's use only. The recommended earthworks classification procedure for chalk, and other materials which may be mixed with it, is given in a flow diagram in Figure 5/2. Whilst this procedure should be applied individually to each cutting containing chalk, it is possible that a small cutting may be included with a larger adjacent cutting for the purposes of earthworks classification. However, the chalk must be looked at as a whole and the chalk classification described above should be viewed as an aid to assessing the chalk mass rather than a mechanical exercise which is complete in itself. There are areas of greatly disturbed chalk, often containing solution features, where the quality does not always improve with depth, and in these areas a simple chalk classification may be inappropriate because of the variability. In such cases, unless there is an overwhelming requirement for the chalk to be used as a fill, it should be classified as acceptable for landscaping area only (Class 4) or as unacceptable because of the difficulty of specifying suitable overall compaction methods.

Chalk Compaction

5.12 The Designer should bear in mind when considering the use of cut material that Chalk Classes B and C may occasionally produce unstable conditions

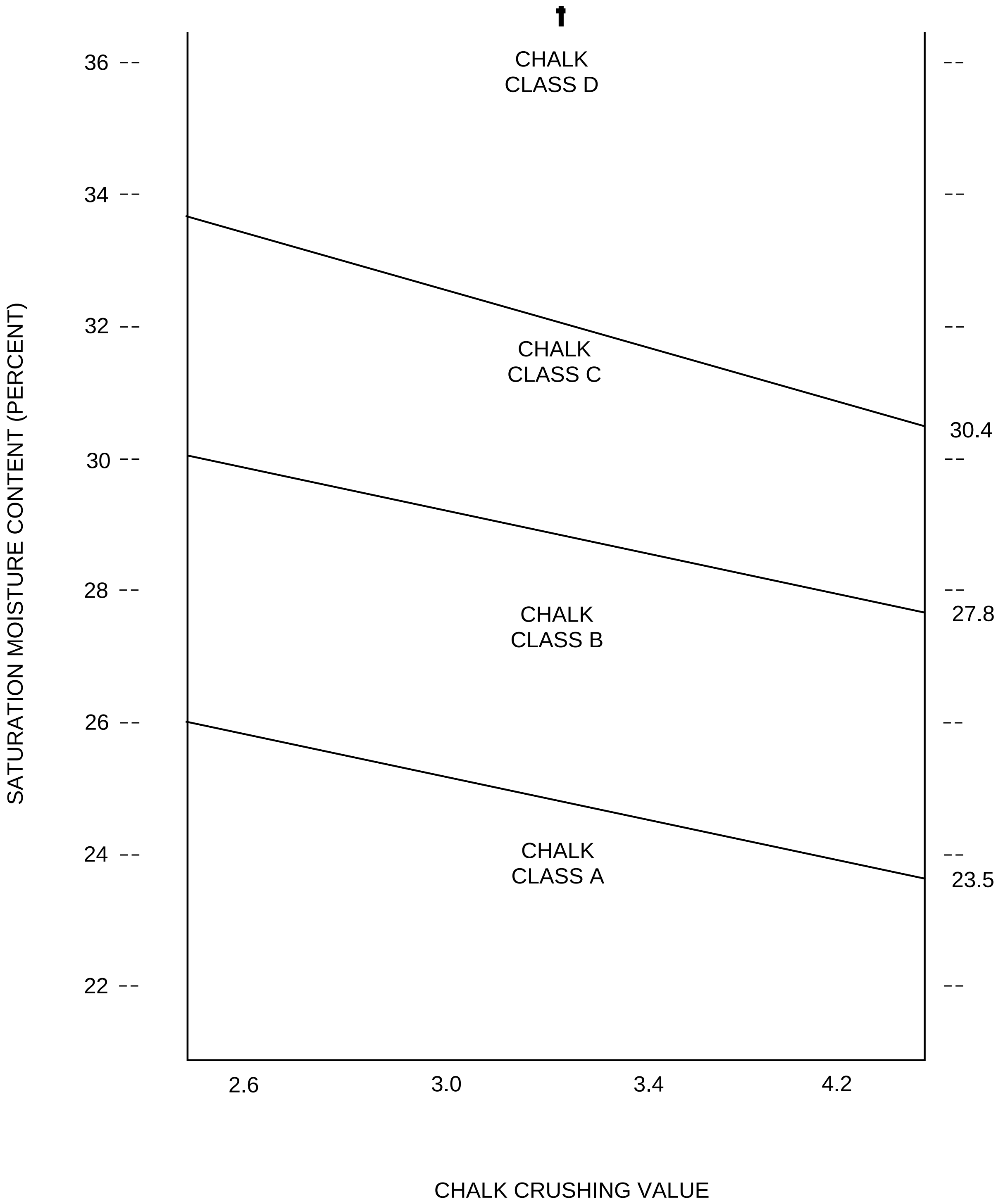


Figure 5/1 Chalk Classification Chart

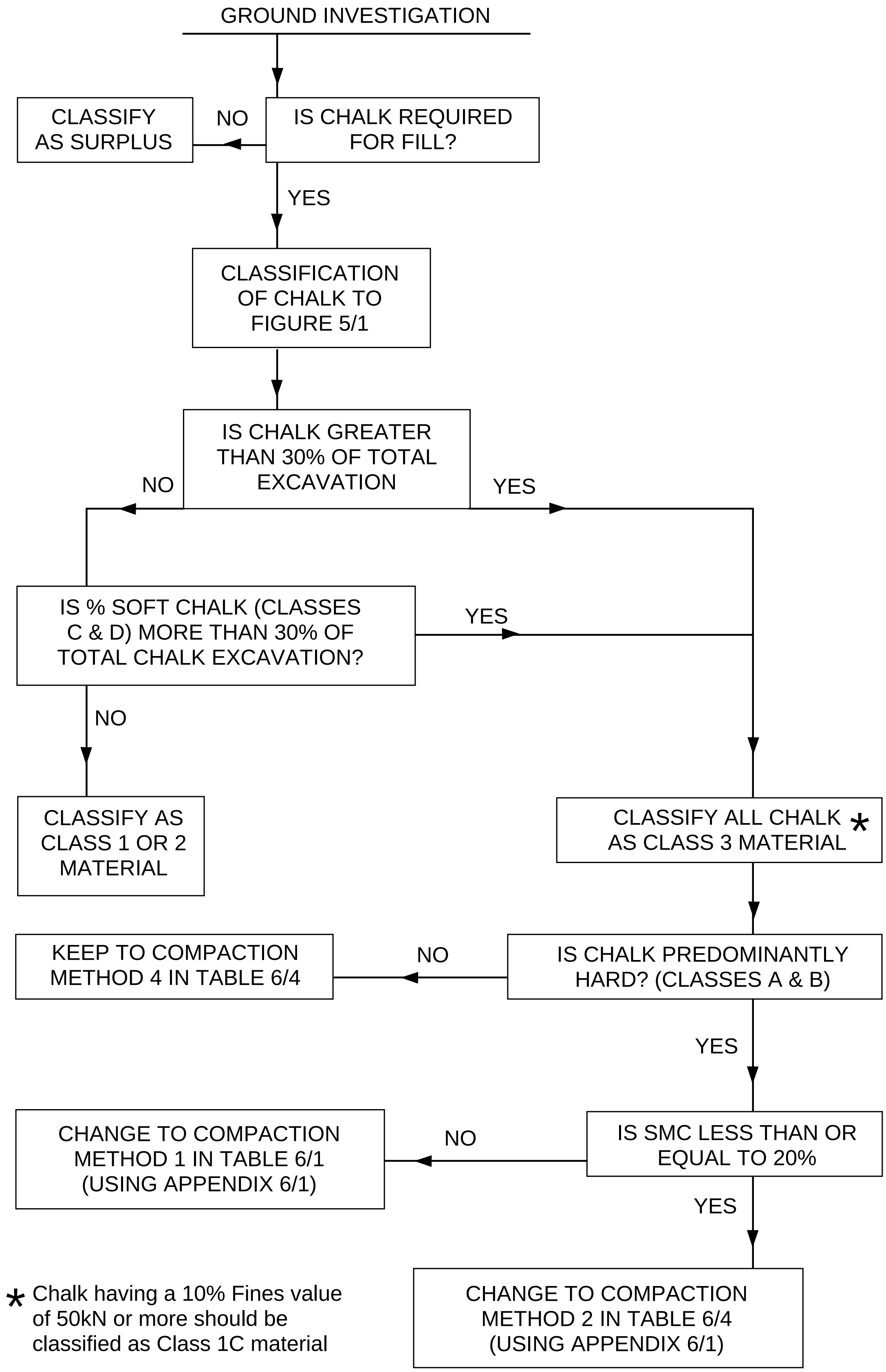


Figure 5/2 Flow diagram for use in classifying chalk even when the specified type of compaction plant is used. It may be better to leave the compaction method unchanged and remove occasional unstable material when it is found or allow for delays to recover stability rather than reduce the compactive effort and risk long-term settlement in the under-compacted material. The flow diagram also indicates which compaction method from Specification Table 6/4 should apply. Method 4 compaction should be used for chalk designated Class 3 material which is wholly or mostly composed of Chalk Classes other than A or B. If the Class 3 Chalk is predominantly A or B the compaction group should be amended to Method 1 if the SMC is greater than 20%, or Method 2 if the SMC is less than or equal to 20%. The Resident Engineer therefore may reduce the compaction requirement if it is found to consistently cause instability on site.

Information for Appendix 6/4

5.13 If Class 3 material is designated in Appendix 6/1; further information should be given in Appendix 6/4, on the following.

5.14 The time period, when no Class 3 earthworks shall take place (Clause 605.1(i)) ie Winter. This is intended to be the period when the rainfall exceeds the evaporation rate, over monthly or weekly periods. A method does exist for calculating the evaporation rate but the formula is very complex and is not really suitable. A practical approach to the prediction of the most likely period is to use the rainfall and the evaporation figures from the Meteorological Office long-term statistical records. Analyses of records for the last 25 years are available for any area in the United Kingdom and may be obtained from the Meteorological Office Advisory Service at Bracknell. If the appropriate rainfall or evaporation figures are not available, then a winter period of 1st November to 31 March should be specified.

5.15 Minimum height of excavation (Clause 605.1 (iii)). If possible this height should be extended to 5m and worked as a quarry face. However it would be preferable to work a 6m high cutting in two layers of 3m each rather than a 5m layer followed by a 1m layer, and so the depth of chalk in the cuttings must be considered before amending the minimum excavation height. If the chalk is surplus and will be taken off-site, then the plant restriction and minimum height may be ignored.

5.16 If the majority of chalk is estimated to be Class A or B the restriction of haulage vehicles to a 15m3 maximum capacity (Clause 605.1 (v)) may be relaxed. If there is any doubt it would be preferable to leave that decision to the Engineer on site after close observation of the behaviour of the material.

5.17 Layering of Class 3 Chalk with other material is not recommended unless absolutely unavoidable (Clause 605.1 (vii)). However if chalk is placed on a less permeable material, that material must be cambered to shed water transversely. Failure to do so could cause free water released during the placing and compacting processes to pond inside the chalk with possible long term instability setting in. Therefore if composite embankments are envisaged the surface of any relatively impermeable material must be cambered, and a suitable requirement to this effect included in Appendix 6/4. For the same reason Class 3 material must extend the full width of the embankment and must not be contained within bunds of impermeable material.

5.18 Period of waiting (Clause 605.1 (viii) and (ix)). If required, a pre-contract trial may be carried out on the chalk to investigate its readiness to become unstable and then stable again. If the time for the hardening process is delayed beyond 4 weeks the period in Appendix 6/4 should be lengthened accordingly: similarly if the chalk hardens quickly the period may be reduced. Chalk which has been contaminated with sandy material sometimes requires a much longer time to harden.

5.19 Rolling to seal material at the end of each working day (Clause 605.1 (x)). If the chalk is estimated to be very hard or very soft, this rolling may be omitted. Also if the chalk has become unstable, rolling may aggravate the situation by preventing the chalk from hardening, and should also be omitted by the Resident Engineer.

Included and Infilling Materials

5.20 Materials which cannot be separated from the chalk and are therefore included with it, such as flint bands or any other material which may affect the Contractor's method of working, should be mentioned in the Contract. Where infilling occurs of solution features by overlying material, the infilling material should be excavated separately and kept separate from the chalk, allowance being made for this in the Contract; the excavated material is usually classified as unacceptable. Treatment of infilled solution features using granular infill and concrete plug or similar methods should be included in a detailed specification especially where extensive treatments have to be carried out.

Coal Measures

General

5.21 Coal is the most important lithology within the Coal Measures, although the bulk of the beds are sandstones, siltstones and mudstones with seat earths, and sometimes limestones and other rock types. During mining operations quantities of these rocks, unavoidably extracted with the coal or in driving the access tunnels, are brought to the surface with the coal. The separated non-coal material is known as Minestone (Paragraphs 5.27 to 5.30) and is usually tipped onto spoil heaps. Many of the problems associated with the shales and mudstones of the Coal Measures also apply to Minestone.

Ground Investigation

5.22 The Coal Industry Nationalisation Act 1946 vested, with a few exceptions, the freehold interest of all coal and mines of coal including shafts and outlets in the National Coal Board later renamed British Coal. It is therefore necessary that British Coal be consulted before any ground investigation is carried out on land beneath which there are coal seams and again when the location, depth and thickness of the seams, are known and the road design is being considered. The Opencast Executive of British Coal should be consulted for information on worked, prospected and proposed sites. Further information on ground investigation procedures for land, where coal seams, opencast sites and mine shafts are known or suspected, is given in Departmental Advice Note HA 34/87.

Disused Mine Workings

5.23 If disused mine shafts and adits or other workings have been located and identified there is the chance that they may well cause surface instability in undermined areas. Methods of dealing with the various problems associated with mine workings are discussed in the CIRIA/PSA publication `Construction over abandoned mine workings', and also in `The Treatment of Disused Mine Shafts and Adits' published by British Coal, Mining Department.

Slope Stability

5.24 Care should be taken when designing cut slopes in Coal Measures rocks. The mudstones and siltstones which are found in the Measures may contain weathered layers which have softened and are potential slip planes. Indeed, there may be existing slip planes already within these soft clay layers and cutting slope failures have occurred because of them. Ground investigations in these materials should be very carefully conducted so that these potential shear zones are identified.

Soil Aggressivity

5.25 The dark colours of some mudstones may indicate a high proportion of disseminated pyrite which may oxidise in time and form sulphuric acid (H_2SO_4). Hawkins and Pinches reported that samples of mudstone retested after 17 months showed a fivefold increase in SO_3 content. Therefore, when these mudstones are present, care should be taken that the effects of such an increase in SO_3 content and consequent increase in soil aggressivity are allowed for.

5.26 One additional problem is that when H_2SO_4 is leached from these materials and comes into contact with carbonate rocks, gypsum is produced. Within a drainage system this can cause blockages and beneath a road pavement it can cause heave. The reaction between the acid and carbonate rocks also produces carbon dioxide (CO_2) which can cause asphyxiation in confined spaces.

Minestone

General

5.27 Minestone is the generic term for unburnt colliery shale which is a mixture of rock debris from the Coal Measures. The debris mainly contains shales, mudstones and siltstones and is brought to the surface during the mining of coal, to be placed in colliery spoil tips. Large amounts of this waste material have accrued over the years and are potential sources of imported fill material. Minestone can, however, have considerable ranges of hardness and durability depending on the strata worked and the method of mining.

Colliery Spoil Tips

5.28 The material in older tips often have a high carbon content whereas newer tips tend to contain more sandstone waste and coarser discards. The older tips are nearly always loose and open textured allowing air to penetrate the tip. If the coal in the tip is ignited, for any reason, a burning section can consequently be sustained and extended. These older burnt tips can contain anhydride salts which expand on exposure to moisture. Newer tips are compacted, significantly reducing the opportunity for sustained fires.

Minestone in Fill

5.29 Minestone, if compacted to the SHW, will not spontaneously combust as oxygen will be excluded. Many examples exist of hot burnt minestone being satisfactorily compacted to form stable embankments. Minestone can therefore make very good all-weather general fill provided it is of sufficient quality. Some shales may be prone to softening and swelling if exposed to moisture and weathering when stockpiled. It is the Department's policy that the Designer should identify possible sources of this material for imported fill and ensure that Tenderers are aware of them. However, Tenderers must be left to choose their sources on a commercial basis, and it is unlikely that a haul distance to site of more than 20km will be economic. The designation of sources of fill does not infer acceptability and testing must still be conducted. Further information on available sources may be obtained from British Coal Minestone Services, Philadelphia, Houghton le Spring, Tyne and Wear, DH4 4TG.

5.30 In the unburnt state, minestone is not one of the permitted materials for the selected fills defined in Classes 6 and 7 of Table 6/1 SHW: the use of Minestone as a fill to bridge abutments and retaining walls is also prohibited in BD 30/87. Further information is given in Tech Memo H4/74, `The use of Colliery Shale as Filling Material in Embankments'. See also Paragraph 12.14.

Sands and Gravels

5.31 Sands and gravels originate from the weathering and disintegration of a wide variety of rocks. Commonly the geological processes of transport and deposition are by water but many deposits are formed by wind action (producing perfectly rounded sand grains) and some by ice action and others under the action of gravity. The majority of sand-size grains are of quartz which is the mineral most resistant to weathering whereas gravel-size fragments and stones can consist of a wide variety of rock types. However, gravels in the Midlands and South of England are usually flint, quartzite or limestone. In the North of England and in Scotland, mixed gravels occur sometimes containing pieces of soft material, such as, clay lumps, shale or soft sandstone, which are a source of weakness.

5.32 Many deposits consist of a mixture of both sand and gravel size particles (as defined in BS 5930 but see also Paragraph 4.4) and some of the Class 1 and Class 6 granular materials in Table 6/1 of SHW can be described as `sand and gravel' whereas other are essentially `gravel' or `sand' only. Table 6/2 of the SHW gives the particle size limits of these classes. Grain shape can affect engineering properties eg angular sands have a higher shear strength than rounded sands because of enhanced granular interlock.

5.33 Acceptability of sands and gravels may be stated in terms of moisture content, although for some sands MCV is also practical. Assessing moisture contents may be difficult especially as the material becomes coarser and the percentage exceeding 20mm size becomes critical. Sands and gravels are free-draining and will become `acceptable' within a short period after excavation, however the presence of fines may inhibit drainage. If initially too wet, sands and gravels exhibit frictional behaviour when sheared even under rapid loading conditions. Settlement under load is almost immediate.

Heavy Clays

5.34 Heavy clay is a term used to denote soils of high plasticity; they are usually encountered in an over-consolidated state and comprise such geologies as London Clay, Oxford Clay, Gault Clay, etc. The excess negative pore pressures of these clays developed on excavation are only slightly reduced with time when placed within the embankment. On the surface of a cutting or embankment slope, with a steady supply of water from various sources, excess negative pore pressures dissipate locally with a subsequent loss of shear strength and reduction in the factor of safety. Failures have occurred as a result of this mechanism, over varying time scales following construction; these usually take the form of shallow slips with a typical depth of failure plane ranging from 1.0m to 1.5m as described in TRRL RR 199 (see Paragraphs 7.2 to 7.6 and Paragraphs 8.2 to 8.4). Instability usually occurs on slopes steeper than or equal to a slope angle of 1 in 3, although a few failures have been recorded at 1 in 4.

5.35 The permeability of clays is generally much lower than sands and gravels but is still significant in the long term and can lead to settlement as a result of consolidation.

Silts and Fine Sands

5.36 These soils are predominantly those identified as Class 2D of Table 6/1 SHW and require Method 3 compaction (Table 6/4, SHW). In their properties and engineering behaviour they tend to intermediate between typical `granular' and typical `cohesive' materials. The silt particles differ from the sand particles in size and are sometimes similar both physically and chemically. However, the surface tension of the inter-particle water confers a degree of cohesion on the soil.

5.37 Silts and fine sands are liable to liquefaction in circumstances where the effective stress, and, for these materials, the shear strength, is reduced to zero. This problem can be encountered during cutting excavations and during embankment construction where they are used as fill or where they form `soft ground' beneath the embankment. Liquefaction occurs:-

i. in class 2D earthworks. When the surface becomes saturated and then trafficked or subjected to other forms of loading. Where Class 2D forms the slope of an earthwork, slides, landslips or mud-flows can occur;

ii. under drains, leading to washing out of fines and the formation of cavities (`piping');

iii. under conditions of rapid increase in pore water pressure (including earthquakes);

iv. when water flows through the soil under a large hydraulic gradient near the surface of the earthworks (cuttings mainly).

5.38 Silts and fine sands are amongst the materials most susceptible to frost heave and must not be included within the top 450mm of the Works unless testing indicates otherwise. (See Clause 602.19 SHW). The permeability of these materials permits ready access of water and the pore spaces are too small to accommodate the growth of ice crystals without disruption of the structure (unlike the coarse sands and gravels).

Glacial Till

5.39 Glacial Till is a widespread drift deposit consisting of mixtures of materials ranging in particle size from clay to boulder and showing considerable variations in gradings. The amount of fines in a till will determine the till's behaviour, and a high silt content makes it very weather susceptible. If control for suitability to form embankments is by moisture content it is important to specify the grading fraction on which it is taken.

5.40 In cutting slopes, the stability of undisturbed fissured clay is conditioned by the orientation and frequency of the fissures rather than the strength of the intact clay. Glacial Till frequently contains lenses of water bearing sand which can cause drainage problems and soften the clay at the interface with the sand.

5.41 As shown in TRRL RR 199, the stability of slopes in this material improves further to the North or West of England and Wales, reflecting its variation in properties. The effect of fissures on the stability of excavated slopes in Glacial Tills is described by McGown (1985).

5.42 Other reference sources include the Symposium on the Engineering Behaviour of Glacial Materials, Birmingham and the Quaternary Engineering Geology Conference, Edinburgh.

Alluvium, and Estuarine and Marine Sediments

5.43 Alluvium is a material which has been transported and laid down in the bed or flood-plain of a stream or river. It can comprise most types of soil from gravel at one extreme to clay at the other. However, it is more typically fine sands, silts and clays often in alternating layers of a seasonal nature as in the case of varved clays (glacial lake deposits).

5.44 The silts and clays are `normally consolidated' rather than over-consolidated and are frequently soft and compressible. The water table is often close to the surface. Hence there is usually a `soft ground' problem when embankments are constructed across alluvial areas. Some of the measures for dealing with these problems are described in Chapter 9 of this Advice Note. Ground water lowering may also be necessary to facilitate cutting excavation and to improve otherwise unacceptable material.

5.45 Estuarine sediments occur in the estuaries of large rivers and are often characterized by the presence of silt and clay with a small amount of sand and gravel, and banks of shells. Similar `soft ground' problems can occur to those cited above.

5.46 Shallow water marine sediments are again variable depending on the geology, onshore and offshore, and the vicinity of other sources of material. Materials vary considerably from gravel to clay and sometimes occur with shells.

Organic Materials and Peat

5.47 These materials are included in Class U1 unacceptable material (Clause 601.2 (ii), (a) and (b) SHW) unless designated by the Engineer as landscape fill Class 4, in Appendix 6/1. When peat and related deposits occur at or below formation level or embankment foundation level, they can lead to embankment instability during and after construction and to long-term settlements causing damage to pavement structures.

5.48 Peat accumulates in wet, flat or hollow areas when the rate of addition of vegetable matter exceeds the rate of decay. Fen peats form on low ground whereas bog peats form on high ground. Bog peat is normally more acidic than fen peat. Organic soils range from peaty clay through clayey peat to peat, and start to behave like peat once the organic content (by weight) exceeds about 27%. Peat can be light brown and fibrous or a dark brown or black amorphous jelly-like material.

5.49 The principal engineering characteristics of peat are its exceptionally high water content and the importance of secondary consolidation. Secondary consolidation under load is prolonged (taking many years) and is dominant over primary consolidation which is complete after a relatively short time (weeks rather than years). Settlement on site may be difficult to predict because the coefficient of secondary compression can vary with time (Hobbs, 1986). Fibrous peats on site have a higher permeability than the small specimens tested in the laboratory. Large scale and prolonged site trials may be necessary to assess the parameters for secondary compression and hence predict the likely settlement over a period of years (eg 30 years). The shear strength (stability) of peat depends on time as well as effective stress because the void ratio continuously decreases under the maintained load. Stability and rate of loading are less critical for fibrous peat than they are for amorphous peat. A change in regional drainage can increase drainage leading to lowering of the surface of peat bogs without any loading.

5.50 Settlement of peaty ground beneath roads or embankments can be reduced either by:-

i. treatment of the ground; or

ii. reducing the load applied to the ground.

5.51 Of the treatment methods described in Chapter 9, dynamic compaction is unsuitable for peat. Vertical (sand) drains will probably not be effective unless the peat is a thin layer interbedded with other materials because such drains only accelerate primary consolidation which involves dissipation of excess pore-water pressure. Secondary compression consists of a process of void reduction involving only a small decrease in pore pressure.

5.52 When the base of the peat deposit is less than 5m below ground level, complete excavation and replacement may be the economic solution provided acceptable fill (eg Class 6A for deposition below water) is available. When the base of the deposit is more than 5m below ground level preloading (surcharge of the embankment) may provide a solution, but specialist advice should be sought. Preloading can improve the properties of the peat with the aim of accelerating and anticipating the settlement to be caused by the service load, over the required period of years.

5.53 Methods of reducing or distributing the load applied to peaty ground include:-

i. changing embankment geometry eg widening its base;

ii. providing a sort of `raft foundation' eg by means of geogrids at or near the base of the embankment or at formation level;

iii. staged construction of embankment;

iv. use of light-weight fill;

v. piled foundations taken down to firm ground.

5.54 Peat may occur in cutting and will require very flat slopes or containment. Drainage is an important design criterion here as flows and slides can occur; also open ditches are likely to collapse.

Waste Materials and Industrial By-Products

5.55 The better known materials, Minestone and PFA are dealt with in Paragraphs 5.27 to 5.30 and Paragraphs 5.58 to 5.66 respectively. Toxic and domestic wastes are discussed in Paragraphs 5.67 to 5.71. Waste materials should be used in road construction where this is environmentally desirable and economic.

5.56 Further information may be obtained from TRRL Laboratory Report 647, `The Use of Waste and Low-Grade Materials in Road Construction', and also from BRE Paper 19/74, `A Survey on the Locations, Disposal and Prospective Uses of the Major Industrial By-Products and Waste Materials'.

5.57 Crushed lean mix concrete is acceptable in those classes where crushed concrete is allowed, provided it meets the other requirements for that class of material.

Pulverised Fuel Ash (PFA)

5.58 PFA is an industrial by-product and for the purposes of SHW Table 6/1 is designated as Classes 2E and 7B material, or Classes 7G and 9C if cement stabilization is required (see Chapter 10). The sulphate content of PFA is likely to be such that SHW Clause 601 is relevant. The dimension restriction on depth below sub-formation and formation in SHW Clause 601 has been introduced for the following reasons:-

i. because of the grain shape and size the upper layers of PFA are difficult to compact;

ii. freshly placed PFA behaves in a manner similar to a pure silt and, if not protected, may liquify following wet conditions;

iii. capping and sub-base materials tend to be relatively permeable and a layer of general fill over PFA is considered desirable to provide some protection.

5.59 The dimension may be marked on the drawings but it is recommended that SHW Appendix 6/3 is more appropriate. A figure of 600mm is considered acceptable, and any reduction needs to be carefully considered.

5.60 Erosion can be a particular problem where PFA embankments are exposed to the weather.

5.61 PFA properties vary, not only from source to source, but also within a single source. These variations will affect the ability to achieve the required compaction which may be particularly critical where PFA is being placed adjacent to structures and where the density achieved may affect the design criteria. SHW Clause 601 enables the Engineer to keep a record of the sources, and places the onus on the Contractor to provide the requisite data including the natural moisture content, and the optimum moisture content and maximum dry density of each consignment. Where field densities are being taken the top 100mm of PFA should be removed before testing.

5.62 The Classes of PFA found in SHW Table 6/1 vary depending on whether the material is conditioned, or from lagoons or stockpiles.

5.63 Conditioned PFA is specified in SHW Class 7B for use as fill to structures and reinforced earths. It is obtained directly from the coal burning process and sufficient water added to bring it to a state suitable for compaction. The optimum moisture content is typically 25%, but may be as high as 35%.

5.64 SHW Class 2E includes lagoon PFA which is material which has been transported in slurry form and stored in lagoons and may be used as a general light weight fill.

5.65 SHW Class 2E also includes stockpiled PFA which can be either (i) or (ii) above which has been stockpiled. After a period of storage this material tends to approach its optimum moisture content. It may also be used as a general lightweight fill.

5.66 As shown in Table 6/1 SHW, for Class 2E material the acceptability criteria need only be sufficient to provide the required compaction; for general fill purposes it is sufficient to rely on the end product specification and bulk density limits need not be stated. If the weight of the embankment is critical to the design (see Chapter 9) then density limits may be stated along with the end product. These limits may typically be between 1.3Mg/m3 and 1.65Mg/m3. Class 7B material as above should rely on the end product specification. For conditioned PFA bulk density limits are also typically 1.3Mg/m3 to 1.65Mg/m3. For both classes of material it is advisable to check on sources and likely availability with the appropriate generating authority who will also provide typical test results for the various sources.

Domestic and Toxic Wastes

5.67 Toxic wastes are classified in Clause 601 SHW as unacceptable material Class U2 and usually consist of either industrial and chemical wastes, or domestic wastes. When toxic or other hazardous waste material is to be removed, the regulations covering excavation, transportation and deposition into a registered tip are closely governed. The local Environmental Health Officer (EHO) must be contacted and consulted regarding the treatment of any toxic waste material located within or affected by the scheme.

5.68 Similarly domestic waste or refuse can produce inflammable or toxic gas and leachate, and surface water drainage from the road which passes through domestic tips must be sealed systems to avoid contamination of the underlying aquifer. To avoid contaminated water reaching the road drainage from the tip, or to prevent gas migration into the road structure, the road, whether at ground level, in cutting or on shallow embankment, must be sealed off from the domestic waste material by a continuous impermeable membrane. On higher embankments, fill of suitable permeability and thickness can be placed to prevent upward migration of the leachates and gases, possibly removing the need for a membrane. The EHO must be consulted about roads through domestic waste tips and the appropriate specification clauses agreed regarding operations within the area, and precautions to be taken for toxic and noxious gases which may be present such as methane, hydrogen sulphide etc.

5.69 In some instances it may be possible to leave the domestic waste in place and treat it by dynamic compaction to produce a dense foundation for the road. The overseeing department should be consulted for details of other similar schemes which may have been carried out successfully.

5.70 Where domestic waste is exposed on a cutting slope it should be protected by an impermeable clay cover and/or membrane to ensure that leachates are directed away from carriageway drainage systems. In deep cuttings, as a result of sealing the slope, the head of water may be sufficient to cause failure of the road or the slope so subsurface drainage must be provided to intercept contaminated water for transport to a treatment works. The Local Water Authority will need to be consulted on the precautions taken. Gases contained within the slope by the sealing of the face or within chambers and ducts for services and drains, should be vented. In some cases where the road is away from leachates and the gas produced does not affect the vegetation on the slope, then no sealing may be required. Monitoring of the amounts and types of gas produced, before, during and after construction is advised.

5.71 References covering construction and safety on marginal, derelict and contaminated land, are included in Cairney (1987), Building on Marginal and Derelict Land, ICE (1987) and Fleming (1991).

Geotextiles and Related Products

General

5.72 Geotextiles, geogrids, geomeshes and other related products comprise a family which may be called `geosynthetics'. They are usually made from such synthetic polymers as polypropylene, polyester, polyethylene and polyamide which have a high degree of inertness to biological and chemical degradation, and a high mechanical strength.

Types of Geosynthetics

5.73 Woven geotextiles are composed of two perpendicular sets of parallel yarns interlaced systematically to form a single planar structure. The manner in which the two sets of yarns are interlaced determines the weave pattern and it is possible to produce an almost unlimited variety of fabric constructions. Woven geotextiles are used in a wide variety of applications.

5.74 Non-woven geotextiles are formed from fibres arranged in either an oriented or random pattern to form a planar structure. The fibres may be bonded either by chemical bonding using a cementing medium; by thermal bonding, where heat is used to partially melt the fibres so they adhere together; or by mechanical bonding using small barbed needles, set into a board, which are punched through the loose fibre web and then withdrawn leaving the fibres entangled. Non-woven geotextiles are also used in a wide variety of applications.

5.75 Composite geotextiles combine multiple layers of the woven and non-woven material, bonded together by a stitching or a needle punching process.

5.76 Geowebs are very coarse woven fabrics made from strips and are used for erosion control, bank protection and soil reinforcement.

5.77 Geonets consist of two sets of coarse parallel extruded strands intersecting at a constant angle. The width of strand varies from 1 to 5mm and the size of opening varies from a few millimetres to several centimetres. They are typically used in soil reinforcement and in fabricating gabions.

5.78 Geogrids can be made by a variety of methods one of which involves drawing a perforated plate of polymer in perpendicular directions. They may be uniaxial or biaxial in character and are normally made of polyethylene or polypropylene. They can be used in soil reinforcement, retaining walls, foundations and sub-bases.

5.79 Geomembranes are sheets of synthetic material which act as impermeable barriers.

Functions of Geosynthetics

5.80 Whilst there is an enormous number of applications for an equally large number of types of geosynthetic, the applications can be grouped into four main functions.

i. separation;
ii. reinforcement and retention;
iii. drainage and filtration;
iv. impermeable barrier.

Separation

5.81 Geotextiles may be used to separate two different materials which would otherwise have a tendency to mix under working conditions. There is also an added function whereby the geotextile, being permeable, allows the passage of water across it to reduce the build-up of undesirable pore water pressures in either of the adjacent materials. Clause 609 SHW defines some of the minimum physical properties that the geotextile must possess. If different values of these, or other, properties are required then they should be inserted in Appendix 6/5. The NG Sample Appendix 6/5 gives details of the information to be included. The amount of lapping of the geotextile is specified as 300mm minimum, but the Designer should consider the maximum possible amount of differential settlement and deformation that could occur in the soil close to the geotextile. If the differential settlement is more than 200mm the minimum lap stated in Appendix 6/5 should be increased accordingly. In normal circumstances, the geotextile should not be physically jointed but allow the material to slide and not `bridge' over low areas where it could become overstressed and tear.

Reinforcement and Retention

5.82 Geosynthetics are widely used as soil reinforcement in embankments and in fill material behind retaining walls or similar structures, and most of the types included in Paragraphs 5.73 to 5.79 are suitable for this use. Embankments can be reinforced for a variety of reasons and in a variety of ways. The basic requirement for the reinforcement is that it must be able to sustain the allowable load over the specified period of time without its strain exceeding a predetermined limit. If the reinforcement achieves the allowable load at large strains then the soil structure will probably have already failed and therefore the maximum allowable strain should be determined by the limitations of the structure. A series of creep tests carried out in accordance with TRRL Application Guide 5, will allow isochronous curves to be produced enabling allowable loads to be determined for different design lives. The reinforcement should also have a coefficient of interface friction with the adjacent soil of sufficient magnitude to prevent failure. This coefficient of friction should be determined using a method similar to the testing requirements of Clause 639 SHW. It is necessary, to ensure that the soils provided during construction have the same characteristics as those assumed for design purposes.

5.83 The technical literature published by the manufacturers usually provides detailed information regarding the physical properties of their products, but a Designer is advised to acquire evidence that the particular product which the Contractor proposes to use satisfies the design requirements for the reinforcement, and the details of the required evidence should be included in the reinforcement specification. It should be noted that the properties of geotextiles (according to BS 6906) and related materials in isolation may be different to the properties when placed in soil, and care must be exercised that the appropriate material properties are chosen.

5.84 The Department's Technical Memorandum BE 3/78 (revised 1987) Reinforced and Anchored Earth Retaining Walls and Bridge Abutments for Embankments provides design criteria and construction requirements for reinforced and anchored earth retaining walls.

Drainage and Filtration

5.85 Geotextiles and related products are also commonly used as drains and filters. Such applications include the following.

i. For use in narrow filter drains to wrap free draining granular fill, containing a perforated pipe (Drain Type 9, HCD). An alternative option is to wrap the pipe directly in a geotextile and fill around and above it (Drain Type 8, HCD).

ii. To form a horizontal drainage blanket under embankments. Here the geotextile can also act as a separator between the embankment and its foundation.

iii. To form fin drains which are usually of composite construction with geotextile filters sandwiching a plastic core which allows the water to flow between the geotextile layers to a carrier pipe (Drain Type 6 and 7, HCD). Some of these cores are designed to have sufficient capacity so as not to require a carrier pipe (Drain Type 5, HCD).

5.86 In all the above applications the function of the geotextile is to retain the soil particles whilst allowing the water to pass through; the size and number of the openings in the geotextile are therefore critical. The filter fabric must be chosen bearing in mind the grading and particle size of the material surrounding the drain as well as the estimated ground water flow.

Impermeable Barrier

5.87 This function applies to geomembranes which are commonly used to contain or exclude liquids and gases. They are frequently used in areas of contaminated land or for construction below the water table level.

Material Durability and Degradation

5.88 A further aspect of the choice of material is the possibility of degradation by environmental agents. Each of the polymers has varying durability and resistance to degradation, and an indication of the degree of resistance of the main four polymers to the various agents is given in Table 5/1. However, very little in the way of degradation testing and long-term durability assessment has been carried out and therefore advice in this area is limited.

5.89 A case has been reported where geonet gabion baskets caught fire after installation and continued burning even on the buried faces until all structural integrity was lost resulting in total collapse of the filling material. Cutting of the geonet by vandals can also lead to loss of structural integrity. Therefore it is recommended that these gabions are not used in exposed positions or critical locations where such a collapse could have serious consequences. Fires on the slopes of reinforced earth embankments are unlikely to affect the geosynthetic if adequate topsoil cover has been provided.

TABLE 5/1

The degree of resistance to degradation

AGENT	POLYESTER	POLYETHYLENE	POLYPROPYLENE	POLYAMIDE
Acid	2	1	1	1
Alkali	3	1	1	2
Dry Heat	2	3	3	3
Moist Heat	3	3	3	2
Abrasion	1	2	2	1
Fungus	4	1	2	2
Oxidising Agent	2	4	2	3
U V Light	1	4	2	2

1 - Excellent 2 - Good 3 - Fair 4 - Poor

Taken from 'Reinforced Earth' by T S Ingold; Thomas Telford 1982

6. SLOPE STABILITY ANALYSIS

General

6.1 The stability of soil and rock is a complex subject and it should be emphasised that failure to understand the engineering significance of the geological strata and groundwater is invariably a more important factor in slope failures than the method of stability analysis employed.

Soil and Soft Rock Slopes and Their Analysis

6.2 Considerations of the stability of cuttings, embankments or natural slopes must always start by defining the topography and geological aspects of the problem. Once the topography, geology and groundwater conditions are known the Designer will be able to make a preliminary assessment of possible safe side slopes based on previous experience in similar strata, and also make a comparison with what is stable or not within the scheme area. However soils of the same geological horizon can vary locally and the detailed ground investigation must check very carefully for any discontinuities or weaker horizons which may affect the stability of the slope. In particular, consideration needs to be given to the dip of any natural strata and direction of any discontinuities in relation to the proposed slope, and the possibility of any pre-existing shear surfaces. The effect of exposure of soil and rock strata to weathering processes must also be taken into account. All these considerations will affect the value of shear strength parameters used in the analysis.

6.3 The Designer should develop an understanding and appreciation of the factors that affect slope stability by using analysis methods that can be done by hand before progressing to methods requiring a computer. Sophisticated computer programs and analysis techniques should only be used when the Designer understands the processes involved and is confident that it is the correct method for a particular situation. Search programs which locate the most critical slip surfaces can be valuable but the Designer must be aware of their limitations. Parametric studies are valuable to demonstrate likely variations in the calculated factor of safety due to a range of input parameters. Unless the soil parameters and water pressures have been accurately determined, sophisticated methods of analysis are not justified.

6.4 In determining acceptable factors of safety for slopes the designer must carefully consider the consequences of failure (ie risk to life or property), the reliability (and conservatism) of the parameters used in the analysis and the accuracy of the method of analysis. (Most methods are subject to error from effects such as, three dimensions, horizontal stresses, progressive failure etc). The most reliable factor of safety is likely to be that based on parameters derived from back analysis (by the same method) of a failed slope in the same soil strata.

6.5 In low permeability materials such as clays, the stability assessment must consider stability both in the short-term, during and immediately after construction, and in the long-term, when pore water pressure equilibrium has been achieved. Which condition proves the more critical is likely to depend largely on whether the ground is subjected to a net increase or decrease in loading by the construction. For an increase in loading, such as the construction of an embankment on a foundation of soft soil, the critical condition is likely to occur in the short-term. For unloading, as in the case of a cutting in stiff clay, long-term conditions are likely to prove the more critical.

6.6 Effective stress methods of analyses should be used for determining stability in more permeable (granular) soils and for assessing the long-term stability in cohesive soils. Effective stress analyses can also be used to assess the short-term stability in cohesive soils provided that adequate information is available on the pore water pressure regime. However, the prediction of pore water pressures during construction in cohesive materials is difficult and for preliminary assessments of stability, consideration should be given to the use of simpler total stress methods based on the undrained strength of the soil. An inherent assumption of such analyses is that no drainage occurs during the construction period. Measurements of undrained strength are often unrepresentative because of problems of rates of testing, confinement conditions and discontinuities in the soil and are likely to depend on the test method. Total stress methods are therefore likely to be most reliable in situations involving soft homogeneous soils.

6.7 There are various methods of stability analysis which may be used to estimate the factor of safety of a slope, most of which use variations of the `method of slices'. The methods differ mainly in the manner in which interslice forces are considered and the shape of failure surface investigated. For circular shaped failure surfaces, the Swedish solution gives very conservative factors of safety when pore water pressures are present. The Bishop Simplified Solution can overestimate or underestimate the factor of safety for certain deep slip surfaces, but generally gives fairly accurate results. The simple method of Greenwood and the simplified method of Morrison and Greenwood may be used for circular shaped failure surfaces and slip surfaces of arbitrary shape. Suitable methods for slip surfaces of arbitrary shape include Janbu Simplified Method, with correction factors as necessary; Janbu Rigorous Method; Morgenstern and Price, although considerable experience and judgement is required to use this method reliably; and Spencer's Method which was originally devised for circular failure surfaces but has been adapted for non-circular failure surfaces. The infinite slope analysis is best used for extensive planar slips where the slip surface is approximately parallel to the ground surface and the influence of the toe and head portions of the slide is negligible. References to the above methods are given in Chapter 14.

6.8 Stability charts can be helpful but their use is restricted to uniform non-layered soils and slopes of regular geometry. Use of charts might restrict the Designer's understanding of the mechanics of slope stability analysis. Charts such as those of Bishop and Morgenstern (Geotechnique, Vol 10, No 1, 1960) are commonly used.

6.9 More sophisticated methods such as `Sarma' are available but these require computer assistance and considerable geotechnical expertise to apply.

6.10 Simple three part wedge analyses, based on active and passive wedges with a middle sliding section, are often most appropriate for embankment stability where the risk of a near horizontal weaker stratum is to be investigated. Such a calculation uses a different definition of the factor of safety and is likely to result in a somewhat different assessment of stability. Other more sophisticated wedge type solutions are available.

6.11 When geomesh reinforcement or ground anchors are included in a slope an initial appraisal of their potential benefit may be obtained by including the reinforcement forces in the Swedish and Greenwood simple (Greenwood 1990) methods of analysis. Embedment lengths and strain compatibility between reinforcement and soil must be carefully considered.

6.12 More rigorous methods of analysis of reinforced soil slopes are available, but care is needed in the choice of materials and design assumptions in this relatively new and developing field.

6.13 A survey of slope condition of motorway earthworks by Perry (TRRL RR 199) provides a valuable source of empirical information for new road construction and maintenance. The survey discovered considerable lengths of shallow failures on the slopes of both cuttings and embankments.

Rock Slopes and Their Analysis

6.14 Rock masses contain numerous planar features which can reduce the bulk strength of the rock considerably. These features such as bedding planes, faults and joints are collectively known as discontinuities and the rock mass is divided into blocks whose shape and size are dictated by dip (inclination), azimuth (direction of the dip) and the frequency of these discontinuities.

6.15 When designing rock cuttings the steepest side slope compatible with stability is generally chosen to reduce the quantity of rock to be excavated and therefore the cost. The stability of rock side slopes is governed by the bulk strength of the rock mass, the excavation technique and the design chosen. The most stable design will be that which matches the rock discontinuities, thereby reducing any subsequent remedial and maintenance works on the slope. This is usually achieved by preventing discontinuities `daylighting' on the slope (plane failure) wherever possible by making the dip of the slope less than the dip of the discontinuities into the cutting, or preventing the line of intersection of two discontinuities from emerging above formation level and daylighting onto the slope (wedge failure). Toppling failure can occur where near vertical columns of rock, which lean into a cutting or excavation, fail along near horizontal discontinuities or by failure of the rock columns as they flex. The Designer will obviously require the maximum information which can be obtained economically from both sub-surface exploration and surface geological mapping before attempting to assess the potential stability of the excavated faces.

6.16 Further information and guidance on rock slope stability, methods of assessment and preventative measures is given in Hoek and Bray (1981), TRRL LR 1039, Matheson (1989), Hudson (1989) and a comprehensive description of available rock support systems is given in Hoek and Wood (1988).

7. CUTTINGS

General

7.1 Cuttings are excavated in the existing ground and therefore the nature and condition of the natural strata should be thoroughly investigated. The ground water regime represents the most critical aspect of cutting design, and attempts must be made during the Ground Investigation to determine water levels, seasonal variations and soil permeability. Cutting slope design must be considered in both the short-term, during or shortly after construction, and long-term, many years after construction. Drainage should be considered for both surface and ground water during these periods.

Stability

7.2 Cutting instability may occur soon after excavation if there are any undetected discontinuities, ie fissures or existing shear surfaces, in the soil structure or if seepage is allowed to occur from semi-permeable strata such as silts.

7.3 TRRL Report RR 199 lists the slope angles, for a wide range of geologies, at which slopes remain stable; these are based only on engineering considerations and account must be taken of land take costs, environmental impact and possible uses of the slope.

7.4 In the long-term, over-consolidated clay soils are prone to softening as the pore pressures, which are initially reduced due to the removal of overburden load, return to equilibrium. This can result in shallow failures as mentioned in Paragraph 5.34. Assuming a reduction of the slope angle to allow for the weakened material is not feasible, the designer has the following options in order to retain steep slopes in cuttings:-

i. to install relatively shallow trenches, filled with stone (slope drains or rock ribs) normal to the slope, and from top to bottom, to remove water; this solution is economic (TRRL RR 30) and may be perfectly adequate (TRRL RR 199).

ii. to install deeper trenches than in (i) above, filled with stone (counterfort drains) normal to the slope, and from top to bottom, which buttress the slope as well as draining the soil to considerable depth; this solution is, however, expensive;

iii. in some circumstances it may be more economic to allow some slope failures and accept higher maintenance costs;

iv. use soil nails or soil anchors with geotextiles to retain the soil in order to prevent failures.

7.5 Slope design should be based on equilibrium water levels with permanent drainage requirements kept to a minimum, consistent with reasonable side slopes, to reduce maintenance commitments.

7.6 Where the cutting is excavated in side-long ground where there is a possibility of relic landslips or slickensided ground, care should be taken to:-

i. ensure adequate drainage (see Paragraphs 7.7 to 7.11 below);

ii. check exposed strata;

iii. watch for signs of fresh instability.

Groundwater

7.7 If stability cannot be achieved without slope drainage, consideration may be given to the use of interceptor drains, deep counterfort drains, shallow batter or blanket drains.

7.8 The filter materials used in slope drains must be designed for the maximum flows likely but bearing in mind the risk of loss of ground if the filter is too coarse. Because natural sand and gravel are often very variable with water flows concentrated in the coarser gravel layers, it may be expedient to use a coarse gravel drainage material surrounded by a filter fabric to prevent silting and loss of ground. In the same way it is extremely difficult to predict water flows in fissured clays. It should also be recognised that the groundwater level at the GI stage may not be the longer term level for cutting design; the level may be affected by a spell of dry weather or water extractions.

7.9 It is for the Designer to ensure that the permanent drainage is adequate for the stability of the completed cutting. If, however, temporary drainage is required for stability during excavation, or until the permanent drainage can be brought into play, then the Appendix should make this clear and the MMHW amended to suit. The Contractor must be made aware that he is responsible for maintaining any temporary drainage required and he should not assume that any drainage which is part of the design will necessarily be adequate for dewatering earthworks for the purposes of rendering soil acceptable.

7.10 Where the natural ground slopes towards the top of a cutting or any erodible soil, a lined interceptor drain or lined ditch may be required to prevent erosion or instability occurring. During design and subsequent excavation, care is needed to identify any existing field drains and intercept their flow before it reaches the cutting face.

7.11 Placing of drainage ditches relative to the top or bottom of a slope may influence the stability of the slope by creating soft areas and can be associated with the formation of failure planes.

Foundations

7.12 When considering structural foundations in cuttings, the Designer must have an understanding of how the soil will behave during excavation, construction and beneath the completed structure. Factors to be borne in mind must include the probable effects on the foundation material of frost heave during construction and after. Changes will occur in the ground water regime from both temporary drainage schemes during construction and permanent drainage. Consequently, the ground water levels as well as the flow directions will vary.

7.13 The material itself may suffer changes due to the removal of surcharge by the excavation of the cutting, by exposure to the weather during construction (moisture susceptibility) and by the application of working loads. Over-consolidated clays can soften when exposed to water with corresponding loss of bearing capacity, swelling etc.

Explosives and blasting

General

7.14 The term `explosives' includes both high explosives and also slow-burning materials such as propellants. Hence devices incorporating a cartridge resembling a shot-gun cartridge for boulder or rock splitting are also covered by SHW Clause 607.

7.15 The use of blasting for excavation should only be allowed in SHW Appendix 6/3 where it is considered to be necessary. Where it is considered not likely to cause damage or nuisance, the contractor should be permitted in Appendix 6/3 to use it as an alternative method of excavation.

7.16 To avoid intrusion, permitted hours for blasting entered in Appendix 6/3 should not extend outside normal working hours except where the site is remote from any inhabited area.

7.17 For further advice, TRRL Report RR 53 should be consulted. Among other subjects this Report covers blasting trials, establishment of vibrational limits (damage criteria), vibrational measurement procedures and equipment, and establishment of site scaling laws.

7.18 At the pre-construction consultation stage a condition survey of neighbouring properties at risk should be carried out as described in BS 5607: 1978 Clause 4.2. Photographs should be included in the survey. Departmental Advice Note HA 34/87 `Ground Investigation Procedure' stresses the need for condition surveys `before, during and after the execution of the main Works'. At the same time notice should be given regarding the need for temporary and permanent protection and for monitoring as the work proceeds. It should be noted that the permission of the Project Manager must be obtained before any condition surveys are carried out.

Plaster Shooting

7.19 Plaster shooting is defined in BS 5607:1978 as `blasting by placing a quantity of explosive against a rock, boulder or other object without confining the explosive in a shot hole'. While this procedure may be acceptable in rock quarries it is too dangerous for use on construction sites and has been prohibited in Clause 607.

Pre-Split Blasting

7.20 This is dealt with in SHW Clause 603 but is subject to the requirements of Clause 607.

7.21 Pre-split blasting is employed to reduce the harmful effects of bulk blasting on the final rock face. It involves the drilling of closely spaced holes along the design slope, charging the holes relatively lightly and firing the charges to form a fracture plane along the slope, before firing the bulk blast. The time interval between the two firings must be at least 50 millisecs. The adverse effects of bulk blasting for the rock excavation on the design slope are therefore almost eliminated by the pre-split fracture plane so that disturbance of the rock in the final slope is minimal. Whilst pre-splitting can initially be expensive, its cost may be offset by the subsequent lack of remedial and long term maintenance works to the rock slope and the designer should consider the whole-life costs of the cutting to see if pre-splitting is economically desirable. For further details and guidance, including reference to the important matter of drilling accuracy, TRRL Reports LR 1094 and SR 817 should be consulted in addition to NG 603.

7.22 Flattening the rock slope may be a cheaper option to pre-split blasting, from the maintenance point of view, in some cases such as when land take is not a problem and the rock weathers quickly on exposure.

Blasting Trials and Trial Explosions

7.23 Pre-contract blasting trials are required by Clause 607 and are fully covered by TRRL Report RR 53. These should preferably be carried out at the ground investigation stage and, if comprehensive, will reduce the extent of the trial explosions conducted by the Contractor at the Main Works stage. Trial explosions should start with small charges and increase to charges similar to the working charges but only if the measurements show that it is safe and environmentally acceptable to do so.

Safety Matters

7.24 In addition to BS 5607:1988, manufacturers safety publications such as `Explosives - Safe Practice and Storage' published by Nobel Explosives Co Ltd should also be consulted. BS 5607 lays down rules for safe blasting on construction sites where a public highway is in use, as in a road improvement or maintenance scheme. Clause 3.12 of BS 5607 defines `danger area'. The temporary works required to retain projectiles are also described in BS 5607. Suitable matting for use in pre-split blasting to contain air blast and reduce fly rock is described in TRRL Report LR 1094 page 36.

Vibrational Limits

7.25 Factors affecting vibration levels when explosives are used include:-

i. size of charge;

ii. pattern of charges;

iii. confinement of charge;

iv. geology and nature of ground eg dip, depth and type of rock, and presence of fault planes.

7.26 The safe level of peak particle velocity (ppv) is governed by the type and state of repair of the structure affected and very importantly by the frequency of the vibrations in relation to the natural frequency of vibration of the structure. Peak particle velocities in excess of 50mm/sec (up to say 100mm/sec) may be safe and acceptable for some structures. However, lower limits may be necessary in other cases. This subject is dealt with in TRRL Report RR 53 and reference should be made to Figure 2 on Page 4 of this publication which gives ppv's for residential structures; these can be much less than 50mm/sec depending on the frequency of vibration.

7.27 The upper limit of 0.2mm vibrational amplitude given in Clause 607 SHW is considered to be more in line with case history data than the limit of 0.1mm quoted in the Fifth Edition of the Specification. Amplitude criteria should only be considered where the predominant frequency of the motions is usually low (<5Hz).

7.28 The delay between explosions necessary to avoid superposition of vibrations from successive delays should be determined during trial blasting.

7.29 Good public relations attitudes and an education programme by the blasting Contractor are essential. Human reactions to vibration can be limiting factor and complaints can arise at ppv levels as low as 2mm/sec. ISO Standard 2631 (1978) and BS 6472:1984 both provide valuable guidance on acceptable levels of human exposure to vibration. Human response should be considered when determining a criterion for ppv but should not be regarded as paramount provided the Contractor keeps the public well-informed.

Peak Overpressures, and Damage to Windows and Glazed Areas

7.30 An accepted maximum safe value for air-blast or peak overpressure is 0.7kN/m2 but this value is unlikely to be reached where the ground vibration ppv is kept below 50mm/sec. Windows may rattle with a peak overpressure of 0.3kN/m2 (see Nobel Explosives Co Ltd Booklet) and this will cause alarm to the public unless they are kept well-informed.

Instrumentation and Measurements

7.31 TRRL Report RR 53 describes the procedures and equipment necessary for effective vibration studies. It should be noted that instruments exist which can measure both the peak overpressure associated with air-blast and ground vibration in three orthogonal planes. In deciding whether the Engineer or the Contractor should make arrangements for the installation of instruments for the monitoring of property off the construction site, (Appendix 6/3 to the Specification), the nature and condition of the property should be taken into account and an assessment made of the sensitivity of the situation regarding the occupiers of such property. Similar considerations apply to the reading of the instruments and the reporting of the results.

8. EMBANKMENTS

General

8.1 Embankments have the basic function of providing a stable foundation for the road, and therefore must be sufficiently strong to carry the imposed loads, and remain stable in terms of settlement and slope failure. Assumptions have to be made at the design stage as to the nature of the material that will form the embankment, which is usually material from adjacent cuttings unless an imported material is specified. The design aspects to be considered are:-

i. possible failure of the embankment slopes;

ii. possible failure of the underlying strata;

iii. both (i) and (ii) together;

iv. settlement of the embankment fill;

v. consolidation of the underlying strata.

Advice on the construction of embankments adjacent to piled foundations is given in BA 25/88, Piled Foundations.

Stability

8.2 Embankment fill material is usually selected on the basis of minimum strength criteria and does not generally have problems of instability during or immediately after construction provided compaction is adequate.

8.3 Granular embankments are usually stable with settlement occurring immediately; however some rock fill embankments may show appreciable settlements with time. Clay embankments may be subject to a longer settling period whilst the moisture content reaches equilibrium for the new overburden load. This may take several years to occur and may be countered by expansion of clays which have been excavated at depth. As explained in Paragraph 5.34, the stiffer over-consolidated clays are particularly prone to moisture content increase and subsequent loss of strength when placed at the top, base and in the slopes of embankments. Experience has shown that over-consolidated clay soils tend to become unstable a few years after construction on embankment slopes steeper than or equal to 1 in 3 (1 vertical to 3 horizontal units), although failures can occur at 1 in 4 but they are infrequent at the present time. TRRL Report RR 199 includes tables which suggest the most suitable slope angle for an extensive range of materials in slopes of varying height. Slope angles steeper than those given will lead to failures in the long-term. It should be borne in mind that, in addition, the economics of land take, the environmental impact of a scheme and the expected use of the slope may all have an affect on the choice of slope angle.

8.4 Embankments slopes of 1 in 2 are often required and therefore if over-consolidated clays are used either the risk of shallow slope failure must be accepted as a maintenance liability or the over-consolidated clays excluded from the slopes of the embankment. Alternatively the slopes may be strengthened as described in Paragraphs 8.9 to 8.13, or buttressed with granular material. The use of tamping rollers on these clays during construction much reduces the risk of premature failure by breaking down aggregates of clay and by disrupting polished surfaces left by smooth wheeled rollers (Whyte and Vakalis, 1988).

Drainage

8.5 Surface water from carriageways should be collected by channels discharging into toe ditches to reduce the risk of surface water entering the pavement layers and the subgrade. The use of surface channels is preferred with water being led away to ditches at the toe of embankments or other available out-falls. Drainage channel blocks may be used down embankment slopes to take surface water from gullies or surface channels to toe ditches or outfalls.

8.6 Over the edge drainage is permitted providing the method has not led to problems in the past or when used in particular situations (see Advice Note HA 39/89). The effect of wetting material in the embankment shoulders should be considered in the design.

8.7 In over-consolidated clay embankments extended sub-base layers have in the past been used to dispose of any water which finds its way into the lower pavement layers. This may contribute to softening and instability of the side slopes. The recommended method of interception of sub-surface water is by means of fin drains (Types 5, 6 and 7, HCD) or narrow filter drains (Types 8 and 9, HCD). Drains and any services in embankment shoulders should be avoided, but if necessary they should be carefully installed as they always carry a risk of creating potential tension cracks allowing subsequent ingress of water.

8.8 When the highway passes from cut to embankment on a downgrade there is sometimes a possibility of groundwater collecting at the interface. Interceptor drains may be installed across the highway at the end of the cutting to collect this water if the ground investigation indicates a problem area.

Strengthening Slopes

8.9 As mentioned in Paragraph 5.34 and Paragraphs 8.2 to 8.4, embankments constructed of over-consolidated clay may develop shallow slips in the long-term. To counteract this development in embankments, and to retain steep slopes, the Designer has some options available other than reducing the slope angle.

8.10 Reinforcement of the embankment slopes by substituting the outer layers of clay fill by materials that are not susceptible to long-term softening such as a granular material. This option usually depends on the availability of the material on-site or suitability of any close by and economic off-site sources.

8.11 Reinforcement of the outer layers of the embankment during construction, using polymeric reinforcement such as geotextiles or geogrids.

8.12 Options (i), (ii), (iii) and (iv) of Paragraph 7.4.

8.13 The advantages of (ii) Polymeric Reinforcement are that it can be installed during construction of the embankment, it does not require the importing of an alternative material or the simultaneous placing of two different types of material adjacent to each other, it is relatively inexpensive, and by varying the vertical spacing and length of embedment of the reinforcement layers, the embankment slopes may be steepened beyond 1 in 2 if required. Further advice is available from the Department.

9. GROUND CONDITIONS REQUIRING SPECIAL TREATMENTS

Weak Materials Beneath Embankments

9.1 Embankments impose considerable shear forces on the basal deposits of the embankment fill and the upper layers of the foundation soils. These areas must be carefully investigated for any weak layers which could cause instability.

9.2 Where soft compressible soils are present, the stability and settlements require careful consideration as described in the following sections. Stability problems also arise in some stiffer materials. These materials were subjected to a more severe climate in the geological past and many have developed low strength pre-existing shear surfaces when located in sloping ground. These surfaces can be difficult to detect and road construction may reactivate the slope movements.

9.3 Restrictions on the rate of either the construction or any staged loading or both may be necessary for many embankments. All embankments should be carefully observed during construction for signs of excessive deformation. Areas particularly at risk may need to be fully instrumented with readings of pore water pressure, settlement and deformation taken regularly as construction proceeds. A line of toe pegs observed regularly for line and level can provide a valuable warning of any problems and these should be installed on all major embankments. Details and rates of controlled filling are to be set out in SHW Appendix 6/3 and instrumentation details in SHW Appendix 6/12.

9.4 Where basal drainage blankets (starter layers) are required by the design their function and permeability must be checked to ensure that water drains freely and they do not act as a reservoir of water which could soften adjacent clayey soils. Horizontal permeability of drainage layers may be measured by means of the `Permeability Box', (see HA 41/90: A permeameter for road drainage layers). In some circumstances the Designer may find his requirements for a basal drainage blanket are met by Class 1C material which can be a cheap and effective alternative to the more difficult to produce Class 6B material.

9.5 Cut/fill transition areas are a common source of sub-grade problems and should also be given special attention. The material immediately below topsoil may be weathered and produce a poor formation and weak sub-grade. Therefore it may be prudent to excavate this material and replace it with a selected material over a suitable area. The appropriate drawing showing details of the treatment should always be included in the contract.

9.6 Soft alluvial deposits always require special attention. Removal of the soft deposits and replacement with acceptable fill material may be economical for limited depths particularly if the area can be readily dewatered. If left in place the soft deposits must have, or develop, sufficient strength to maintain stability during construction, usually staged, and consideration must be given to the likely settlements both in the short-term and in the long-term, under the embankment loading. Embankments on compressible foundations should be built early in the contract so that sufficient time is allowed for complete primary consolidation before pavement construction. Pre-loading, with or without surcharging, allows some or all of the consolidation and dissipation of pore pressures to take place before the road pavement is constructed. Where this method is employed, calculations must be made to optimize the amount of pre-loading with regard to stability. Consideration must also be given to improving drainage using vertical drains and monitoring progress using instrumentation. The Contract Period must be adjusted to include this period of pre-loading, or advance works, carried out prior to the main works contract.

9.7 Analysis will usually be required in order to determine the likely behaviour of weak material as a foundation including finite element analysis for special cases.

Open Water

9.8 Where possible, embankment construction should only commence when any ponds or areas of water have been drained. Where this is not possible a self compacting granular material may be placed by end tipping into the water until sufficient height is achieved to continue with normal construction methods. Soft deposits immediately beneath the embankment must be carefully checked as instability can occur. If varying water levels are possible, the embankment should be designed to withstand the likely drawdown conditions.

Ground Treatment

9.9 Various techniques are available for improving the strength of foundation soils and some are mentioned in this Chapter. Cohesive soils containing pre-existing shear surfaces, may need to be excavated below the critical toe area of the proposed embankment and replaced with acceptable material. The re-use of the clay after reworking has been found to be satisfactory although it is prone to softening following the release of high insitu stresses. Foundation excavations should have battered sides and be backfilled as soon as possible after excavation. Groundwater must not be allowed to enter or stand in the open excavations. Lowering of the water table by deep trench drains may provide some improvement in the factor of safety. If solifluction is known or suspected, it would be wise to destroy the shear planes and provide deep drainage facilities.

9.10 Alternatively, and possibly more economically, the base of the embankment may be reinforced with one of a variety of methods using geosynthetics. This would give adequate factors of safety with the sheared material still present. Such methods would include geogrids or geotextiles laid transversely across the base and turned back at each end to form an anchor arrangement: embankment stability is then provided by the tensile strength of the geosynthetic. Also geogrids may be used to form geocells and vertical web foundations filled with selected material.

9.11 Other ground improvement techniques include deep compaction by vibration to achieve settlement of loose non-cohesive granular soils or fill materials above or below the water table, up to depths of 25m. Columns of coarse granular material can be formed in either soft clays, silts or compressible fills by vibro-displacement and vibro-replacement to reduce their compressibility and improve shearing resistance. In some circumstances it may be possible to alter the engineering properties of the ground by grout injection, and a number of grouts and techniques are available to deal with specific problems. More details of these treatments can be found in BS 8004:1986, Foundations, Section 6.

9.12 Many of these techniques require specialist contractors and further detailed information can be obtained from the technical literature. Whichever technique is adopted the Designer should ensure that his solution does not in itself create a new failure surface.

9.13 Ground improvement details should be provided in SHW Appendix 6/13. Earthworks details for excavation and replacement etc should be detailed on the drawings and in SHW Appendix 6/3. The Designer should be aware of the Specification for Ground Treatment and Notes for Guidance published by ICE in 1987.

9.14 The rate of consolidation of soft compressible soils may be increased, if necessary, by the installation of vertical drains to reduce the drainage path, so assisting the outward migration of the water. Installation of such drains can cause smearing of the faces of the vertical bores thus reducing their effectiveness. A drainage blanket is usually included and consideration may be given to the use of geomesh layers to improve stability and spread the effects of irregular settlement. Lightweight fills (ie PFA, see Paragraphs 5.58 to 5.66) or ultra lightweight fills (ie Expanded Polystyrene Foam to BS 3837) may be considered to reduce the settlement and instability problems. Further advice may be obtained from the Department.

Groundwater Lowering

9.15 Groundwater lowering may be either `active' or `passive'. Active techniques include well-pointing or deep trench drain systems which temporarily or permanently lower the water level. Passive techniques are those which are designed to reduce the drainage path and thereby decrease pore water pressures generated by increased loadings. Typically vertical wick drains, sand drains and drainage blankets beneath embankments can fulfil this role.

Voided Ground

9.16 If the highway passes either over or close to underground voids, where there is a danger of subsidence affecting the road, it is common practice wherever possible to fill these voids as a preventative measure.

9.17 Underground voids are either man-made or of natural origin. Natural cavities are usually the result of the flow of water through soluble rocks. If they still form part of an underground watercourse they may present special difficulties, since any measures to fill the cavities could interfere with the ground water flow and could cause problems upstream or downstream. Therefore the consequences of such a filling should be thoroughly investigated beforehand. In some cases cavities are disguised by being covered with detritus, such as is sometimes found in swallow holes in chalk and magnesian limestone, and are only discovered during construction. It is therefore sensible to allow for this when the scheme passes through these materials, and a detailed specification of the treatment required for such natural voids included in the Specification. One suggested method of dealing with swallow holes is to fill them with a granular material, sealed over by a layer of clay or other relatively impermeable material such as lean-mix concrete.

9.18 Man-made cavities are usually the result of mining, whether for building stone, coal or other minerals. The procedure for locating these mines is given in Departmental Advice Note HA 34/87, Ground Investigation Procedure. Guidance on measures to render openings into coal mines safe is given in a British Coal publication `The Treatment of Disused Mine Shafts and Adits'. Further advice on measures to render such openings safe is given in Departmental Advice Note HA 34/87, Ground Investigation Procedure. British Coal must be consulted and informed, at all stages, about any operation which will affect their workings. The treatment of other mineral mines may be dealt with in a similar manner, and further advice may be obtained from CIRIA Special Publication 32, Construction over Abandoned Mine Workings as well as Departmental Standard BD 10/82, The Design of Highway Structures in Areas of Mining Subsidence.

9.19 A method for the stabilisation of ground affected by shallow mine workings is the use of grout injection. This is a noisy and dirty operation, and care should be taken in specifying the equipment to be used especially in urban areas. Air flushed drilling rigs should be fitted with a suitable dust extractor to prevent the emission of dust or other airborne particles. All drilling rigs and associated pneumatic equipment should be fitted with mufflers or silencers of a type recommended by the manufacturer, and all compressors should be `sound reduced' models fitted with properly lined and sealed acoustic covers which must be kept closed when the machines are in use. Suitable provision should also be made to eliminate any dust problems caused by wind action on stockpiles of grouting materials, particularly PFA.

9.20 Embankment bases may be reinforced by geogrids over ground where differential settlement is anticipated above filled voids, or the embankment may be surcharged to accelerate any such movement. Instrumentation, as discussed in Paragraphs 4.45 to 4.50, should also be considered to monitor the ground profile during and after construction.

Dynamic Compaction

9.21 Clause 617 in Appendix A of Departmental Standard HD 6/80 is superseded by Clause 630 of the 7th Edition of the Specification for Highway Works. The Method of Measurement for the 7th Edition supersedes Appendix B of Departmental Standard HD 6/80. The advice given below supersedes that given in Departmental Advice Note HA 11/80 and is intended to assist in the compilation of the data to be inserted in Appendix 6/13.

9.22 Dynamic Compaction (DC) is the process by which a rapid increase in the density of soil or other material is achieved by dropping a free-falling heavy mass (pounder) a number of times from pre-determined heights at pre-determined spacings onto the surface of the ground or fill. This rapid densification is produced by the expulsion of air, reduced spacing and rearrangement of the particles by mechanical means and results in settlement of the ground surface. However, this is different from the Menard system of `dynamic consolidation' which employs the improvement of the properties of saturated soft fine-grained soils by means of the expulsion of water. Whereas DC has been used extensively in Continental countries for the treatment of soil on roads and other civil engineering sites, experience in Great Britain is more limited. The Engineer is, therefore, often obliged to follow the advice provided by the specialist firms which carry out this process which, in many cases, will result in an end-product specification being followed rather than a method specification. In other cases the choice between method and end-product specification will depend on the type of material to be compacted, the conditions on site and the experience of either the Engineer or the Contractor or both with the materials and methods used and the quantity of testing required for end-product specification.

9.23 DC is intended for the improvement of the foundation soils beneath earthworks, pavements and structures. Its use within earthworks would be exceptional. The process is not suitable for use on all soil types. In general, it is unsuitable for application to peat, soft heavy clays and probably soft medium clays and silty clays. At present it is advisable to restrict its use to granular material such as silty sands, sands and gravels, uncompacted man-made fill and refuse. Consideration should be given to its possible effects on the underlying layer of soil as well as the layer being treated. If the former is a soft clay, dynamic compaction of the material above it could have detrimental effects at the lower level, causing indentations which serve as reservoirs for ground water which is unable to drain away. Limited experience in this country does indicate that the process is particularly suitable for the treatment of deep deposits of domestic and industrial refuse where the alternative would be removal and replacement. In areas where open-cast mineral workings have been filled with uncompacted soil, surcharging has been found to be as effective as DC. In saturated low permeability ground with a high water table, particular care should be taken to prevent an excessive rise in pore water pressure and any liquefaction of the ground caused by over-tamping: piezometers should be installed before the treatment commences. From investigations carried out by the Building Research Establishment at sites where dynamic compaction has been used to compact loose fills, the depth of effectiveness Z in metres is given as a conservative approximation by:-

$Z = 0.4 \sqrt{(W.H)}$
where W = mass of pounder (Tonnes)
H = drop height (m)

9.24 Limited feedback suggests that where construction is delayed after Dynamic Compaction, further settlement may be expected, due to the dissipation of pore water pressures built up during the initial treatment: possibly as much as 50mm of settlement could occur over the 25 years after treatment.

9.25 Vibration caused by the falling mass will be transmitted throughout the ground and air and could conceivably cause damage to property off the site, as could flying debris. The close proximity of houses or other buildings may preclude the use of Dynamic Compaction and this must be considered at the early design stage.

9.26 Vibration may be minimised by decreasing the height of free fall while increasing the number of blows of the pounder. Consideration should be given to the construction of isolating trenches so as to inhibit the ground vibrations from reaching adjacent property.

9.27 The safe vibrational limits at adjacent structures are those specified for blasting in Clause 607 of SHW, and the advice given in Paragraphs 7.14 to 7.31 and TRRL RR 53 applies. If there is any risk of these limits being exceeded, an instrumented trial should be carried out.

9.28 The primary aim of the process is to achieve, in a relatively short period, settlements of the ground which would normally occur some time after completion of the works. where a high embankment is constructed by conventional methods on ground that has been dynamically treated, it is important to differentiate between any settlement within the embankment and settlement within the treated ground. Failure to do so may give rise to unresolvable argument as to where responsibility lies. It is therefore important to install suitable gauges which will indicate the degree of settlement within the two components of the work.

9.29 The total and differential settlements to be expected should be indicated in preliminary tests made by the specialist contractor. In specifying the maximum acceptable value, it is essential to bear in mind the time interval between completion of the embankment and the construction of the pavement. Higher values of acceptable total and differential settlement would be expected as this time interval increases.

9.30 While it is very difficult to specify values for the total and differential settlements at different stages of the works, since each site is unique, the following figures may give some guidance on the amount of settlement, in the treated ground, which can be tolerated in the time between completion of the embankment and trimming the formation:-

i. mean differential settlement between all pairs of adjacent gauge locations not to exceed
40mm or 1 in 250,

ii. maximum differential settlement between any two adjacent gauge locations not to exceed
80mm or 1 in 125,

iii. mean total settlement not to exceed
80mm

9.31 Settlements of the treated ground taken at later stages can be expected to exceed these figures by a proportion dependent on the time that has elapsed. In the long-term, settlement of treated refuse, which is subject to decay, may be difficult to predict to any accuracy.

9.32 The extent and magnitude of the densification achieved can be verified by appropriate strength tests including:-

i. plate bearing tests at the surface or in a trial pit;

ii. dynamic probing, static probing (including electric cone penetrometer) and self-boring pressuremeter all from the surface;

iii. pressuremeter tests, vane tests or penetration resistance tests in boreholes drilled for the purpose.

9.33 In selecting a suitable test, the type of material being processed, the accessibility of the site to plant and vehicles, and the presence of stones and larger objects in the material must be considered. In fills of rubble or refuse which contain large obstructions, only plate bearing or `ad hoc' load tests at the surface would be feasible. The equipment and procedure for all these tests is set out in Section 8 of `Specification and Method of Measurement for Ground Investigation Contracts. First Edition. 1987'.

9.34 Further information on a specification and test methods is given in ICE publication `Specification and Notes for Guidance on Ground Treatment'.

10. SUBGRADE AND CAPPING

Pavement Design

10.1 For the purposes of this Advice Note `pavement' covers all material from surfacing down to the upper surface of the sub-base: sub-base covers material between the bottom of the pavement and formation. Capping, and materials below sub-formation and formation, are earthworks materials (see Figure 10/1). The acceptability limits for earthworks materials are selected on the basis of how the material is to be used and its ability to be handled, compacted and trafficked. Each earthworks layer provides a platform upon which the next may be placed and compacted. In the same way the sub-formation is used as a platform on which to place and compact the capping and the formation is used as a platform for the sub-base. The pavement is designed on the basis that the sub-base can be constructed to a particular standard and that its integrity can be maintained. This requires construction on a formation of a certain quality, achieved by ensuring that the whole of an embankment, or at least the upper zone, is constructed of good quality materials. If the material in the upper layer is zoned then, in effect, a capping has been applied. In cuttings, where the subgrade may not be suitable as a platform for the sub-base, then a capping must be used.

10.2 The capping performs two functions.

i. In the short-term it acts as a working platform for construction of the sub-base, and provides protection for weak subgrades.

ii. Acts as a structural layer, in the long term.

10.3 Pavement design is covered by Departmental Standard HD 14/87 and Departmental Advice Note HA 35/87 `Structural Design of New Road Pavements'. The background to the design procedure is contained in TRRL Report LR 1132. As a reliance has been placed on the formation to provide a sufficient platform on which to place and compact the sub-base, the sub-base thickness, with one exception, does not vary for a given design. Table 10/1 shows how the normal sub-base thickness of 150mm must be increased to 225mm for flexible and flexible composite pavements on formations with a CBR percentage less than or equal to 15 but greater than 5.

10.4 It is the combination of subgrade strength, capping thickness and capping material which gives the required strength characteristics at formation. The capping materials and stabilisation processes described in the SHW are deemed to provide similar strength characteristics at formation. The only variables are, therefore, the thickness of capping, and strength and stiffness of subgrade. Appendix A of Departmental Standard HD 14/87 relates these variables in a series of steps shown in Table 10/1.

10.5 The designer needs, therefore, to be able to assess the CBR of the subgrade. In the design, a CBR of greater than 15% is required at formation for construction of rigid and rigid composite pavements and a CBR of greater than 5% is required at formation for flexible and flexible composite pavements. If the subgrade soil cannot achieve this then a capping will be required of sufficient thickness, depending on the CBR of the subgrade, to provide the necessary strength and stiffness at formation.

10.6 The materials permitted for capping have been chosen to meet the requirements for a formation of CBR of greater than 15%.

10.7 A further consideration is frost susceptibility. No material within 450mm of the designed final road surface shall be frost susceptible, as tested in accordance with Clause 602.19 SHW, whether used as pavement sub-base, capping, fill material in embankments or insitu material in cuttings. If the total construction depth as designed is less than 450mm, and the insitu material below is frost susceptible, then the capping or, when there is no capping, the sub-base must be thickened to give a total minimum construction depth of 450mm irrespective of CBR or other strength considerations. Frost susceptibility requirements for construction layers are defined in SHW Clauses 602 and 705.

Subgrade Assessment

10.8 The CBR is an index of bearing ratio used to indirectly measure soil strength and stiffness. Its ability to measure these depends upon the soil type and its state; see Hight and Stevens `An Analysis of the California Bearing Ratio Test in Saturated Clays'. The CBR has been correlated with pavement performance in the development of empirical design methods.

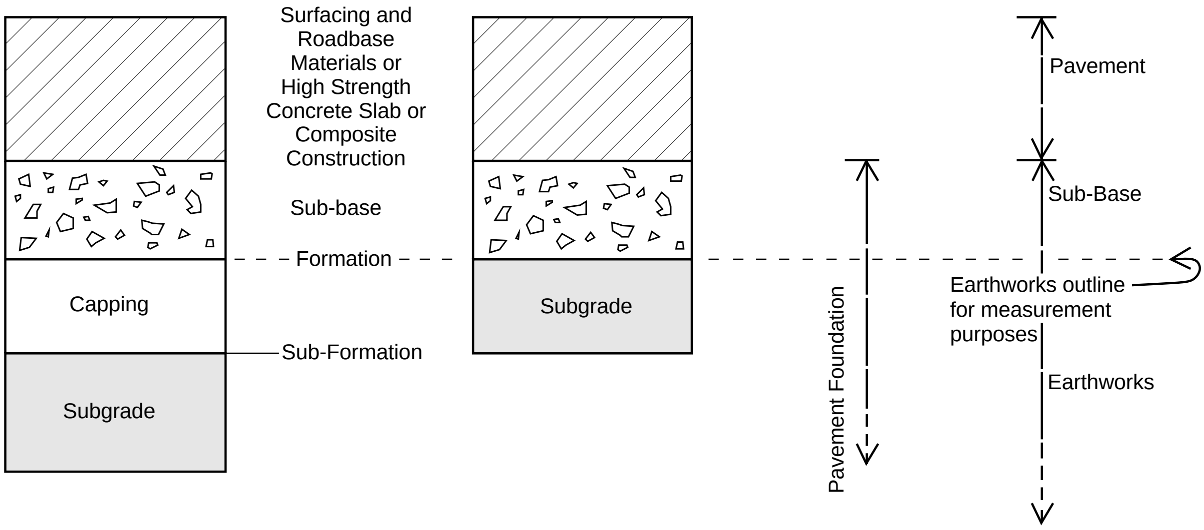


Figure 10/1 Composition below road surface

TABLE 10/1

Capping thickness for different subgrade strengths and stiffnesses

CBR (%)	CAPPING THICKNESS (mm)
CBR ≤ 2	600
2 < CBR ≤ 5	350
5 < CBR ≤ 15	150 for rigid and rigid composite. No capping for flexible and flexible composite but sub-base increased to 225
15 < CBR	No capping

10.9 Although not wholly satisfactory the CBR is currently used as a measure of subgrade strength and stiffness. BS 1377: Part 4 describes how CBR is measured in the laboratory. Field measurements may be carried out insitu but lower values are possible for soils coarser than fine sands due to the soil not being confined in a mould (see p191 of `The Design and Performance of Road Pavements' by Croney). Other indirect methods of arriving at a CBR may be used, such as the insitu plate bearing test. See Wood and Boud `Foundation Design Principles'. It is essential that laboratory CBRs reproduce as closely as possible the moisture content and the state of compaction likely to apply to the subgrade.

10.10 The CBR of a soil under particular conditions will vary with changes in moisture content. The moisture content will depend on the position of ground water table, stress history, overburden or surcharge load, availability of free water etc. In addition, the CBR will be different for a soil in its remoulded state and will depend upon the degree of remoulding. In practice most subgrade soils may be assumed to be remoulded due to earthmoving and compaction, trimming, preparation and compaction of formation or sub-formation. The moisture content of soils suitable for placing in embankments will invariably be different from that when preparing the formation or sub-formation for sub-base or capping and will almost certainly be different to the long term `equilibrium' moisture content attained under the completed pavement. The CBR of the material will vary with time as well and the `equilibrium' CBR is not necessarily the lowest value of CBR achieved under certain conditions. It would not be inconsistent for construction CBRs to be lower than long-term CBRs. The Designer should ensure that the CBR during construction is checked against his design values for construction and the long-term CBRs.

10.11 The long-term or equilibrium moisture content will depend on a number of factors including:-

i. soil type and properties;
ii. groundwater table;
iii. drainage;
iv. weather;
v. state of pavement maintenance;
vi. state of stress and stress history;
vii. construction history.

10.12 The construction history embraces amongst other things whether the material became very wet or very dry or alternated from wet to dry. There are procedures available (LR 1132) for estimating the equilibrium moisture content and, therefore, the equilibrium subgrade strength of cohesive soils. However, these can produce conservatively low values of CBR for saturated cohesive soils.

Capping Design

10.13 It is for the Designer to assess how much investigation and testing is required to produce his design. However, this should be tailored to the soil type and the road's use. Reference to Table 10/2, based on Table C1 of LR 1132, indicates that sands and sandy gravels present no problems as formations, whilst silts and cohesive soils will require consideration. Generally with these latter soils it is not until the CBR falls to around 2% that difficulties occur with placing, trafficking, compaction and embankment slope stability.

10.14 The estimation of the equilibrium CBR for clays is critical. Drainage conditions are discussed in Paragraphs 10.25 to 10.27; however, it may be assumed for the purposes of using Table 10/2 that the `high water table' condition is relevant. Silts are deemed to have a CBR of 2% or less and will therefore have the full thickness of capping given in Table 10/1. The Designer should assume that the SHW is complied with and, therefore, `average construction conditions' will pertain. The Designer must give his reasons if other than average conditions are assumed. The thinnest pavement proposed in HD 14/87 consists of a 150mm thick concrete surface slab. Including a 150mm thick sub-base and 350mm thick capping gives the least depth to subgrade of 650mm, for CBRs greater than 2 and less than or equal to 5 per cent. The thickest pavement given in HD 14/87 is a 450mm thick flexible composite pavement with an indeterminate life. A sub-base thickness of 150mm and a capping thickness of 350mm will give a depth to subgrade of 950mm for CBRs greater than 2 and less than or equal to 5 per cent. Interpolating between the depths given in Table 10/2 for the critical clay subgrades gives equilibrium CBRs for all but the heaviest clays of greater than 2 and less than or equal to 5 per cent and confirms the assumption of a 350mm thick capping. The heaviest clays are less affected by the thickness of construction and with a predicted equilibrium CBR of 2 per cent or less require a thicker capping of 600mm thickness. Table 10/2 may therefore be rewritten in the form given in Table 10/3.

TABLE 13/2
(Based on Table C1 of LR1132)
Equilibrium suction - index CBR values

TYPE OF SOIL	PLASTICITY INDEX	HIGH WATER TABLE						LOW WATER TABLE					
		CONSTRUCTION CONDITIONS:						CONSTRUCTION CONDITIONS:					
		POOR		AVERAGE		GOOD		POOR		AVERAGE		GOOD	
		Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick	Thin	Thick
HEAVY CLAY	70	1.5	2	2	2	2	2	1.5	2	2	2	2	2.5
	60	1.5	2	2	2	2	2.5	1.5	2	2	2	2	2.5
	50	1.5	2	2	2.5	2	2.5	2	2	2	2.5	2	2.5
	40	2	2.5	2.5	3	2.5	3	2.5	2.5	3	3	3	3.5
SILTY CLAY	30	2.5	3.5	3	4	3.5	5	3	3.5	4	4	4	6
SANDY CLAY	20	2.5	4	4	5	4.5	7	3	4	5	6	6	8
	10	1.5	3.5	3	6	3.5	7	2.5	4	4.5	7	6	>8
SILT*	-	1	1	1	1	2	2	1	1	2	2	2	2
SAND (POORLY GRADED)	-	← 20 →											
SAND (WELL GRADED)	-	← 40 →											
SANDY GRAVEL (WELL GRADED)	-	← 60 →											

* ESTIMATED ASSUMING SOME PROBABILITY OF MATERIAL SATURATING

NOTES:
1) A HIGH WATER TABLE IS 300mm BELOW FORMATION OR SUBFORMATION
2) A LOW WATER TABLE IS 1000mm BELOW FORMATION OR SUBFORMATION
3) A THICK LAYERED CONSTRUCTION IS A DEPTH TO SUBGRADE OF 1200mm
4) A THIN LAYERED CONSTRUCTION IS A DEPTH TO SUBGRADE OF 300mm

TABLE 10/3

Equilibrium subgrade strengths and stiffnesses, and capping thicknesses for typical construction conditions.

TYPE OF SOIL	PI	PREDICTED	
Heavy Clay	70	2	600mm capping
	60	2	
	50	2	
	40	2 to 3	350mm capping
Silty Clay	30	3 to 4	
	20	4 to 5	
Sandy Clay	10	4 to 5	

10.15 Further investigation will be required using Table 10/2 if average construction conditions and `high water tables' are not appropriate.

10.16 There are, therefore, three distinct assessments of the subgrade to be made.

i. Suitability, where appropriate, for construction.

ii. Prediction of CBR during construction - this will have some relationship with soil acceptability.

iii. Prediction of the long-term CBR at equilibrium mc.

10.17 The GI should normally be designed to obtain the full range of natural moisture contents of each relevant soil type. The relation between laboratory CBR and moisture content for at least six to ten points should then be determined for each soil type to provide a basis for suitability for construction through knowledge of the natural moisture content. This relationship also allows the CBR during construction to be estimated, bearing in mind the SHW requirements for protection of the subgrade, and that CBR is dependent on soil type and dry density. The CBR during construction may also be estimated using the plasticity characteristics as shown in LR 1132. Laboratory CBRs should be determined in accordance with BS 1377. Cohesive samples should be compacted as closely as possible to the density expected in the road, but to not less than 5% air voids to avoid spurious effects from pore water pressures. The static compaction method should be used for most soil types; the dynamic compaction method should only be used for sandy soils. The total annular surcharge weight should be calculated to equate to the pavement overburden pressure and the CBR calculated on the 2.5mm plunger penetration for cohesive soils.

10.18 Where possible insitu CBRs should be taken at the GI and compared to the relation between laboratory CBR and mc for the same soil type to ensure a correlation. In some circumstances insitu CBRs can give different values than laboratory CBRs. If there is a significant difference then corrections will have to be made when using the lab CBR/mc relation.

10.19 The assessment of the long-term CBR at equilibrium mc may be found by using one of the following predictive methods depending on the design assumptions made in the case of (i) (ii) and (iii), or the availability of a comparable site for (iv).

i. Estimation or calculation (Blood and Lord, and LR 889)
ii. Using LR 1132
iii. Using Table 10/3
iv. Prediction by testing (see Paragraph 10.20)

All these methods will require soil classification, PI etc, (see also Paragraphs 4.8 to 4.32).

10.20 Prediction by testing at the GI stage would take the form of locating an existing pavement, concrete slab or similar semi-impermeable construction over the soil type in question. This may take the form of an existing road, farm track, farm yard etc. As far as possible the history of the construction should be obtained and it should be at least 3 years old. The procedure is then to excavate through the construction and obtain insitu CBRs and moisture contents. This will give good guidance, depending on circumstances, as to the equilibrium CBR and mc to be expected. Because of edge drying effects tests should be made at least 2m from an unpaved or drained edge. If this is not possible tests should be carried out in the period November - April.

10.21 Soaked laboratory CBRs do not mirror the equilibrium situation or the stress history, but they do give an indication of worst conditions and will detect the presence of sulphates in the soil; this is important for soil that are to be stabilised.

10.22 If both the long-term CBR and the short-term (during construction) CBR are greater than 15% for rigid and rigid composite pavements, or greater than 5% for flexible and flexible composite pavements, then a capping is not required. If conditions during construction are likely to result in a lower CBR value than in the long-term, then the construction CBR should be used for design. However, normally, the CBR value for design will be the equilibrium value as this is usually the lower value of CBR.

10.23 In the absence of better methods of assessment using strength measurements it is suggested that the CBR for cohesive soils for both long and short term conditions is assessed using the plasticity properties of the soil. For the long-term condition Table C1 of LR 1132 may be used or the simplified form given in Table 10/3. `High water table' conditions should generally be used for design (see Paragraph 10.27).

Capping Materials

10.24 Capping materials described in SHW are designed to provide a stiffness and strength equivalent to a CBR of at least 15% at their top surface when placed and compacted to the thickness commensurate with the subgrade CBR given in Table 10/1. SHW Table 6/1 Class 6F1 and 6F2 are selected granular materials of fine and coarse grading respectively. Other materials which may be stabilised insitu to form a capping are described below in Paragraphs 10.28 to 10.67 on Lime Stabilisation and Paragraphs 10.68 to 10.80 on Cement Stabilisation. The Designer should allow the widest possible choice of capping materials to be used and should only restrict the choice if there are sound engineering reasons.

Drainage

10.25 It is strongly recommended that surface water is never collected by systems which might allow water to flow into the subgrade. It is preferable that surface and sub-surface water drains are kept separate. A minimum of 2½% crossfall should normally be provided at all locations. A narrow filter drain or fin drain should always be provided on the low side of pavements immediately adjacent to the pavement layers to collect and dispose of water seeping into the pavement. The pipe for such a system should always be set slightly lower than the sub-formation of a 600mm capping to enable it to act as a sump for percolation along the capping/subgrade interface. For thinner capping the pipe will be below sub-formation level anyway (see HCD and HA 39/89: Edge of Pavement Detail). Although the drainage will always extend below sub-formation level, it should be remembered that it is not a requirement of capping materials that they are free draining. If the capping is free draining, water percolating through it will be disposed of by the drainage, but the drainage will still have a beneficial effect if the capping is relatively impermeable. In cuttings where appreciable inflows of ground water are expected additional `cut-off' drains should be provided.

10.26 In moisture sensitive soils it is considered beneficial to install sub-surface drainage prior to placement of capping which for 600mm capping would result in drains deeper than the minimum specified in HCD. Measures should be taken to ensure these drains are not used as surface water drains during construction as they may become choked and unable to perform their long-term function.

10.27 For cuttings in cohesive soils of low permeability (all those in Table 10/3), it is unlikely that `cut-off' and sub-surface drains can be set in sufficiently deep to economically and efficiently lower the `water table' to ensure the `Low Water Table' conditions discussed in LR 1132 and denoted in Table 10/2. Because of this and the discussion in Paragraph 10.25 it is recommended that `High Water Table' conditions are designed for and drainage is provided accordingly. In embankments, the Designer should consider if lower `Water Table' conditions will be appropriate for the design, in which case CBRs should be interpolated from Table 10/2. For heavy clay a change in `Water Table' level does not change the CBRs given in Table 10/2. Changes do occur, however, for silty clay and sandy clay.

Lime Stabilisation

10.28 Lime and cement stabilisation are defined in Clause 601 of SHW. The second edition of the `Lime Stabilisation Manual' (referred to below as the `Manual') was jointly published by ICI and BLA in January 1990. Except in cases where it may conflict with this Advice Note, the SHW or other DTp documents, the Manual can be regarded as a guide to good practice and to the current `State of the Art', and reflects the liaison between the Department and the lime-producing industry. It should be noted that lime stabilisation as described in Clause 615 (SHW) depends on both the initial and long-term chemical reactions described in Section 3 of the `Manual'.

Materials and Chemical Reactions

Suitability of soil for stabilisation

10.29 The values of mc or MCV to be inserted in SHW Appendix 6/1 for Class 7E should be determined in the same way as for Class 2, general cohesive fill, except where it is desired to `modify' wet material by adding extra lime before the normal processing and compaction takes place. By using quicklime the moisture content can be reduced by the order of 5%. In this case the minimum and maximum mc or MCVs for Appendix 6/1 should be established by laboratory testing followed if necessary by site trials.

10.30 Organic matter interferes with the normal reaction between the lime and the soil. The maximum acceptability limit for Class 7E in Table 6/1 has been set at 2%. This is based on DTp experience and information from the literature. Oxford Clay is, however, an example of variation in this limit, since it contains up to 10% of finely divided bituminous matter and it can be successfully stabilised with 3% lime. It is possible, therefore, that some relaxation may be possible for some materials following experience and feedback.

10.31 At a high pH value, when lime is introduced, a reaction occurs between the clay fraction and the sulphates in the soil causing expansion and disruption. The value of total sulphate content to be inserted in Appendix 6/1 should normally be 1.0% except where, case histories or a prolonged trial indicate that a higher value is acceptable.

10.32 There are a few clay soils, otherwise complying with Class 7E, which may not develop the required strength after lime stabilisation. These include occurrences of Weald Clay and other silty clays in Kent and East Sussex and some Upper Coal Measure clays such as fireclays. In such cases preliminary laboratory tests are recommended.

Choice of Lime

10.33 The `Manual' discusses the advantages of quicklime (para 4.1 p 19). Semi-hydraulic quicklime, while having a low `available lime' content, behaves like a weak cement: if it is proposed to take advantage of its cementitious properties in the design, a trial would be required. In the case of quicklime a high reactivity is desirable (BS 6463: Part 3: 1987). This is measured in terms of the temperature rise when the quicklime reacts with water.

Chemical Reactions

10.34 The `Manual' (Section 3 pp15-18) refers to the drying out effect of lime but this is only strictly true in the case of quicklime. Hydrated lime addition makes the soil appear drier due to an increase in plastic limit but real moisture content reduction is only of the order of 0.5% for a 4% addition. This initial or short-term effect is referred to as `lime modification'.

10.35 The long-term pozzolanic reaction between the lime and the clay particles (referred to as `lime stabilisation') leads to a linear increase in strength with respect to the logarithm of time up to an age of several months. After this period the increase in strength may continue or become less marked, depending on the soil.

Lime content and strength

10.36 A bearing ratio of up to 8% can be expected immediately following final compaction, equivalent to an upper limit for MCV of 12. The addition of a minimum of 2½% available lime should result in the CBR rising to a minimum value of 15% in the short-term after compaction and it should not fall below a minimum value of 7-8% in the long-term. This allows for some degree of softening which may occur.

10.37 For clay soils amenable to lime stabilisation the long-term strength increases with lime content up to a lime percentage roughly equal to one tenth of the percentage clay content of the soil. However, above 4% lime content the increase in strength for each 1% increase in lime content becomes less marked.

Temperature

10.38 The strength developed due to long-term pozzolanic reactions increases with temperature. Thus for laboratory specimens cured for 56 days, the strength at 25 C is about 25% greater than the strength at 15 C. However, at 0 C little or no strength increase takes place. The use of lime as a construction expedient on waterlogged sites, relies only on the effect that lime has on plasticity in the short-term. This is not temperature sensitive and in fact this process is most likely to find application in winter months. This certainly is not the case for lime stabilisation as defined in the SHW where the long-term temperature sensitive effects are important, and hence the strict temperature requirements given in Clause 615 must be complied with.

Processing and Compaction

Spreading the Lime

10.39 In order to perform to the SHW, the `approved spreading machine' referred to in Clause 615.6 is likely to be one with a device capable of accurately metering the lime supplied, gearing to allow for variation in speed of travel, large capacity and spreading width, and provide precautions against dusting.

Rotovation and Mixing

10.40 Clause 615.10 SHW requires the stabilising machine to have an integral spray bar. This is to ensure that water can be added evenly at the first or second pass of the machine which is particularly important when quicklime is used. Machines which do not incorporate their own integral water supply are unlikely to be able to properly operate the integral spray bar specified in SHW.

10.41 It is important to maintain close site supervision to ensure that the depth processed and the final compacted thickness are correct, particularly for layers thicker than 250mm.

10.42 Whenever there is a delay between passes of the stabilising machine or before final compaction the surface must be sealed as described in the SHW Clause. This is to minimise evaporation loss, reduce carbonation of the lime and reduce rain damage.

Addition of Water

10.43 If the soil is too dry for slaking quicklime and/or mixing purposes, water should be added through the spray-bar during the first or second pass of the stabilising machine.

10.44 The MCV required for Class 9D immediately before compaction is such that 5% air voids, or better, will be achieved following compaction to Method 7. The maximum MCV to be inserted in SHW Appendix 6/1 should normally be 12. It has been asserted that for some soils (eg Oxford Clay) an MCV of 12 represents too wet a condition for satisfactory compaction. However recent experience suggests that compaction of Class 9D material in the dry condition represented by an MCV greater than 12 has led to inadequate compaction resulting in swelling and softening at a later stage when water is available. Hence, a maximum MCV of 12 should be specified unless there are strong grounds for choosing a higher value, in which case the demonstration area should be carefully assessed to ensure that compaction to 5% air voids has been achieved. The minimum value to be inserted in Appendix 6/1 may be 9 or other value as the Engineer considers appropriate. The minimum value should be based on the achievement of the desired bearing capacity within an appropriate curing period.

Mellowing

10.45 This is the interruption in processing referred to in Clause 615.11. It was originally based on American practice. Combined with the light compaction required for sealing, it allows the diffusion of lime in the presence of water and by progressive base exchange reaction breaks down the clay lumps to a more friable structure. This process also serves to correct any original unevenness in the distribution of the lime and ensures that quicklime is completely slaked so that there is no risk of expansion from this cause after compaction.

Compaction

10.46 Method 7 has been introduced as an amendment to SHW Table 6/4 for Classes 9B and 9D to produce 5% air voids at an MCV of 12. It is based on TRRL data and takes account of the fact that the tamping roller is much more effective on cohesive materials, including lime stabilised cohesive materials, than it is on granular materials. There is evidence that vibrating tamping rollers are effective but a site trial will be necessary before such a roller can be approved. The heavier pneumatic tyred rollers are unsuitable for the 150mm layer because there is evidence that serious overstressing of clay soils could occur under the high loads exerted. Care is needed in correctly assessing the weight of the heavier vibratory rollers, particularly when dealing with thick layers, thicker than 250mm.

10.47 Where the Engineer agrees that layers more than 250mm thick can be constructed, the Contractor is required to demonstrate that such an operation is feasible with the plant he proposes to use. In the demonstration area, therefore, the state of compaction at the bottom of the thicker layer must be shown to be adequate; normally an air void content not exceeding 5 per cent should be achieved in the bottom 150mm of the layer. To reach this level of compaction in layers of such thickness will require the soil-lime mixture to be prepared at a lower MCV (higher moisture content) than the maximum value compatible with thinner layers as specified in Appendix 6/1. The MCV of the material in the demonstration area should be determined, therefore, immediately prior to final compaction. If the construction of the proposed thicker layers is approved the soil-lime mixture in the subsequent construction work should be compacted at a MCV no greater than that of the demonstration area. For a 350mm thick layer, only heavy pneumatic rollers exceeding 6,000kg mass per wheel are usually considered to be suitable.

Environmental and Health Aspects

10.48 Safety and health are fully dealt with in Section 5.11 of the `Manual' (pp30-31).

10.49 Dust can be a problem when powdered lime is used. Lime which has been spread should be covered or mixed into the soil within 6 hours of application to prevent wind loss and carbonation (reversion to $CaCO_3$).

10.50 Dust produced when granulated quicklime was used at a site in Yorkshire in 1979 was investigated by TRRL. It was found that the wind distribution effects were mainly confined to an area close to the working area and that further away, the dust levels were similar to other construction sites during conventional earthmoving operations. In the measurements it was not possible to separate lime and soil dust.

Durability

Softening due to Soaking

10.51 Lime stabilised clays lose strength due to soaking, even after the long-term pozzolanic reactions have taken place, because there will always be particles or patches of unreacted clay in between the areas containing cementitious reaction products. The loss of strength is not easy to predict because it depends on a number of factors including:-

I. stage at which the soaking takes place;
ii. confining conditions;
iii. lime content;
iv. permeability.

10.52 Laboratory tests often show a complete loss of strength in unconfined compression when specimens stablised with less than 5% lime are soaked without lateral support at curing ages less than about 26 weeks. On the other hand, specimens soaked and tested in the CBR mould show a 30% to 50% reduction in CBR regardless of the curing age. Because of this uncertainty about the effect of soaking, it is important to ensure that lime stabilised cappings are kept free of water by appropriate design of drainage and the cross-section. Cases on schemes have occurred where lime stablised cappings have lost strength and poor drainage has been a factor. This also demonstrates that confining pressure is important. Proper attention must be shown to compaction and protection.

Frost Susceptibility

10.53 The test specified in Clause 602.19 SHW is the BS 812: Part 124 test which should be carried out on lime stabilised specimens of the materials proposed for the lime stabilisation during the ground investigation. Further tests should also be carried out on specimens cured for at least 28 days after mixing with lime.

10.54 Lime stabilised capping can be damaged either by the freezing of water entering the surface of the unprotected material before the pavement is constructed or by frost-heave when water is drawn in from below in freezing conditions. The BS 812 test will establish the susceptibility of the material to the latter type of damage.

10.55 Lime stabilised cappings should be covered during the period October to February by a weather protection at least 300mm thick if the overlaying pavement has not been constructed, unless the result of the BS 812 tests demonstrate that the stabilised material is non frost-susceptible. This should be established at the ground investigation stage so that if weather protection is required it can be included in the Contract. The sub-base is not sufficient as weather protection if it is less than 300mm thick.

10.56 The reaction of lime with clay soils initially produces a silt-like material which is more frost susceptible than the original clay. However, the pozzolanic reaction which also occurs leads to the progressive growth of cementitious compounds and a resulting reduction in permeability and increase in tensile strength, which may make the material non frost-susceptible after 28 days cure or longer depending on the percentage of lime added. But some stabilised materials may remain frost-susceptible even after considerable curing periods.

10.57 If the design indicates that the lime-stabilised capping will occur within the top 450mm below road surface (Clause 602.19) and the results of the BS 812 test show it to be frost susceptible at 28 days, then the option of lime stabilisation in the Contract should be restricted to the lower layer of capping or deleted altogether. Where there has been a reduction in frost susceptibility after 28 days cure but the results still fail the test, consideration should be given to carrying out further tests after 56 days cure.

Case Histories

10.58 A number of important case histories are given in the Proceedings of the Lime Stabilisation Symposia of 1988 and 1990, arranged by the British Aggregates Construction Materials Industries (BACMI) and the British Lime Association (BLA). These include early schemes, and the more recently completed M40 Sections which contained considerable lengths of lime stabilisation.

Rain Damage

10.59 Heavy rain falling immediately after stabilisation can cause leaching out of lime in the top 50mm of construction. The leached material should be removed before further construction and design modified if necessary.

Stabilisation with lime plus another additive

10.60 The following combinations have been used:-

i. lime plus cement;
ii. lime plus PFA;
iii. lime plus gypsum (Japan, Scandinavia);
iv. lime plus salt (Lees, Abdelkader and Hamdani, The Highway Engineer, December 1982).

Lime plus cement

10.61 This is a two-stage process which is quite well established. It is applicable to clay soils, which do not normally benefit from cement stabilisation. A common procedure is:-

i. process with 3% lime in a similar manner to Clause 615;

ii. 2-3 days interruption for mellowing,

iii. process with 4-7% cement in a similar manner to Clause 614.

10.62 The reaction with lime gives the clay a more granular texture so that it can be cement stabilised and the final product has a strength 50-80% greater than that obtained using either lime or cement alone.

Lime plus PFA

10.63 The lime and PFA react to form a cementitious material, which can be used in place of cement for stabilising granular soils.

10.64 If the Contractor wishes to produce a stabilised capping using any of the combinations given above he must demonstrate that the required strengths can be obtained by:-

i. pre-contract laboratory trial tests as described in the following Section on Testing;

ii. constructing a demonstration area as required in SHW Clause 613.

Testing

10.65 If preliminary laboratory trials are required, in order to determine the stabilisation method, they should be carried out by the Engineer or the Contractor as appropriate. The strength of laboratory compaction specimens can be determined either by the bearing ratio test of BS 1924 or by the BS 1924 unconfined compressive strength tests, unsoaked and soaked. In general, the bearing ratio is approximately equivalent to one twentieth of the compressive strength (in units of kN/m2) but a calibration should be performed for each type of soil. For laboratory mixing a mechanical mixer with a high speed blade will give the best results. It should be noted that the strength or bearing ratio of laboratory mixed material will be significantly greater than that of site-mixed material, and will need to be scaled down for design purposes, on the basis of `mixing efficiency' which is defined below.

$$\text{Mixing Efficiency} = \frac{\text{Strength of material mixed on-site and compacted in laboratory}}{\text{Strength of material mixed and compacted in laboratory.}}$$

If a trial is not conducted, the strength of the site mixed material is taken as 60% of the strength achieved in the laboratory.

10.66 The Engineer's tests on the demonstration area may comprise:-

i. BS 1924 laboratory tests for strength (as described above) on processed material obtained before compaction or on cores cut immediately after compaction;

ii. MCV to establish values to be met immediately prior to compaction, required for Appendix 6/1 and Table 6/1;

iii. in-situ CBR tests;

iv. dynamic or cone penetrometer tests;

v. BS 1924 tests to determine the dry density of the full thickness of the stablised layer, excavating a pit if necessary.

10.67 There is evidence that the lime stabilisation of clays with small percentages of lime can increase the voids ratio of the material, and therefore its permeability, after compaction. Care should be taken to avoid the softening of underlying untreated layers by percolation of surface water through the stabilised material.

Cement Stabilisation

Materials and Chemical Reactions

Suitability for stabilisation

10.68 The grading envelope for Class 6E material comprises essentially granular materials for cement stabilisation whereas Class 7F material has been introduced so as to include the silty sands, sandy silts and mixtures of sand, silt and clay, all capable of being stabilised with cement. High percentages of cement are required for the stabilisation of uniformly graded materials, either because the total surface area of the grains is large or because the cement is required to fill the voids. This may lead to uneconomic design and can cause difficulties when the material is compacted. Consequently a uniformity coefficient of greater than 5 is advisable for Class 6E material in addition to the grading limits given in SHW. Stabilisation of pulverised fuel ash Class 7G material, when not ruled out by a high sulphate content, has been an accepted procedure for many years, but may not always be economic. The values of mc or MCV for Class 7F to be inserted in Appendix 6/1 should be determined in the same way as for Class 1 and Class 2 (general cohesive fill). There is no sharp cut-off between Class 7F material for cement stabilisation and Class 7E material for lime stabilisation and some of these soils can be stabilised satisfactorily using either cement of lime. For the two-stage process employing both lime and cement see Paragraph 10.61.

Cement content and strength

10.69 The minimum bearing ratio to be inserted in Appendix 6/1 for Classes 9A, 9B and 9C will normally be 15% at 7 days. This is the strength required in the short term during construction when the capping has to act as a `platform'. In the long term a design bearing ratio of 7-8% will allow for possible softening effects. A cement content of 2% added to Class 6E material will normally give the required 7 day strength for strength for Class 9A material, but this should be checked on the demonstration area. Class 7F and 7G materials will in general require larger additions of cement to produce Class 9B and 9C cappings respectively. In nearly every case there should be a requirement in Appendix 6/7 for the construction and testing of a demonstration area, unless pre-Contract trials have already been carried out. Except for uniformly graded materials, the strength of a cement-stabilised material is directly proportional to cement content.

Variation of strength with temperature and time

10.70 For granular materials, strength increases with curing temperature in a linear manner similar to concrete, but there is a larger increase in strength with increase in curing temperature for materials containing some clay due to a pozzolanic reaction between the clay and the lime released by the cement during hydration. The normal laboratory curing temperature is 20 C. The strength of cement-stabilisation materials increases with increase in curing period and a linear relation is generally obtained up to an age of 1-2 months between the strength and the logarithm of the age the specimen. After 2 months the relation is no longer linear against the logarithm of time if the soil contains any clay.

Effects of other constituents

Organic matter

10.71 A safe upper limit is generally regarded as 2%. If there is any doubt, the total organic content test in BS 1924: Part 1 should be carried out. This detects the presence in soils of organic matter able to interfere with the hydration of Portland cement. Since this test was developed, however, changes in the composition of cements has caused difficulties in the interpretation of the test's results. Also the type of organic material, as well as the amount, is important as explained for Oxford Clay in Paragraph 10.30. Until a suitable alternative test is developed and until more information becomes available an organic contents, the above recommendations should continue to be used.

Total sulphate

10.72 For Class 6E materials a maximum total sulphate limit of 1% is given in the SHW. In the presence of lime and excess water, sulphates may react with clay minerals to form products that occupy a greater volume than the combined volume of the reactants. The result is expansion and cracking of the material. This reaction can only be avoided by the exclusion of water. This applies to some materials falling in Class 7F and for such materials the upper limit for total sulphate is 0.25% rather than the more usual value of 1% used for non-cohesive materials. For Class 7F material the Engineer should indicate whether the upper limit for total sulphate is 1% or 0.25%. See also `Lime-induced heave in sulfate-bearing clay soils' by D Hunter.

Processing and Compaction

Spreading the cement

10.73 The spreading machine likely to perform to the SHW is given in Paragraph 10.39.

Rotovation and water addition

10.74 It is important to maintain close site supervision to ensure that the depth processed and the final compacted thickness are correct, particularly for layers thicker than 250mm. The limiting values of moisture content to be inserted in Appendix 6/1 for Class 9A and Class 9B materials are those that will be obtained after pulverising and mixing immediately before compaction and should be chosen so that the permissible range is close to the optimum for the stabilisation mixture (as determined by BS 1924, 4.5kg rammer method or vibrating hammer method). The MCV requirement for Class 9B immediately before compaction is as described in Paragraphs 10.75 and 10.76 for Class 9D, and will override the moisture content requirements if there is a conflict. Similarly Class 9C which has an end product compaction specification will need to be compacted at a moisture content close to the optimum obtained in BS 1924 Part 2 (2.5kg rammer method).

Compaction

10.75 Where it is proposed to construct layers of Class 9A or 9B material with thicknesses greater than 250mm, the Contractor is required to demonstrate that such an operation is feasible with the plant he proposes to use. In the demonstration area, therefore, the state of compaction at the bottom of the thicker layer must be shown to be adequate; normally an air void content not exceeding 5 per cent should be achieved in layers of such thickness and this will require the soil-cement mixture to be prepared at a lower MCV (higher moisture content) than the maximum specified in Clause 614.10. The MCV of the material in the demonstration area should be determined, therefore, immediately prior to final compaction. If the construction of the proposed thicker layers is approved, the soil-cement mixture in the subsequent construction work should be compacted at a MCV no greater than that of the demonstration area.

10.76 If processed material is not compacted as soon as mixing is complete, some of the hardening effects of the cement will be lost and, in addition, extra compactive effort will be required to break down the cement bonds that have formed. These effects are additive and may lead to a serious reduction in strength. The rapidity of the MCV determination will assist in reducing the time scale between mixing and compaction to a minimum.

Testing

10.77 Preliminary laboratory trials, if required, may be carried out as described in Paragraphs 10.65 and 10.66. The type of laboratory mixer used will depend on whether the soil being stablised is granular or cohesive. It should be noted that the strength or bearing ratio of laboratory mixed material will be significantly greater than that of site mixed material and will need to be scaled down for design purposes, on the basis of `mixing efficiency'. If a trial is not conducted, the strength of the site-mixed material may be taken as 60% of the strength of laboratory mixed material.

10.78 Tests on the demonstration area may comprise those listed under Paragraphs 10.65 and 10.66.

Durability

Softening due to soaking

10.79 Some cement stabilised soils containing clay may lose strength due to soaking as described in Paragraphs 10.51 and 10.52. This can be checked in confined conditions by determining the bearing ratio, unsoaked and soaked, in accordance with BS 1924 and for unconfined cylindrical specimens by the BS 1924 test. A soaked test has the added benefit of determining those soils with a high sulphate content as the addition of water allows the expansive reaction, mentioned in Paragraph 10.72 take place in the laboratory.

Frost susceptibility

10.80 Although soil-cement CBM 1 for sub-bases and roadbases (SHW Clause 1035) is generally non-frost susceptible, the same is not always true for capping where the cement content is lower. In particular, stabilised PFA (Class 9C) is known to be frost susceptible. All cement stablised materials should be tested after 28 days curing using the procedures in Clause 602.19 SHW. Weather protection during the winter months, as described in Paragraphs 10.53 to 10.57, should be provided for doubtful Class 9B materials and all Class 9C materials.

11. SOIL STRUCTURES

Soil Reinforcement

11.1 The variety and range of application for soil reinforcement is almost unlimited. Therefore only those applications which apply directly to highways will be mentioned. Some of the materials used are described in Paragraphs 5.72 to 5.89.

Structures

11.2 Reinforced and anchored earth techniques are used in the construction of retaining walls and bridge abutments. The term `reinforced earth' describes a type of soil structure and does not refer to the product of any particular organisation. These types of soil structure comprise a mass of fill which is reinforced by tensile elements and retained by facings which are attached to those elements. Design criteria and other information is given in the Department's Technical Memorandum BE 3/78 (Revised 1987), Reinforced and Anchored Earth Retaining Walls and Bridge Abutments for Embankments.

11.3 Alternative retaining wall facings could include gabions (metal or geonet baskets), tyres fastened together with steel pins, or geotextile.

Embankments

11.4 Geosynthetics such as geotextiles, geowebs, geonets, geogrids etc may be used as reinforcement in embankments in order to:-

i. increase the embankment overall stiffness;

ii. allow the embankment side slopes to be steepened. The reinforcement layers are usually only placed in the outer layers of the embankment; see also Paragraphs 8.9 to 8.13;

iii. improve the internal stability of the embankment. The reinforcement layers will need to be placed across possible slip planes which could occur wholly within the embankment;

iv. improve overall stability of the embankment and subsoil. Where a slip plane could extend into the soil beneath the embankment, reinforcement would be required across the base;

v. improve foundation stability where the subsoil is weak enough to deform appreciably under the imposed load of the embankment. This should be achieved by the use of geocell mattresses laid under the embankment or stone columns in geogrid tubes under the embankment edges.

11.5 In applications where the polymer reinforcement is taken to the face of the embankment slope, it can be returned back into the fill either at the level of the next reinforcement layer above or at an intermediate level. This will stabilise the face of a steep sided slope susceptible to erosion.

11.6 It should be noted that the presence of a geogrid usually reduces the efficiency of the earthworks' compaction. There may also be practical problems if the geogrid is to be returned back into the fill; these include:-

i. the area involved;

ii. the need to trim the slope to shape after filling operations are complete;

iii. the geogrid being exposed to plant working on the slope during topsoiling etc;

iv. the geogrid being exposed to ultraviolet light until covered by topsoil;

v. deep services being more difficult to install, eg drainage.

Cuttings

11.7 The reinforcement may be used insitu or placed in a layered construction similar to embankments. Techniques for insitu reinforcement include soil nailing, and ground anchorages in which the reinforcing anchors are prestressed. BS 8081, Code of Practice for Ground Anchorages is referred to in the Notes for Guidance NG 624.2.

Reinforcement Properties

11.8 The principle requirements of reinforcing materials are strength, stability (low creep), durability, ease of handling, high coefficient of friction/adhesive with the fill, combined with low cost and high availability.

11.9 Since the design life of soil structures may be as long as 120 years, the durability of the reinforcement and its resistance to degradation is very important and must be considered by the Designer. Where metallic components are used a sacrificial thickness of the metal is usually necessary and a corrosion allowance for metallic components exposed to various environments is given in Table 4 of BE 3/78 (Revised 1987). These allowances should be increased by the appropriate amount in areas where the metallic reinforcement overlaps. Care should also be taken with galvanised reinforcements in cohesive soils as zinc is sensitive to illite and therefore may not fulfil its intended function leaving the metal unprotected (Earth Reinforcement and Soil Structures by Colin J P F Jones: Butterworths). Glass-fibre reinforced plastic (GRP) is not affected by electrolytic corrosion, although some loss of strength may occur when the material is kept in wet conditions over long periods of time. Geosynthetics appear unaffected by electrolytic corrosion or biological attack. Table 5/1 gives details of the resistance of the various polymers to degradation from various sources.

Fill to Structures and Retaining Walls

11.10 Detailed information and criteria on this subject may be obtained from Departmental Standard BD 30/87 - Backfilled Retaining Walls and Bridge Abutments. Bedding and backfilling requirements for buried concrete box type structures are given in BD 31/87.

Fill to Corrugated Steel Buried Structures

11.11 Where it is intended to construct corrugated steel buried structures in partial or total trench condition, tests must be carried out to measure the corrosivity of the insitu material as well as for the proposed imported bedding and surround material. If these tests have not been carried out by any reason, a provisional sum should be detailed in the Contract Bill of Quantities in case it is later found that the soil is aggressive. This provisional sum can be based either on sacrificial thickness of steel for the structure, or for the provision of additional surround material either side of the structure and above it. Detailed information is given in Departmental Standard BD 12/88. Advice on firms carrying out soil corrosivity tests may be obtained from Bridges Engineering Division, Department of Transport.

12. LANDSCAPING AND PLANTING

Planning

12.1 As with other parts of the scheme development liaison is very important to avoid abortive design work and last minute pre-contract additions. In this case the Project Manager, GE, GLE, Regional Landscape Architect and Horticultural Officer will be involved. Outlines of landscaping and planting will need to have been developed prior to the consultation stage as the necessary plans will be made available at any Public Exhibition that may be held.

12.2 The design of earthworks, environmental bunds and structures should not be carried out purely from an engineering viewpoint. The blending of these into their surroundings may be carried out at only small additional cost and makes the scheme more acceptable to the public and less environmentally intrusive. There are a number of ways in which the earthworks may be enhanced environmentally, many of which should be considered good engineering practice, including:-

i. slackening slopes at the top of deep cuttings;

ii. providing berms in deep cuttings in sympathy with the strata and local topography;

iii. rounding slopes and bottoms of embankments and cuttings;

iv. slackening cutting and embankment slopes to return to agriculture and placing the fenceline beside the road verge;

v. avoiding fencelines on skylines, tops of bunds etc;

vi. avoiding the use of imposing concrete retaining walls by the use of strengthened embankments, reinforced soil, gabions, etc;

vii. tree planting as a visual and noise barrier with the added benefit of stabilising slopes in certain circumstances.

12.3 Other factors requiring attention during the early design stages include:-

i. the possible difficulties of planting and topsoiling steep slopes;

ii. ensuring that sufficient topsoil of the local type is available;

iii. ensuring that sub-soils are suitable for planting and are not toxic or restrictive to plant life.

Topsoil

12.4 The relation between soils in the agricultural and the engineering sense is shown in Figure 12/1. For the purposes of this Advice Note, topsoil is the `A' and `B' horizons which support plant growth. For the purposes of this section of this Note, `sub-soil' is the weathered `C' horizon. The depth of topsoil and availability of sub-soil will need to be established during the GI. When assessing volumes of topsoil strip, it is usual to neglect volumes in wooded areas where disturbance from site clearance invariably renders any topsoil unsuitable. Where soft foundation soils exist underneath proposed embankments it is usual to leave the topsoil to avoid disturbing the desiccated crust.

12.5 Depending on the underlying sub-soil types, topsoils may be either acid, neutral or alkaline, all of which are capable of supporting different plant species. If the soil types on a scheme are very variable (eg acid or alkaline, or stoney or clayey) and the consequences of using variable topsoil types are serious then restrictions on where different types of topsoil are to be placed should be made in the Contract. This may be particularly relevant to imported topsoil and to accommodation works affecting farms and market gardens etc. It is usual for the earthworks drawings to show the extent and depths of topsoiling. The drawings will also show the treatment type in accordance with SHW Clause 618.4. Other details on the treatment of topsoil should be set out in SHW Appendix 6/8. Further information on topsoil may be found in BS 3882, Recommendations and Classification for topsoil.

12.6 The minimum depth of topsoil for planting, other than grassing and seeding, is generally 300mm. Topsoil of this thickness tends to be unstable in wet weather prior to good root growth on slopes greater than 1 in 2, and thicknesses of 150mm likewise are difficult to place and maintain on slopes steeper than 1 in 1.5. The SHW requires only cutting slopes to be harrowed prior to topsoiling. The exception to this will be for

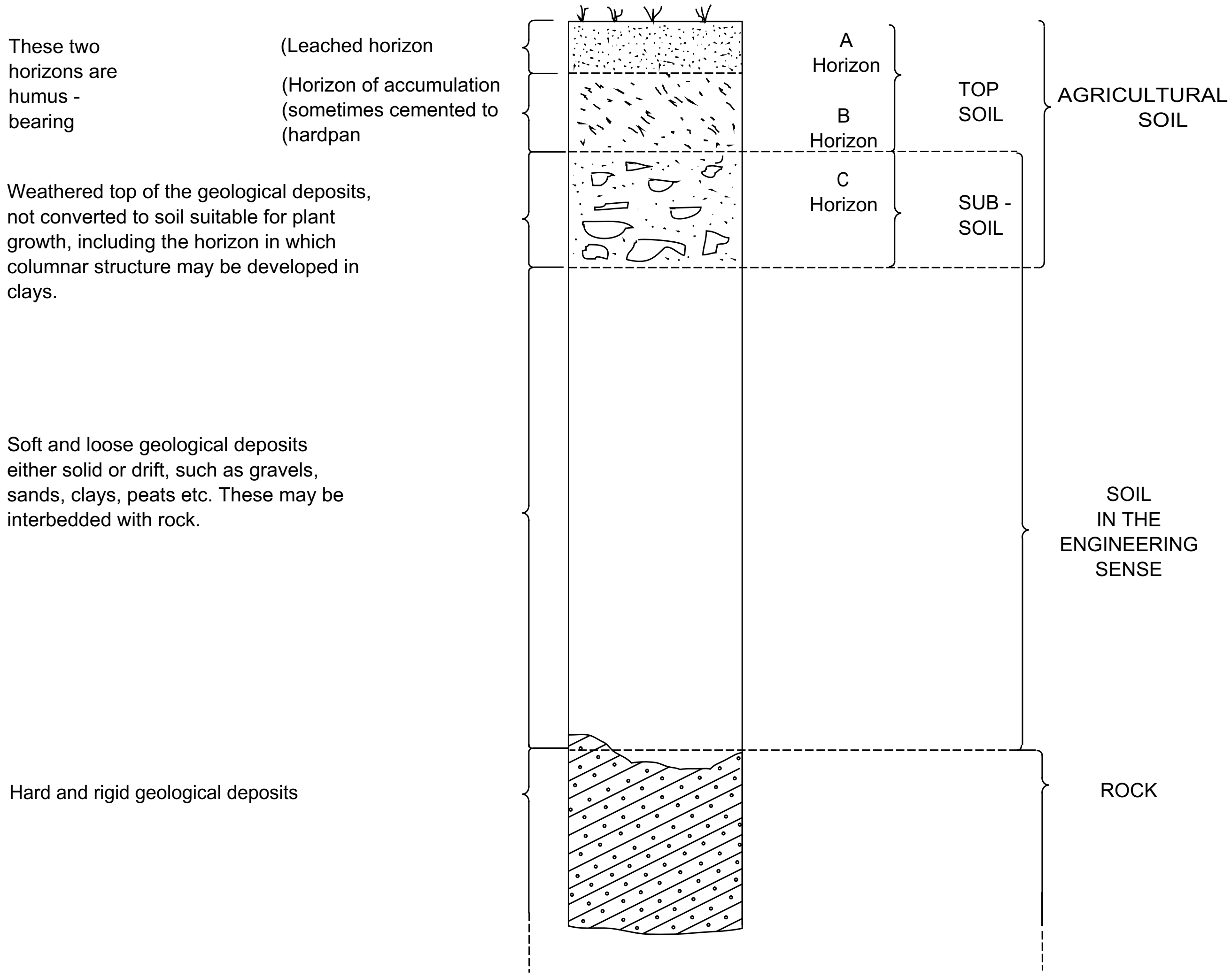


Fig 12/1 The relation between agricultural and engineering soil cuttings in rock or hard materials where consideration will be required to maintaining the stability of the topsoil. Where harrowing is not required in cuttings this should be noted in SHW Appendix 6/8.

12.7 The requirements of SHW Clause 618.3 only apply when the sub-soil is a heavy clay. Where this occurs then the extent of the restriction will need to be set out in SHW Appendix 6/8. The rainfall figure quoted in SHW Clause 618.3 is satisfactory for most soils likely to be encountered in this country.

Sub-soils

12.8 Most plants require water-retentive soils and have the ability to send down tap roots (anchor roots). Growing plants on areas of rock, rock fill and granular materials may therefore prove difficult and so if planting is required a topping layer of a more acceptable sub-soil is necessary. The sub-soil used will need to be cohesive in order to retain nutrients and moisture, and, for most planting requirements, the depth, including topsoil, will need to be at least 1m.

12.9 A depth of topsoil of 300mm is normally required for planting, although pits for individual trees and shrubs may be used. The interface between a clay or marl and the topsoil should be broken up to prevent topsoil slip and promote root penetration. Some clays can be very hard and these may require more disturbance to allow root penetration. If the soil around pits is not sufficiently broken up, tree roots will be restricted to the original pit resulting in sub-standard growth.

Waste Materials and Industrial By-products

12.10 There is increasing pressure for available waste materials and industrial by-products to be used in highway construction. Possible sources of these materials, either occurring on site or as possible import, should be identified. These materials will effect different plant species in different ways, as do naturally occurring soils. They can be toxic to plant life and special precautions may need to be taken.

12.11 The following comments on waste, and other materials which may be encountered, should be noted:-

12.12 China Clay waste is the waste quartzitic sand from the production of China Clay and is suitable, both chemically and physically, for planting using sufficient topsoil and nutrients. However, because of its granular nature it is liable to erosion and lacks water and nutrient retention characteristics.

12.13 PFA is described in Paragraphs 5.58 to 5.66. Pozzolanic action can form a hard-pan. It may have a high boron content and high pH. It is generally acceptable if covered with 1m of sub-soil and topsoil.

12.14 Minestone is described in Paragraphs 5.27 to 5.30. This material may contain trace elements, heavy metals, sulphates and sulphides. Fresh colliery waste may well be neutral or alkaline (pH 7.0 or higher). However on exposure to air and moisture, sulphuric acid is produced and the pH falls. The rate of production of acid is very variable and will, to a large extent, depend on the constituents of the waste material. The leaching out of sulphuric acid into streams and waterways can have a devastating effect on plant and animal life. Depending on the likely severity of the problem a topping of 1m sub-soil may be required. Consideration may be given to biotechnical methods of restoration, although these are not yet fully established. Biotechnical methods include; symbiotic association, where the association of a fungus around the roots of trees acts as a sheath and protects the roots from aggressive agents; and pelletized refuse, where incinerated refuse pellets which achieve high pH are used to neutralize the acids produced in the fill. Heavy metals are likely to be present in all mining waste materials.

12.15 Foundry Sand may have a low pH and contain heavy metals resulting from the casting process. Consequently, a topping of sub-soil may be required.

12.16 Blastfurnace Slag is usually a basic limestone flux and unlikely to be a problem except in environmentally sensitive areas particularly near to watercourses where leached sulphates can cause problems.

12.17 Useful references are given in Chapter 14.

Landscape Areas

12.18 The presence of a landscape area may be beneficial to the earthworks balance by providing a reserve of fill material if there is a shortfall, or a suitable on-site location for placing excess material.

Therefore, the design contours should, if possible, leave room for movement and only the general shape our outline of the area provided on the drawings. However, the basic functions of the landscape area such as screening, protection etc should never be compromised by removal of too much material. If a particular feature was shown at, or required by, the Public Inquiry then this feature must remain unchanged.

12.19 The shape of the landscape area should be such as to adequately discourage squatters and include ditches across potential access routes. Surface water run-off should also be considered and drainage provided as appropriate to protect adjacent roads or properties from flooding.

12.20 All that is required in Clause 620 SHW for compaction purposes, unless there is a specific requirement such as a picnic area, is that the compaction shall be sufficient to remove large voids and produce a coherent mass. The act of placing and moulding materials into the desired landscape shape using tracked plant usually provides sufficient compaction for the landscape area's intended use. Where particular requirements are needed for the construction of landscape areas according to SHW Clause 620, they should be included in SHW Appendix 6/9.

Environmental Bunds

12.21 Earthworks for environmental bunds are covered by SHW Clause 619 with particular requirements being set out in SHW Appendix 6/3. The Appendix will need to set out the location, particular materials, construction and compaction requirements and topsoiling and seeding or turfing requirements.

12.22 The usual practice is to site environmental bunds as close as possible to the noise and visual source in order to make them as effective as possible: they can be placed close to the area to be protected, but this is generally an inferior solution. In general, environmental bunds constructed near the source are only effective in ameliorating problems within a distance of about 330m of the carriageway edge. A common solution is to build a steep sided bund close to the carriageway using soil reinforcing or strengthening techniques. In many cases fences will have to be placed on top of the bunds and therefore sufficient room for construction and stability of the fence should be provided. There may be a conflict between locating a fence on top of a bund to reduce noise, and the visualimpact of such a fence (Paragraph 12.2(v)). The Designer will then have to choose the best course of action for the scheme.

12.23 The best form of noise protection is to maintain the road in cuttings. A minimum depth of 2m should be adequate although the depth should increase according to the distance to the nearest buildings. The Departmental of Transport Manual of Environmental Appraisal gives further advice.

Planting

12.24 Planting is mostly labour intensive and very expensive. Not all trees and shrubs grow successfully in the first round of planting so a limited number of failures is to be expected. The number of failures can be reduced by paying attention to the earthworks details set out below:-

i. sufficient pre-planning and design should be provided to ensure stable side slopes, topsoil and sub-soil;

ii. the depth of topsoil and sub-soil should be adequate to maintain plant growth and allow rooting for the species proposed; protective measures against the effects of deleterious waste and refuse may be necessary.

iii. the correct topsoil and sub-soil types should be used which are appropriate to the proposed area and species;

iv. low-lying areas should be adequately drained;

v. excessive compaction should be avoided.

12.25 Whilst only appropriate species for the area concerned should be planted, other considerations should include avoiding spread into sight lines or neighbouring property, and avoiding planting trees which may be toxic to grazing animals, such as yew, where there is a risk of encroachment.

12.26 A number of factors affecting plant growth have been identified. When dealing with chemical content of the soil or toxicity it is difficult to obtain objective advice. If imported materials such as PFA or Minestone are likely then restrictions should be identified in the contract documents (SHW Appendix 6/3). This will normally take the form of providing a topping material of sufficient thickness and with sufficient nutrients to encourage plant growth. However, the notes below may be used as a guide and where relevant materials are encountered appropriate tests should be carried out at the GI stage if possible.

i. Acidity is critical: a pH of 5 or below will be too acid for most plants;

ii. Traces of heavy metals should not be greater than 1000ppm and traces of aluminium, zinc and copper combined should be less than 20ppm. (Broadshaw and Chadwick, `The Restoration of Land').

iii. The percentage of CaC03 or acid neutralisation coefficient, should be greater than the percentage of pyrites.

iv. Extractable boron should be less than 20ppm.

v. Materials with pozzolanic action require a topping to allow plant growth.

12.27 The judicious use of planting will not only assist in erosion control but can have positive benefits by reducing the long-term maintenance costs caused by shallow failures. See Barker DH 1986 `Enhancement of Slope Stability by Vegetation' and Greenway DR 1987 `Vegetation and Slope Stability'. The following points will encourage healthy, stable plants thus improving the stability of earthworks.

i. Do not plant where root wedging may disrupt soils and rocks leading to instability or water ingress.

ii. Current horticultural/forestry practice is to have tree stakes one-third of the height of the tree with one tie to encourage early bole development.

iii. All tree ties to be removed as soon as possible (3 years maximum) to ensure a stable plant.

12.28 A wide range of applications for vegetation in Civil Engineering and methods of cultivation of plants and grasses are given in Coppin and Richards (1990).

Watercourses

12.29 Rivers, streams and watercourses are important ecological corridors and when cleaning out, regrading or altering the alignment is required, positive measures towards conservation should be taken. The Landscape Architect can provide advice. The concrete lining of watercourses and ditches is often very much out of sympathy with the landscape and surroundings and so the need for lining ditches and the materials used should be carefully considered. Geosynthetics can be particularly good at controlling erosion of river banks without being obtrusive.

12.30 Where rivers and streams are to be regraded, careful attention should be paid to the profile adopted, both in plan and in cross section, to ensure that they are in keeping with the nature of the original watercourse.

12.31 Useful advice is given in the Nature Conservancy Council publication on River Engineering. The excessive use of sand-bags, riprap and gabions to line watercourses should be avoided. There are currently many materials and techniques available for protecting river banks from erosion which allow a quick covering of plant life to develop.

12.32 When regrading of a watercourse or cleaning out is required, attention to detail in the contract documentation may avoid the wholesale destruction of a natural habitat and still serve the required engineering purpose. It is preferable to coppice rather than pull out shrubs and trees. It will also help to work from the side of the watercourse which will be the least spoilt during the regrading or cleaning out. The drawings will need to clearly define the extent of work required.

12.33 Treatment and construction of new and existing watercourses is covered by SHW Clause 606 and special requirements should be set out in SHW Appendix 6/3.

Turfing and Grass Seed Mixtures

12.34 The grass seed mixture and rate of sowing to be used are set out in SHW Clause 618. If alternative mixes are considered appropriate to suit local conditions, approval must be obtained from the Horticultural Officer. Amendments are to be included in SHW Appendix 6/8. SHW Appendix 6/8, and the drawings, should be used to show the areas of seeding and turfing and also the measures to be used to secure turfing on slopes where appropriate. The mowing plant and mowing requirements shall be set out in SHW Appendix 6/8.

12.35 Propagation of grass on steep slopes, subject to erosion, can be improved by the use of geosynthetic sheets or jute. These materials provide a stable face for grass to become established and hence make the slope more stable.

12.36 The optimum time for sowing grass seed is late August or early September; April or May are almost as suitable. However priority must be given to establishing some grass growth whatever the season so as to prevent erosion of susceptible soils.

12.37 Weed control is important to avoid difficulties with adjacent landowners. A number of weeds including ragwort and wild oats are notifiable and some, particularly ragwort, are extremely poisonous to grazing stock, especially when cut down. The Weeds Act 1959 should be consulted and complied with.

12.38 Turves are extremely useful where immediate surface stability is required or difficulties are encountered maintaining topsoil whilst growth from seed takes place. If a particular seed mix is required and not available in turf form the turves may be laid up-side-down and the root side sown with the appropriate mixture. There are a number of methods of fixing turves involving pegging individual turves or using netting, pegged over a number of turves. Netting should be of a type that degrades after two years.

12.39 In some cases, wildflower swards may be more in keeping with the surroundings than grass. Further advice on this subject can be obtained from the Horticultural Officer.

13. USE OF COMPUTERS IN DESIGN

13.1 A number of computer programs are available for earthworks design and soil-structure interaction, some of which were developed by the Department of Transport. There are programs available for use in slope stability, consolidation settlement and finite element analysis. Advice on selection and use of programs should be sought in the first instance from Highways Engineering Division. For analysis of pile groups, reference should be made to Departmental of Transport Departmental Advice Note BA 25/88 `Piled foundations'. Finite element programs are particularly useful for analyzing problems involving soil-structure interaction and also stability of slopes where deformations are required.

14. REFERENCES

For ease of use the references have been listed under the Chapters referred to in the text.

2. Ground Investigations

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(b) Notes for Guidance on the Specification for Highway Works, as detailed in Paragraph 1.8.

(c) Method of Measurement for Highway Works, as detailed in Paragraph 1.9.

(d) Highway Construction Details as detailed in Paragraph 1.10.

Since these documents are referred to frequently throughout the document, they are listed only once to save repetition.

(e) DTp. Specification for Road and Bridge Works. 5th Edition. HMSO. 1976. (SRBW).

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13. Use of Computers in Design

(a) DTp. BA 25/88. Piled Foundations.

15. ENQUIRIES

All technical enquiries or comments on this Advice Note should be sent in writing to:

Head of Division Road Engineering and Environmental Division The Highways Agency St Christopher House Southwark Street London SE1 OTE	N ORGAN Head of Road Engineering and Environmental Division

The Deputy Chief Engineer The Scottish Office Industry Department Roads Directorate New St Andrew's House Edinburgh EH1 3TA	N B MACKENZIE Deputy Chief Engineer

Head of Roads Engineering (Construction) Division Welsh Office Y Swyddfa Gymreig Government Buildings Ty Glas Road Llanishen Cardiff CF4 5PL	B H HAWKER Head of Roads Engineering (Construction) Division

Assistant Chief Engineer (Works) Department of the Environment for Northern Ireland Roads Service Headquarters Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	D O'HAGAN Assistant Chief Engineer (Works)

A** HERNCASTER BYPASS

APPENDIX 6/1
CLASSIFICATION AND ACCEPTABILITY CRITERIA

CLASS	GENERAL MATERIAL DESCRIPTION	TYPICAL USE	PERMITTED CONSTITUENTS	MATERIAL PROPERTIES				COMPACTION REQUIREMENTS
				PROPERTY	TEST	LIMITS		
						LOWER	UPPER	
1B	Uniformly Graded	General Fill Drawings Nos 305/6/1B-5	Any materials other than Class 3	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 3
				uniformity coeff	ratio of D_{60} to D_{10}	-	10	
				optimum mc	BS 1377: Part 4 (2.5kg rammer method)	-	-	
				mc	BS 1377: Part 2	Opt mc - 2%	Opt mc + 2%	
1C	Coarse Granular Material	General Fill Drawing Nos 305/6/1B-5	Any materials other than Class 3	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 5
				uniformity coeff	ratio of D_{60} to D_{10}	5	-	
				10% fines value	Clause 635	50 kN	-	
2B	Dry Cohesive Material	General Fill Drawing Nos 305/6/1B-5	Any materials other than Class 3	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 2
				MCV	Clause 632	13	16	
2D	Silty Cohesive	General Fill Drawing Nos 305/6/1B-5	Any materials other than Class 3	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 3
				MCV	Clause 632	8	15	
4	Various	Landscape Area Fill	Any materials	grading	BS 1377: Part 2 Passing 500 mm Passing 63 mm	 - 10%	 100% 100%	Clause 620.2
				MCV	Clause 632	6	18	

A** HERNCASTER BYPASS

APPENDIX 6/1 (Cont'd)
CLASSIFICATION AND ACCEPTABILITY CRITERIA

CLASS	GENERAL MATERIAL DESCRIPTION	TYPICAL USE	PERMITTED CONSTITUENTS	MATERIAL PROPERTIES				COMPACTION REQUIREMENTS
				PROPERTY	TEST	LIMITS		
						LOWER	UPPER	
6J	Selected Uniformly Graded Granular	Fill to Reinforced Earth Drawing No 305/10/1 305/10/2B	See Table 6/1	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	Tab 6/4 Method 3
				uniformity	ratio of D to D_{10}	$_{60}$ 5	10	
				mc	BS 1377: Part 2	17%	21%	
				effective c' effective ø'	Clause 632	50 kN/m 25°	^{2} - -	
				coeff of friction adhesion	Clause 639	0.6 50 kN/m^2	-	
6P	Selected Uniformly Graded Granular	Fill to Structures Drawing Nos: 305/9/1/5 305/9/2/4 305/9/3/6 305/9/4/5&6 305/9/6/4	See Table 6/1	grading	BS 1377: Part 2	Tab 6/2	Tab 6/2	95% maximum dry density of BS 1377: Part 4 (Vibrating Hammer Method)
				uniformity coeff	ratio of D_{60} to D_{10}	5	-	
				10% fines value	Clause 635	30 kN	-	
				undrained c shear ø	Clause 633	50 kN/m 20°	^{2} - -	
				effective c' shear ø'	Clause 636	40 kN/m 25°	^{2} -	
				coefficient of permeability	Clause 640	5 x 10 m/sec	^{4} - -	
				mc	BS 1377: Part 2	17%	21%	

CLASS	GENERAL MATERIAL DESCRIPTION	TYPICAL USE	PERMITTED CONSTITUENTS	MATERIAL PROPERTIES: PROPERTY	MATERIAL PROPERTIES: TEST	LIMITS: LOWER	LIMITS: UPPER	COMPACTION REQUIREMENTS
7F	Selected Silty Cohesive Material	For Stabilisation With Cement To Form A Capping (Class 9B)	Any material, or combination of materials, other than chalk, unburnt colliery spoil and argillaceous rock	grading	BS 1377: Part 2	Tab	Tab	Not applicable
				uniformity coefficient	ratio of D_{60} to D_{10}	5	-	
				MCV	Clause 632	8	15	
				Liquid Limit	BS 1377: Part 2	-	45	
				Plasticity Index	BS 1377: Part 2	-	20	
				organic	BS 1377: Part 3	-	2%	
				total	BS 1377: Part 3	-	1%	
7G	Selected Conditioned Pulverised Fuel Ash. Cohesive Material	For Stabilisation With Cement To Form A Capping (Class 9C)	Conditioned material direct from power station dust-collection system and to which a controlled quantity of water has been added	mc	BS 1377: Part 2	15%	35%	Not applicable
				total sulphate content	BS 1377: Part 3	-	1%	
9B	Cement Stabised Silty Cohesive Material	Capping	Class 7F with addition of cement according to Clause 614	pulverisation	BS 1924: Part 2	30%	-	Tab 6/4 Method 7
				MCV immediately before compaciton	Clause 632	8	12	
				bearing ratio	BS 1924: Part 2	15%	-	
9C	Cement Stabilised Conditioned Pulverised Fuel Ash. Cohesive Material	Capping	Class 7G with addition of cement according to Clause 614	pulverisation	BS 1924: Part 2	60%	-	End Product 95% of maximum dry denisty of BS 1924 (2.5 kg rammer method)
				bearing ratio	BS 1924: Part 2	15%	-	
				mc	BS 1924: Part 1	To enable compaction to Clause 612		
9D	Lime Stabilised Cohesive Material	Capping	Class 7E with addition of lime according to Clause 615	pulverisation	BS 1924: Part 2	30%	-	Tab 6/4 Method 7
				MCV immediately before	Clause 632	8	12	
				bearing ratio	BS 1924: Part 2	15%	-	

APPENDIX 6/1 (Cont'd)

Classification

The classification and confirmation of acceptability of the earthworks materials shall be carried out by the Contractor at the point of excavation for on-site materials, and at the point of deposition for imported materials. Trial pit locations for classification purposes shall be agreed with the Engineer in advance. If in the opinion of the Engineer the material has altered its classification or become unacceptable for whatever reason, he may require the Contractor to repeat the classification and acceptability tests given in Table 6/1 and this Appendix. The rate of testing required shall be as stated in Additional Clause 656 AR, Appendix 0/1.

The Contractor shall submit two copies of all test results to the Engineer within one working day of the completion of the test. The copies shall be signed by the Contractor's responsible engineer/ technician.

Class 3 Material

No material is designated as Class 3.

Ground Water Lowering - Cut No 3 (sub-Clause 602.17)

After stripping topsoil, but before commencing any bulk excavation between Ch 3570 and Ch 4090 in cut No 3, the Contractor shall construct a deep temporary cut-off filter drain in the position and to the invert levels as shown on Drawing No 305/5/17A. This drain shall be maintained to allow ground water to outfall freely into the stream at Ch 3570 (South side) until the permanent road drainage through Cut No 3 is completed. A full description of these works is given in Additional Clause 652 AR, Appendix 0/1.

Permeability Test (Clause 640)

The permeability test referred to in Clause 640 shall be the constant head or constant hydraulic gradient permeability test as described in BS 1377: Part 5 and Part 6 for vertical permeability. The permeability box test described in HA 41/90 is to be used for horizontal permeability.

APPENDIX 6/3

EARTHWORKS REQUIREMENTS

Drawings

305/6/1B
/2
/3A
/4
/5

General earthworks drawings showing quantities, cut and fill numbering and chainages, selected fill locations, location of unacceptable material, simplified GI borehole information and soil strata.

Blasting

Blasting is a permitted alternative within the confines of Cut No 4 (Ch 5200 to Ch 6500) for the excavation of the Lower Lincolnshire Limestone only.

Pre-split blasting shall also be employed if the limestone is to be excavated by bulk blasting. Details of the diameter, dip, azimuth, spacing and depth of the pre-split drill holes together with the minimum panel size and face lift are given in Additional Clause 643 AR, Appendix 0/1. Other specification requirements for pre-split blasting including the trial are given in Additional Clause 644 AR, Appendix 0/1.

(Note: For details of pre-split blasting procedures, specifications etc see TRRL LR 1094; pre-split blasting for highway rock excavation, and also TRRL SR 817; A device for measuring drill rod and drill hole orientations).

Blasting of any kind shall only take place between the hours of 0930-1130 and 1430-1630 Mondays to Fridays, and 1000-1200 Saturdays. No blasting shall be carried out on Sundays or Bank Holidays.

Cutting Faces

In Cut No 2 (Ch 2900-3340) South side, the verge drains within the silty cohesive material shall be excavated in such a manner that only 20 m of drain trench over 1 m deep may be open at any one time.

Excavation of further lengths of trench may not commence until a sufficient length of any existing trench has been backfilled and fully compacted so that the 20 m length of open trench is not exceeded.

All cut-off drains and ditches alongside cuttings and embankments shall be completed and outfalls provided prior to the commencement of any adjacent earthworks excavation or filling operations.

The faces of cuttings requiring attention prior to topsoiling shall be treated in accordance with sub-Clause 603.7(i)(a), with the exception of Cut No 4 where the cut faces in limestone shall not be topsoiled but treated in accordance with sub-Clause 603.6(i).

Watercourses

The existing stream at Ch 2010-2040 North side shall be realigned and regraded to the line and level as shown on Drawing No: 305/5/6. Concrete lining, 150 mm thick, as shown on Drawing No: HBS/5/17B shall be placed on the invert and that side slope adjacent to the embankment from Ch 2005-2050. The redundant stream bed shall be treated in accordance with sub-Clause 606.4 and backfilled with Class 1B material. Drainage pipes along the redundant stream bed shall be as detailed in Drawing No: 305/5/6.

Embankment Construction

The Contractor shall not allow fills of more than 2 m height to remain at side slopes of 1:2.5 or steeper for more than 48 hours before trimming back to the design slope, and in any event the side slope shall not be steeper than 1:2 at any stage of construction.

No surcharging of embankments is required, and the Contractor shall not stockpile material on fill areas to a height greater than that of the finished embankment. See also Additional Clause 649 AR, Appendix 0/1.

The minimum thickness of capping material when used as a weather protection layer shall be 450 mm and the minimum thickness of sub-base when used as a weather protection layer shall be 150 mm. See sub-Clause 608.7.

Compaction

The additional compaction for the top 600 mm below sub-formation is not required for the farm access tracks at Ch 8052 and Ch 4990. Elsewhere sub-Clause 612.10(ii) shall apply.

Nuclear Density/Moisture gauges may be used for the measurement of field dry density. In Classes 6P and 9C materials, BS 1377: Part 4 (vibrating hammer method) and BS 1924: Part 2 (2.5 kg rammer method) respectively shall also be used as a basis for compliance.

APPENDIX 6/5

GEOTEXTILE SEPARATORS

Drawings

305/6/1B
/6

Location

A geotextile shall be used under the fill material for the full width of Bank No 1. It shall be laid directly on top of the existing ground surface after topsoil has been stripped between Ch 1500 and Ch 1800. The geotextile shall be manufactured from synthetic fibres and shall have a life expectancy of 40 years.

Design Criteria (sub-Clause 609.4)

The geotextile shall sustain a tensile load of not less than 10 kN/m and have a minimum axial strain of 20% at failure. It shall also, have a minimum water flow rate through it at right angles to its principal plane, in either direction, of 50 litres/m2/S. See Substitute Clause 659 SR, Appendix 0/1.

Sampling

Samples for testing in accordance with Clause 609 shall be taken at the rate of 1 set of samples per 400 m2. A set of samples shall consist of that minimum number of test pieces sufficient to carry out all the tests required in Clause 609.

Installation

The geotextile shall be laid from rolls in a longitudinal direction along the line of the bypass, and jointing shall be by lapping only. Physical jointing is not permitted. The lap width shall be 500 mm minimum at any location. See Substitute Clause 659 SR, Appendix 0/1.

APPENDIX 6/6

FILL TO STRUCTURES

Drawings

305/9/1/5	Bridge No 1 - Fill to Abutments	
/2/4A	No 2 - Fill to Abutments and Piers	
/3/6	No 3 - Fill to Abutments and Piers	
/4/6B	No 4 - Fill to Abutments	
/5/5	No 5 - Fill to Abutments	
/5/6	No 5 - Fill to Piers	

Location and extent of selected granular fill Class 6P material.

The Contractor is required to show that his proposed material is stable when compacted and trimmed to a slope of 1 vertical to 1.5 horizontal as described in sub-Clause 610.6.

APPENDIX 6/7

SUB-FORMATION AND CAPPING

Drawings

305/7/2	305/6/3A	HBS/7/1
/3B	/4	to
/4	/5	HBS/7/12
/5B		

Pavement drawings, earthworks drawings and standard detail drawings showing the extent, locations, widths and thickness of Capping materials.

Sub-formation shall have the same shaping requirement as formation as shown in Drawing Nos HBS/7/1 to HBS/7/12.

Where formation is formed in the Lower Lincolnshire Limestone in Cut No 4 between Ch 5200 and Ch 6500 the material shall, in accordance with sub-Clause 616.4, be either:

1) excavated to a depth of 500 mm and the material crushed to give a maximum particle size of 500 mm or

2) where the surface is tabular, it shall be regulated where necessary with a cement bound material Class CBM2 as specified in Clause 1037.

APPENDIX 6/8

TOPSOILING AND SEEDING

Drawings

305/11/1A
/2B
/3A

Topsoil

No imported topsoil Class 5B is required.

As stated in sub-Clause 618.3, topsoil shall not be excavated from stockpiles which have been exposed to accumulative rainfall of 150 mm over the preceding 28 days measured at the main site offices. For details of protection of topsoil stockpiles see Additional Clause 655 AR, Appendix 0/1.

The areas to be grassed and the treatment each area shall receive are shown on Drawing Nos: 305/11/1A and 2B.

Topsoil depths to be deposited in Treatments I and II are 225 mm on slopes of 10 except those areas shown on Drawing No: 305/11/3A as being areas of tree planting where the topsoil depth shall be 300 mm.

All turfing on slopes of gradient of 1:3 or steeper shall be pegged as described in Additional Clause 656 AR, Appendix 0/1.

The hydraulic mulch seeding used in Treatment III shall contain glass fibre as a retaining agent.

Areas of grass which require to be mown 3 times in the vicinity of Bridge No 5, East abutment are detailed in Drawing No: 305/11/3A.

The locations, access points and approximate contouring of permanent topsoil storage areas are shown on Drawing No: 305/11/3A.

APPENDIX 6/9

LANDSCAPE AREAS ETC

Drawings

305/10/1
/2B
/3
/4
/5

Location of noise bunds and landscape areas, cross-sections and contours, details of dense planting areas.

Environmental Bunds

Environmental Bund No 1 shall be constructed using a strengthened embankment to Clause 621. Environmental Bund No 2 shall be constructed as a normal embankment to Clause 619 except that Appendix 6/1 shall apply instead of Table 6/1, using Class 1B material.

Environmental Bund No 1 is detailed on Drawing Nos 305/10/1 and 2B using a geosynthetic reinforcement material at the vertical spacings as shown. The reinforcing material shall comply with the requirements of Additional Clause 660 AR, Appendix 0/1 and shall be laid and jointed as detailed in Additional Clause 661 AR, Appendix 0/1. The fill material shall be Class 6J material as detailed in Appendix 6/1, laid and compacted in accordance with Clauses 608 and 612.

The reinforcing material shall comply with the following additional requirements:-

1. Tensile strength of 20 kN/m at a maximum strain of 10% according to BS 6906: Part 1 (1987) except that the strain rate shall be set at 2% per minute ± 0.4% per minute.

2. Water flow of 30 litres/m2/s minimum under a constant 100 mm head of water according to BS 6906: Part 3 (1989).

3. Minimum Tear Strength of 400 N when tested in accordance with ASTM D-4533-85 (Geotextiles only).

4. Minimum Puncture Resistance of 2kN when tested in accordance with BS 6906: Part 4 (1989).

5. Be unaffected by acid/alkali or biological attack.

6. Contain an UV inhibitor if left exposed for more than 7 days in cumulative time.

7. Have a design life of 40 years.

Landscape Areas

Locations of Landscape Area Nos 1-4 together with details of access points and contours are given in Drawing Nos 305/10/3, 4 and 5.

Compaction of Landscape Area Nos 1, 2 and 4 shall be in accordance with sub-Clause 620.2. Compaction of Landscape Area No 3 shall be in accordance with Table 6/4 Method 4.

Construction of Landscape Area No 3 may be carried out at the same time as the adjacent Fill No 4 subject to the conditions described in sub-Clause 620.4.

All landscape areas shall be topsoiled and seeded in accordance with Treatment I, Clause 618. The minimum depth of topsoil shall be 300 mm throughout.