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State Road Authorities

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**THE DESIGN OF
ROAD EMBANKMENTS**

**TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS**

TRH 10

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ROAD EMBANKMENTS**

1994

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PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. They are based on South African experience and the results of research and have the full support of the Committee of State Road Authorities (CSRA).

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SYNOPSIS

Procedures for the design of road embankments are presented. Embankment design is viewed as being a process consisting of a number of stages of increasing sophistication regarding both investigation and analysis.

This document deals with the routine or simple methods which should be employed in the earlier stages. More complex cases require specialist geotechnical expertise.

This does not suggest that geotechnical expertise may only be required during the later stages of projects and it is emphasised that early recognition of potential problems is vital. Use of the appropriate engineering geological and geotechnical experience and skills at the initial stages of investigations is therefore strongly recommended.

Slope stability analysis is often considered to be synonymous with embankment design. In practice, post-construction settlement of road embankments is a much more common, and often more complex, problem. Emphasis is therefore given to settlement estimation.

All embankments must be designed. Although the majority will have standard slopes and no significant settlement problems, it is necessary to consider each embankment individually. To ensure this, it is recommended that some formal design procedures be adopted.

SINOPSIS

Prosedures vir die ontwerp van padopvullings word aangebied. Padopvullingontwerp word beskou as 'n proses bestaande uit 'n aantal stappe wat toeneem in sofistikasie ten opsigte van ondersoek asook ontleding.

Hierdie dokument handel oor die roetine of eenvoudiger metodes wat in die vroeër stadia gevolg moet word. Vir meer ingewikkelde gevalle moet daar van spesialis geotegniese kundigheid gebruik gemaak word.

Dit beteken egter nie dat geotegniese kundigheid slegs gedurende die later stadiums van projekte benodig mag word nie. Dit word benadruk dat vroegtydige identifisering van 'n potensiële probleem uiters belangrik is. Die gebruik van toepaslike ingenieurs geologiese en geotegniese kundigheid en ondervinding by die beginstadium van ondersoeke word daarom sterk aanbeveel.

Hellingstabiliteitsontleding word dikwels as sinoniem met opvullingsontwerp gesien. In die praktyk is nakonstruksievassakking 'n meer algemene en dikwels 'n meer ingewikkelde probleem. Klem word derhalwe op die skatting van vassakking gelê.

Alle opvullings moet ontwerp word. Alhoewel die meerderheid standaardhellings en geen beduidende vassakkingsprobleme het nie, is dit nodig om elke opvulling individueel te oorweeg. Om dit te verseker word daar aanbeveel dat sekere formele ontwerpprosedures aanvaar moet word.

KEYWORDS

Embankment, settlement, slope stability, consolidation

1 INTRODUCTION

This publication, in common with others in the Technical Recommendations for Highways series, is not addressed to specialists, but is intended to reflect current good practice.

It is aimed at non-specialist engineers who should have a basic understanding of the relevant geotechnical methods. The purpose of the document is to provide straightforward tools for assessing the potential difficulties of embankment problems so that an informed decision can be made regarding the necessity for calling on specialist advice. Practically everyone will recognize the potential problems caused by embankments on recent alluvial deposits and be prepared to go to some lengths to carry out sufficient investigation and design. The problems, however, of embankments on slopes may be much more subtle and require considerable experience to recognize and adequately investigate.

For convenience the investigation and design of embankment is viewed as a process which should be carried out in stages. These range from the application of engineering judgement to the use of highly sophisticated investigation and analytical techniques. This document adopts the approach of describing the earlier stages in some detail and then indicating when appropriate expertise should be called in to deal with the more complex problems. The majority of embankment problems arise because they are not identified and not because they are foreseen and then inadequately analyzed. The emphasis in the investigation and design process should fall on early recognition of potential problems. The stage process described, although seemingly beginning at a simple engineering judgement level, is probably the most important stage and it is at this point that the judgement is best exercised by an experienced geotechnical engineer.

The increased geometric standards for modern roads have led to much higher and longer embankments; the increased capital costs and utilisation have demanded much higher road performance standards, and these factors have led to the necessity for road engineers to give more attention to the geotechnical problems involved in embankment design. Although the most obvious and dramatic problem is slope failure, it may well be that the more costly problem in the long term is settlement which requires ongoing maintenance. The document therefore emphasizes the latter aspect.

Design and site investigation are necessarily closely interlinked and frequent reference is made to various investigation techniques and the results that are obtained from them. Detailed descriptions of these techniques, or of sophisticated analytical methods are not, however, given and the reader is referred to various publications for more information.

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BASIC STEPS IN SYSTEM

A = AIM OF INVESTIGATION
I = INFORMATION AVAILABLE
M = METHODS OF OBTAINING
INFORMATION OR ANALYSIS
E = EVALUATION CRITERIA
D = DECISION

STAGES IN SYSTEM

SITE INVESTIGATION AND DESIGN

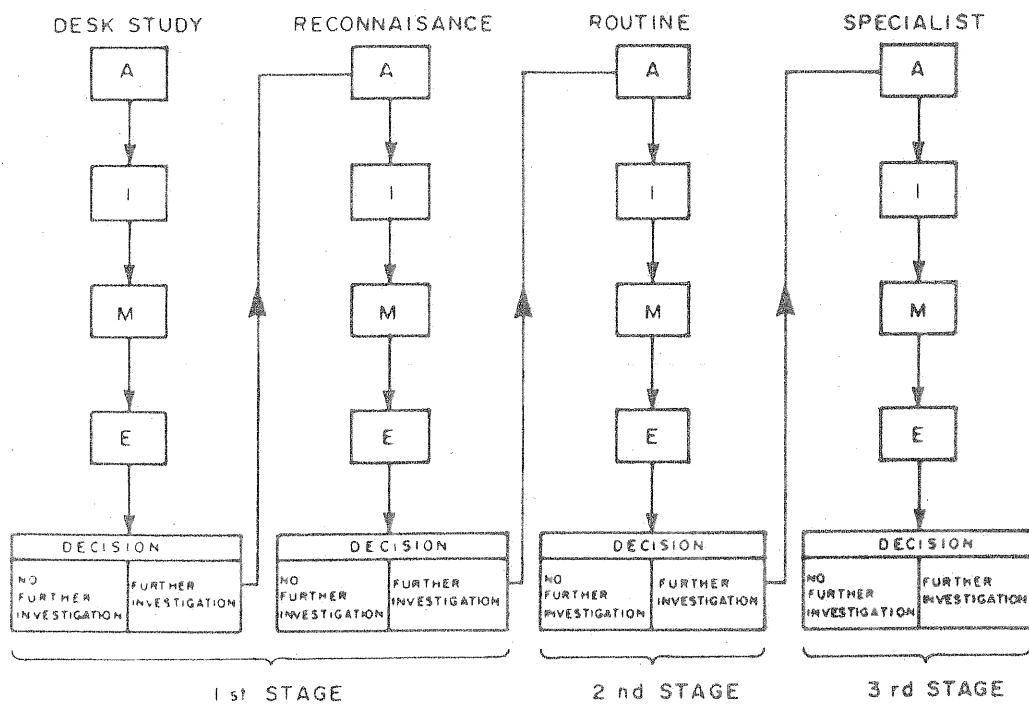


FIGURE 1

FLOW DIAGRAM OF GEOTECHNICAL INVESTIGATION AND DESIGN

2 DESIGN SYSTEM

The design of embankments should be seen as a rational process embodying a series of phases and stages of investigation and analysis, generally of increasing complexity, resulting in a decision. This decision may be either that sufficient information is available, or that more information is required, and/or that the problem should be addressed by a specialist.

Since in many cases initial potential problem recognition may be the most difficult aspect of a project it is recommended that consideration should be given to utilising a specialist at the reconnaissance stage where his input could be most useful both in appreciation of the problem and in indicating appropriate investigation. It may well be that little further specialist advice would then be required, depending on the nature and extent of the potential problems.

It is convenient to consider each of the stages as comprising a number of elements which combine to form an AIMED or rational process. Figure 1 illustrates the overall system, the elements of which are described in more detail in the following sections.

2.1 AIM

The ultimate aim, or objective, of the investigation and design is to provide sufficient information so that satisfactory embankments can be constructed economically.

The term "satisfactory" as applied to earthworks, means that subsequent deformations of the embankment should not exceed acceptable norms.

Considerable emphasis is usually placed on potential slope stability problems. However, in practice it is likely that excessive embankment settlement, with the consequent loss of riding quality, results in far greater economic loss.

Within each of the various stages of investigation and design the aims will be limited; for example, at the first stage, the aim may be to establish the in-situ soil profile with qualitative assessments of strength and compressibility. This may be sufficient to decide that no significant problem exists, or, on the other hand, to decide that either a stability or a settlement problem could occur. The aim of the next stage would be to obtain quantitative data on the relevant soil parameters.

2.2 INFORMATION

It is useful to consider the information available at any particular embankment against the background of knowledge of embankment performance generally. In order to do this it is convenient to classify embankments in terms of the probability of the occurrence of the two distress modes of stability and settlement.

2.2.1 Embankment classification

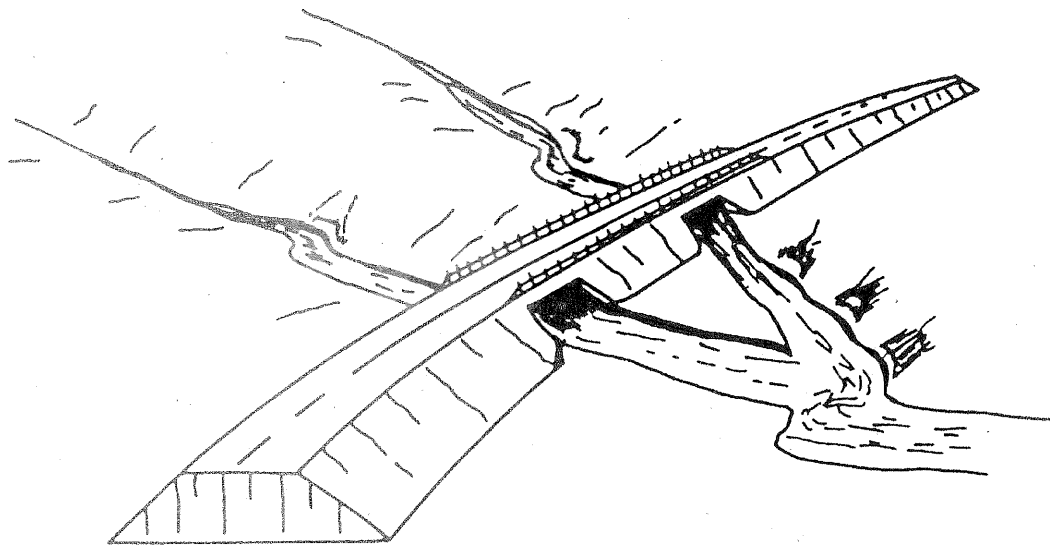
Embankments can be classified on the basis of four simple parameters, viz. topography, geology, climate and height, and this classification can be used to indicate the relative probability of significant potential problems of either stability or settlement as shown in Table 1.

In broad terms the topography can be classified as flat, side slope or gulley as illustrated in Figure 2.

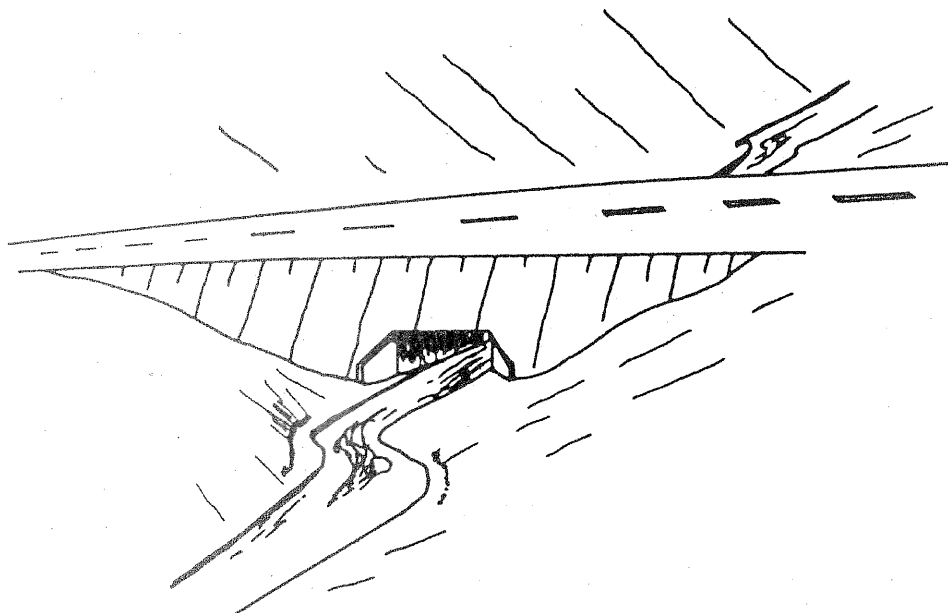
The geology can also be broadly classified, and combined with the topography to give an indication of the probabilities of distress for the two failure modes as shown in Table 1.

TABLE 1: Distress Probabilities

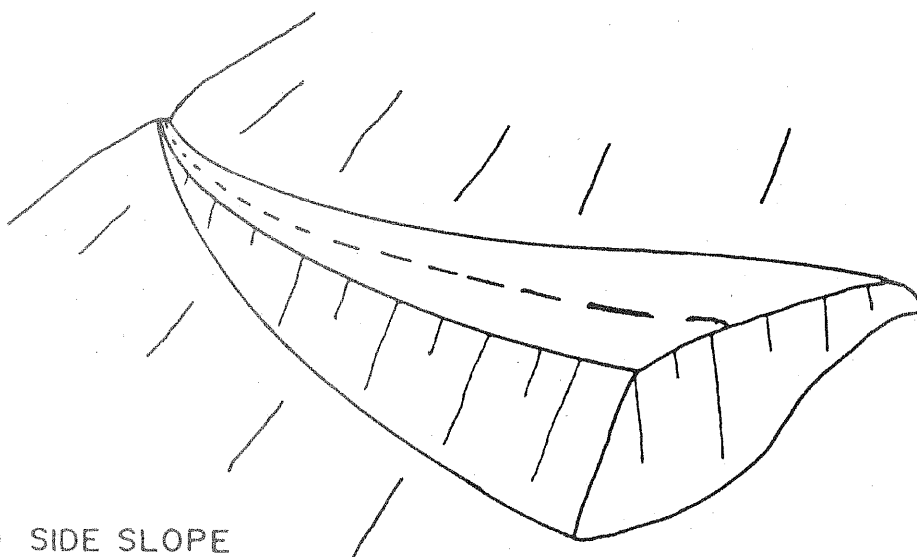
Topography	Problem	Probability of distress				
		Transported soils		Residual soils		Rock
		Alluvial	Other	Soft	Other	
Flat	Settlement	probable	likely	possible	unlikely	negligible
	Stability	probable	likely	possible	unlikely	negligible
Gulley	Settlement		possible	possible	unlikely	negligible
	Stability		probable	likely	possible	possible
Side	Settlement		possible	possible	unlikely	negligible
	Stability		probable	likely	possible	possible



(a) FLAT



(b) GULLEY



(c) SIDE SLOPE

FIGURE 2
TOPOGRAPHIC EMBANKMENT TYPES

No attempt is made to ascribe numerical values of probability to the classifications since it is emphasized that the five categories, ranging from probable, likely, possible, unlikely to negligible, only serve to give a relative basis for comparison of potential embankment problems on any project. In this way attention can be directed to the more probable potential problems. These relative probabilities should then be increased in direct relationship to both the rainfall and heights of the embankments.

Rainfall has a most important bearing on slope stability, particularly for embankments on side slopes or gulleys, since the phreatic surface may rise as a direct result of rain and hence reduce the effective stresses and also therefore the shear strength of the subsoil. The annual rainfall has a direct bearing on the probability of stability failure of embankments. In the water surplus areas of the country in particular, it is essential to take into account the potential effect of changing moisture contents on slope stability. The water surplus areas of South Africa are shown in Appendix A.

Both slope stability and settlement are also directly related to embankment heights. Hence, in general, potential problems for high embankments are more probable than for low embankments. Cognizance should therefore be taken of this aspect in rating the relative probabilities of distress. It is, nevertheless, necessary to take care not to give undue significance to the embankment height to the extent that low embankments are ignored. There are for example, numerous embankments below 5m height which have suffered severe distress from both excessive settlement and instability and, conversely, many embankments over 15m height which have no possibility of distress.

2.2.2 Site Information

By the time an individual embankment design has to be considered a large amount of specific information should be available; eg from the proposed geometry of the embankment, ie height, width and length, and data from the preliminary investigations carried out as recommended by TRH2¹. In addition, aerial photographs will probably be available, but not least of the sources of information, is local experience. This experience is seldom formalized, but exists in the minds of people familiar with the area; these may be road authorities, or their consultants, but they may also be agriculturally orientated with a natural interest in sources of water and drainage. Careful checking of aerial photographs, or other remote sensing, could well prompt the relevant questions on water which local knowledge can answer.

2.3 METHOD

This refers to both the investigation and design methods. It is again emphasized that the whole process is a stage, or an iterative one, with multiple loops. In even a fairly simple case, the process could consist of, for example, four investigation stages, viz. soil engineering mapping, reconnaissance of embankment

sites, borehole investigation and, finally, a more sophisticated sampling and testing programme. After each of these stages, the information is evaluated and a decision is made regarding the necessity for further investigation.

Method refers both to the process of obtaining that information, which may be aerial photo interpretation, visual inspection, boreholes or in-situ testing and to the methods of analyzing the problem. The latter could vary from a simple judgement, a rule-of-thumb approach, or a sophisticated analysis of stability or settlement. Descriptions of preliminary methods of stability and settlement assessment are described in this document. Methods of soil investigation are not described in detail, since they are adequately described, and specified where relevant, in other publications.

2.4 EVALUATION

This refers to the process of evaluating the results of any investigation or analysis. Evaluation can only be accomplished by comparison with a predetermined set of parameters. In the majority of embankment situations, evaluation is no more than straightforward engineering judgement, eg the embankment will be satisfactory because it is to be constructed over competent materials and there are no apparent drainage problems. In other cases, it may be clear that insufficient information is available on which to base a reasonable judgement.

In more complex cases, the evaluation phase may be the comparison of calculated factors of safety for slope stability, or settlement predictions, with criteria which would normally be considered acceptable for the particular standard of road and individual embankment situation.

It must be emphasized that evaluation is only one part of the overall design process. There may well be a number of evaluation stages required to complete this process.

2.5 DECISION

It is strongly recommended that this phase, although very closely tied to the preceding evaluation, should nevertheless be seen as a separate formal process.

There can be little doubt that the discipline of recording a formal decision serves to prevent the oversight of potential problems: usually it is the failure to take account of potential problems, rather than their misunderstanding, or inappropriate design, which gives rise to real difficulties.

Essentially, the decision will either be that sufficient information exists for a reasonable judgement to be made, or, alternatively, that further information is required. Hence more investigation and another stage in the process is necessary.

The application of the AIMED procedure is considered to be equivalent to a normal engineering design process. In order to plan the investigation and analysis of embankments it is necessary to formalize the process. It is recommended that this should be achieved by drawing up a list of all embankments and completing the information which is available at the beginning of the design stage. This information is described in the preceding Section 2.2 and comprises a description of the position and size of the embankment, the topographically determined category, an assessment of the nature of the subsoil and hence of the possible problem, ie stability or settlement, and its severity.

From the foregoing classification of the embankment size, the topography and the potential problem, it is recommended that a design sheet for each embankment on the project be drawn up before the initial site inspection, so that the site investigation can be planned.

It is essential to study the aerial photographs and to indicate on these the road line, the embankment positions and any relevant subsoil information given on the soil engineering maps. It must be emphasized that this initial desk study of the problem is the key to the efficient and successful design of embankments.

Examples of design sheets are given in Tables 2 and 3. These examples were chosen to represent extreme situations; the first a comparatively difficult site where eventually a sophisticated investigation and analysis is required, and the second, a simple site where the preliminary information is almost sufficient to enable the designer to decide that no further investigation is required. Most embankments fall somewhere between these two.

It is important to take cognizance of the fact that the information obtained during the site inspection may be influenced strongly by the weather, and in particular the rainfall, at the time. The recorded information should specifically indicate the actual conditions at the time of the inspection and try to relate these to the range of conditions which may be expected through normal seasonal changes. For example, the inspection may have been carried out during an unusually wet or a dry period and this should be noted, preferably with reference to rainfall data.

TABLE 2: Embankment information and design sheet

PROJECT	NR 123. STARTVILLE - ENDBURG
---------	------------------------------

POSITION	km 11 100
----------	-----------

DESCRIPTION	2 x 150 m x 4 m approach fills to river bridge
-------------	--

PRELIMINARY INFORMATION	Deep clayey recent alluvium, subject to flooding, old road bridge piled and embankments distorted.
-------------------------	--

PRELIMINARY CATEGORY	Flat/alluvium/settlement/stability/probable
----------------------	---

INVESTIGATION SUMMARY

RECONNAISSANCE

DATE/PERSONNEL	9/10/83	A B CHARLES / D E FIELD
----------------	---------	-------------------------

OBSERVATIONS	River banks clay, old road embankments settled. Highly probable severe settlement and stability problems	
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DECISION	Specialist investigation needed	Signed A B Charles
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FIRST-STAGE

METHOD	8 Bore-hole samples for lab (6 at bridge), 10 CPTU, lab shear/consol
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EVALUATION	Settle 0,5 m; FS = 1,7. More sandy and permeable than expected, Rock 7 m
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DECISION	No further investigation	Signed A B Charles
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FINAL-STAGE

METHOD	
--------	--

EVALUATION	
------------	--

DECISION	Signed
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ASSUMPTIONS/ RECOMMENDATIONS	Build fills early including at abutments, say 1 year before piling. Monitor settlements by simple level plates at base of fills
---------------------------------	---

TABLE 3: Embankment information and design sheet

PROJECT	NR 123. STARTVILLE / ENDBURG	
POSITION	km 12 250	
DESCRIPTION	100 x 4 m with 4 x 3 box	
PRELIMINARY INFORMATION	Rock exposed each side gulley, small watershed fairly flat slopes, well defined stream, but possible marshy areas	
PRELIMINARY CATEGORY	Gulley/transported alluvium/settlement	
INVESTIGATION SUMMARY		
RECONNAISSANCE		
DATE/PERSONNEL	9/10/83	A B CHARLES/ D E FIELD
OBSERVATIONS	Rock exposed in fairly flat gulley, sandstone 5° down gulley dip, probably very thin colluvium	
DECISION	Further investigation	Signed A B Charles
FIRST-STAGE METHOD	4 pits with backactor	
EVALUATION	Pits show colluvium up to 1 m thick at centre but thinning to average 0,5 m thick. Considerable seepage on underlying rock; some springs	
DECISION	No further investigation	Signed A B Charles
FINAL-STAGE METHOD		
EVALUATION		
DECISION	Signed	
COMMENT	Recommend remove colluvium and replace with rock blanket to control springs. Box culvert does not require floor	

The purpose of using an individual design sheet for each embankment is that it ensures a disciplined engineering approach, gives a convenient summary of the conditions at each embankment, eventually provides communication of the geotechnical engineer's assumptions and intentions to the construction team, and, possibly not least important, it formalizes the design responsibility.

At the initial desk-study stage it is likely that only the upper section of the design sheet will be completed. The lower section will almost certainly require at least a site inspection and can only be completed at that stage. A number of subsequent stages could be necessary for the more complex problems and one design sheet may not be sufficient to contain all the data and decisions. Nevertheless, it is advocated that the formality of the decision-making and recording process should be followed.

The information from the design sheets for all the embankments on the project should be summarized in a form similar to that shown in Table 4.

Many embankments have structures through them. It is often the interaction between these structures, the embankment and the in-situ material which gives rise to potential problems, hence the advisability of bearing in mind the presence and type of proposed structures.

At the desk-study stage only the left-hand side of Table 4 can be completed. The right-hand side would be completed after the site inspection stage, from the individual embankment investigation and design sheets, eg Tables 2 and 3.

TABLE 4: Site Investigation for embankments

PROJECT		MR 123	STARTVILLE - ENDBURG		km 10 700-14 300		
EMBANKMENT CATEGORY			PROPOSED INVESTIGATION				
km	SIZE m STRUC TURE	TOPO- GRAPHY	GEOLOGY PROBLEM	PITS AUGER	BORE- HOLES	PROBING	SPECIAL
11 100	300 x 4 2 span bridge	flat	deep soft clay alluvium settlement stability		8 with sampling	10 CPTU	Lab shear consol
11 700	80 x 3 1 small pipe	side	residual granite ? collapse settlement	3 pits			in situ density Lab collapse tests
12 250	100x 4 4x3box	gulley	colluvium ?stability	4 pits			
13 100	200x3	side	rock stability	visual check dip/faults/signs of water			
13 750	150 x 8 2 x 2 box	flat	transport ed sand? settlement		3 with SPT		
14 050	400 x 8 3 span river bridge	flat	shallow transport ed sand? settlement		6 at bridge for rock qual ity. SPT	4 CPTU	
			TOTALS	7	17	14	

3 SITE INVESTIGATION

The overall investigation is divided into the desk-study phase and the site-investigation phase. The latter, in turn, is subdivided into stages. The first of these is reconnaissance. This is followed by more detailed investigation stages as required. There is no limit to the number of stages, but it can be expected that a second, and possibly a third, will be the most that will be required by the general road designer before specialist geotechnical expertise is called upon.

3.1 RECONNAISSANCE

There can be no doubt that this is the most important phase of any investigation. From it the initial decisions are made as to whether or not further direct investigation will be carried out.

At the time of reconnaissance the geotechnical and soil engineering maps and reports recommended by TRH2¹ should be available, together with aerial photographs. If, for any reason, the maps and reports have not been compiled, then it is practically essential that preliminary maps showing the geology should be drawn up by an engineering geologist, using existing information. These should show the proposed road centre line, the embankment positions and sizes and geological formations. The individual design sheets, Tables 3 and 4, can then be completed, so that by the time the site is visited, not only has a good picture of every embankment site been formed, but an idea of the potential problems has also been obtained. The reconnaissance stage is the transition from an engineering geological emphasis to that of geotechnical engineering. It is therefore most useful, if not essential, that the inspection should be carried out by the engineer responsible for the embankment design, in the company of the engineering geologist responsible for the mapping.

From an understanding of the geology it should be possible to look for information specifically relevant to the particular embankment. It is useful to do this on the basis of the embankment's topographical type, ie flat, gully and side slope.

3.1.1 Flat

Firstly it is necessary to decide whether the subsoil is transported or residual. If it is transported, then problems are likely, since the material may vary from soft alluvial clays to collapsing sands or expansive clays, and further investigation becomes essential.

Residual materials in flat areas are not as likely to give rise to embankment problems unless they have collapse or expansive potential. It may nevertheless be necessary to carry out some field investigation.

The type and amount of investigation, ie pits, auger holes, boreholes, etc. should then be assessed. Every effort should be made to site the boreholes so that maximum information is obtained where it is most relevant. Where embankments cross alluvial deposits there will probably be a river or stream requiring a structure. The investigation for the structure will have its own requirements. However, it is economical to use the same boreholes to provide information for both the embankment and structural design.

It may, for example, be obvious that the structure will have to be piled and information may be needed mostly on the depth to the rock and the rock quality. These aspects are of little relevance to the approach embankments for which information on the overlying soil is required. The type of the borehole and the sampling and testing should then be defined by the requirements of the embankment, not the structure. Even if the structure is to be piled, information on the soil cover is still required in order to choose the correct type of pile.

However, unless the proposed embankments are fairly large, information from conventional boreholes for the structure may be sufficient for the embankment design. If further boreholes or probes are required, they should be positioned with care. The topography and the air photos, and even the vegetation, will probably give some indication as to where further investigation should be located. Only rarely is it good practice to position boreholes at regular chainages and at standard offsets from the centre line.

Nevertheless, selective positioning of boreholes probably reflects the most pessimistic view and the designer should take this into account to avoid excessive conservatism.

3.1.2 Gulley

This topographic type refers to gulleys with a significant side slope, for example, where the road traverses the flank of a hill intersected by drainage features. Gulleys with small crossfalls, ie where the crossfall has no significant influence on potential modes of failure, are essentially similar to flat topography situations.

Gulleys often develop due to geological discontinuities, such as faults or weathered intrusions. It is therefore essential to look for these features to assess their possible influence on the proposed embankments.

Most embankment problems at gulleys are a direct result of poor drainage and consequent high pore pressures.

This may be the result of inadequate design and construction, but is more probably because drainage paths were not noticed at the time of the investigation, or may have changed as a result of construction.

One very important purpose of reconnaissance is, therefore, to look for all signs of water, not only in the main water course, for which an understanding of the geology is essential. In some cases, there may be changes in grade along the stream course indicative of more resistant bedrock which could give rise to springs. These features, or any others likely to cause a build up of water under the embankment, must be noted and provision made for them during construction, since they are likely to cause problems.

In gulleys with moderate to steep crossfalls, there are not likely to be large depths of colluvium. Stability failures, particularly on slopes, however, do not necessarily require such conditions. A fairly thin layer of poor material may be sufficient to cause instability. Colluvium adjacent to the main water course may have a low moisture content at the time of the investigation and appear to have a satisfactory shear strength. This will almost certainly change after construction, due to a gradual wetting up and caution must be exercised in assuming strength parameters based on the initial inspection.

3.1.3 Side slopes

In many respects the problems at these sites are similar to those encountered at gulleys with the complication that usually no obvious water course exists. It is likely that there will be some water flow across the line of the embankment, due to the crossfall, and it is important to envisage how this flow pattern will be changed by the embankment construction.

The reconnaissance must therefore be particularly aimed at examining the terrain for signs of water. Aerial photographs are often most revealing. These should be annotated to indicate any anomalies which require closer inspection during the investigation. In particular, vegetation changes or irregularities in the contours must be checked. The area may already be potentially unstable before the embankment is constructed, or could even be the site of an old geological failure. One must therefore look for scarps, anomalous bulges, odd outcrops, broken contours, ridge top trenches, fissures, terraced slopes, abrupt changes in slopes or in stream directions, and springs or seepage zones. It is necessary to examine not only the immediate area that will be covered by the embankment, but also the area on both sides.

During the reconnaissance stage of the investigation, notes should be taken at every embankment site with the intention of completing design sheets similar to those given in Tables 2 and 3. It is often useful to tape-record comments and to photograph the site. Whatever method is used, it must be most strongly emphasized that it is an integral part of the overall investigation and design process and as such should

be given proper attention. On projects requiring reconnaissance of a large number of embankments, sufficient time must be allowed for a thorough examination of all the sites.

3.2 SECOND-STAGE INVESTIGATION

The reconnaissance will have determined the proposed type and amount of second-stage investigation required, and this should be recorded in the design sheets and summarized in Table 4.

It is probable that the investigation for cuttings and structures will take place concurrently with that for the embankments, using the same field equipment and crews. The investigation must be planned so that any further stages can also be included in the same contract. It is not necessarily most efficient to start at one end of the road project and work through to the other; it could be preferable to tackle the more difficult sites first, where it is thought that further investigation may be necessary, and thus to allow time to evaluate the results and to decide what is required.

The optimum investigation should provide just sufficient information so that the design can be adequately performed. This is not easy to achieve and all too often the investigation team has to return to the site to complete the work; frequently, the possibly more expensive, although less obvious, mistake is made, of having too many boreholes, followed by excessive laboratory testing, the results from which are neither needed, nor used in the design.

Considerable emphasis is given in this document to in-situ testing and the consequent assessment of soil parameters. This does not suggest any dichotomy between in-situ and laboratory testing, since both have their place. It does, however, indicate that for the degree of sophistication which the document addresses, it is probable that adequate information will be readily obtained from in-situ inspection and testing, which is generally convenient and economical.

The aim of the second-stage investigation is two-fold.

The first is to confirm the provisional information obtained on the various strata using the geotechnical and soil engineering maps and reports and by reconnaissance.

The second is to measure relevant subsoil parameters, either through in-situ testing, or by taking samples for laboratory testing.

Brief descriptions of the field techniques used to achieve these two objectives of strata delineation and soil parameter determination are given in Appendix B.

The following two sections describe these two objectives in some detail because it is believed that they are of fundamental importance in developing realistic models of the embankments being designed. It is again stressed that the problems of analyzing the models are relatively simple. It is much more difficult to correctly represent the real situations with appropriate models.

The modelling process should be seen as consisting of macro and micro-scale phases, the former being the strata definition and the latter being parameter determination.

3.2.1 Strata delineation

Strata are delineated from the description of the soil profiles obtained from the trial pits, auger holes and boreholes, and in some cases by the use of Cone Penetration Testing.

The method of describing the soil profile is generally accepted in southern Africa to be that recommended by Jennings, Brink and Williams².

Similarly rock descriptions should be as recommended by the "Guide to Core Logging" of the Association of Engineering Geologists³.

The primary emphasis should be on obtaining a three dimensional view of the geology. It is also important to bear in mind the potential embankment problem when describing the profiles, so that emphasis can be placed on delineating the particular stratum, or strata, which are expected to cause the problem. In most cases, the soils likely to give trouble are close to the surface and probably transported. It is important, therefore, to comprehend the particular transportation history and mechanism and the result this has on the nature of the soil and its distribution. Engineering geological input is essential for this.

In many instances a number of boreholes may be put down that show, with one or two exceptions, very similar strata; the exceptions may be significant and it is most important to understand these anomalies, because they may reveal the source of potential problems.

It clearly makes the most economic sense to carry out any further field work, which may be necessary to satisfactorily delineate the strata, when the drilling machines are on site. It follows therefore, that the logging of all boreholes should preferably be carried out as an ongoing process. This reinforces the recommendation given previously that close supervision of the investigation is essential, since changes to the proposed investigation, as a result of the information being obtained, will produce the most economical and efficient investigation and subsequent design.

Although many embankment problems originate in transported soils, numerous examples of problems, particularly stability problems, have occurred over residual materials. Similar strata delineation considerations apply and these may be more critical than those for transported materials. For example, if the potential problem is stability over a residual material on a side slope, the failure surface may include a large volume of the subsoil. It would be unrealistic to assume the lowest measured shear strength to represent the strength for the full failure surface. Alternatively, the failure surface could be limited to a relatively weak thin stratum. A detailed description of the delineation of that stratum then becomes essential. It is important to understand the potential problems, if the most appropriate investigation is to be carried out. Appropriate does not necessarily imply sophisticated. For example, a series of shallow holes, or trenches, may be of much more value than deep boreholes, although some boreholes may be required to establish the overall geology.

3.2.2 Soil parameters

Embankment problems are classified under stability and/or settlement. The relevant soil characteristics for these are strength and compressibility.

Strength: The shear strength of the subsoil is required to estimate the stability of slopes. This can be assessed from the description of the material, or from in-situ tests, such as the CPT, SPT and vane shear tests. Criteria for the assessment and evaluation of the tests and subsequent analyses are given in the section on stability design. If these indicate a stability problem, more sophisticated testing may be required. Laboratory tests usually comprise shear box or triaxial testing. The shear strength of clays is represented by either the undrained strength, c_u , or the drained strength, c' and ϕ' .

Undrained and drained strength model respectively, the situations of a rapid application of load, which does not allow time for excess pore pressure dissipation, and the slow application of load, which allows excess pore pressure dissipation and hence a gain in strength.

It may be noted that few embankments require laboratory shear strength tests. The emphasis should be on obtaining good quality, reliable field results and, in particular, a measure of the distribution of shear strength within the strata. If these results indicate stability as a potential problem, then carefully controlled sampling and laboratory testing becomes a necessity. Haphazard taking of samples, followed by laboratory tests, in case they are needed, can never be justified.

Compressibility: This is considered as comprising two components representing the short-term and long-term behaviour. Short-term behaviour is modelled by an elastic modulus, E , and long-term behaviour

by coefficients of volume change m_v - and coefficients of consolidation - c_v . The latter are dependent on overall subsoil permeability.

The elastic moduli can often be determined by in-situ tests, which take a short time, whereas assessment of long-term compressibility requires sampling and laboratory testing. In-situ tests, however, are often relevant to assess rates of settlement since these depend on subsoil permeabilities, which may be best measured in-situ.

Wherever possible, laboratory measurements should be compared with field results and considerable weight should be given to the latter if the tests have been properly conducted.

3.2.3 Moisture regime

It has been stated that soil mechanics is not the study of the engineering of soils alone, but is primarily the study of the effects of moisture movements within soils. This may be a truism, nevertheless, it is remarkable how many geotechnical site investigations give little or no attention to the existing moisture regime and the possible changes in the regime, both due to seasonal variations and to the influence of the proposed earth structures.

It is easy to make such obvious statements, but in practice, it is difficult to obtain reliable quantitative moisture-related data.

The most basic information can, however, be readily acquired, namely the water table at the time of the investigation and the moisture conditions of the soils.

The extreme significance of pore water pressures on stability and settlement is discussed in more detail in subsequent sections on design. Suffice it to say, at this point, that investigations that do not include measurements of the current moisture conditions, and assessments of these in future, are totally inadequate if any real design is required.

4 EMBANKMENT DESIGN CRITERIA

4.1 INTRODUCTION

The design of embankments, in common with the design of all engineering structures, is the process of modelling a real situation, analyzing the model and comparing the results of the analysis with predetermined criteria. In engineering, these criteria are either explicitly stated in terms of factors of safety relative to failure state - as in structural design - or may be implicit, through a performance-related standard, such as the California Bearing Ratio (CBR) method of pavement design.

This design philosophy applies to embankment design. Stability is expressed in terms of factors of safety. Settlement is assessed relative to a norm judged to be suitable for the particular embankment.

The analysis of the selected models by calculations of factors of safety for stability, or estimations of settlement, are relatively straightforward procedures, once the input parameters have been selected.

Difficulty often arises, however, in defining acceptable criteria for factors of safety and for settlements, and in deciding the confidence level required for the calculated values.

Since the aim of the investigation and design is to determine stability factors of safety and estimated settlements, and to compare them with established criteria, it is necessary to define these criteria. These cannot be universal for all embankments, eg factor of safety not less than 1.5 or settlement not greater than 0.5 m but they must reflect the particular circumstances.

4.2 STABILITY CRITERIA

Stability is usually defined in terms of a factor of safety concept:

$$\text{Factor of Safety, } F = \frac{\text{Resisting Forces, } R}{\text{Shearing Forces, } S}$$

The common acceptance criterion is that F should not be less than 1.5. However, it must be emphasized that such a criterion is incomplete because calculated factors of safety are dependent on the analytical method used. Different methods may result in values of factor of safety varying by approximately 20 %, with identical input parameters. Therefore, a stated factor of safety should also state by which method it is calculated. This is possibly a refinement which should be left in the realms of specialist geotechnical engineering.

Reference, if required, should be made to textbooks on slope stability. A particularly useful one is "Slope Analysis"⁴. This reference also discusses valid, and possibly more rational, alternative concepts for assessing safety.

For example, a safety margin (SM) could be defined as the amount by which the resisting forces (R) exceed the shearing forces (S):

$$SM = R - S$$

and if $F = 1,5$

then $SM = 0,5S$

however, if $F = 2,0$

then $SM = 1,0S$

Although the safety margin has doubled, the factor of safety has only increased from 1,5 to 2,0. The concept of safety margin may be considered as analogous to a simple financial concept of profit expressed as income minus expenditure which gives a monetary value. Although this may be meaningful in some respects within a specific situation, it does not form a useful basis for the comparison of performances between different situations. A much more useful concept is to relate profit to income as a percentage, or for embankment stability, a margin (M) is defined in terms of the safety margin (SM) and the resisting forces (R):

$$M = \frac{SM}{R}$$

$$M = \frac{R - S}{R}$$

$$\text{or } M\% = 100 \left(1 - \frac{1}{F}\right)$$

It is clear that a factor of safety of 1,5 gives a margin of 33 %, which is twice that of F of 1,2, which gives a margin of 16 %.

The factor of safety is, therefore, non-linear relative to safety expressed by the rational margin concept. This exemplifies the significant disadvantages in the use of the factor of safety concept, since relative factor values are difficult to assess.

For example, at any project, the view could be taken that the required F should be conservatively set at say 1.8 for large embankments, instead of 1.5. To achieve this would result in increasing the margin from 33 % to 44 %. This would probably be done by decreasing the embankment slope, which would be expensive and almost certainly unjustified. It would be much more fruitful to establish the subsoil and moisture parameters with more confidence, possibly by more investigation and testing, and then using the lower margin or factor of safety of 1.5.

Despite the problems with the definition of the conventional factor of safety, it is likely to be the standard criterion for some time. Therefore guidelines for acceptable factors of safety for road embankments are useful.

Embankments, and their appropriate factors of safety, may be divided into two types on the basis of whether the pore pressure is a dependent variable, or an independent variable. Figures 3 and 4 illustrate these situations.

Figure 3 shows a typical u -dependent situation in which the pore pressure at a point P under the embankment increases during construction and then dissipates eventually back to the same equilibrium ground water level. If the pore pressure ratio, which is defined in the figure, is calculated for the point P throughout the same time path it will show the same pattern. In addition to this, however, if the average pore pressure ratio around the slip surface is calculated by varying both $h_{u,s}$ and also h_u under the embankment slope, then this too follows the same pattern. It may also be noted that in practice it is a reasonable approximation to use a single average value of r_u around the slip surface where u is calculated directly from the vertical pressure due to the height of fill. The justification for this use of an average value for r_u is that the pore pressure under the centre of the embankment is exaggerated, because some dissipation takes place even during construction, whereas immediately outside the toe, the calculated excess pore pressure, ie zero, would be underestimated since pore pressures would in fact be generated because the induced stresses from the fill are not only vertical.

The figure shows that stability criteria for two situations are required, viz. firstly at the end of construction, when undrained conditions are assumed and hence a total stress $\phi = 0$, analysis is appropriate, and, secondly, during and at the end of the period of pore pressure dissipation, when an effective stress, c' , ϕ' analysis is appropriate.

Figure 4 illustrates the u -independent situation. It shows a hypothetical case in which at the beginning of (1) before the embankment is constructed, the upper subsoil is dry but there is a phreatic surface as shown. During construction there may be a phase where u is dependent on the height of fill and then dissipates to an equilibrium level. During (2) the subsoil moisture content may increase, possibly due to

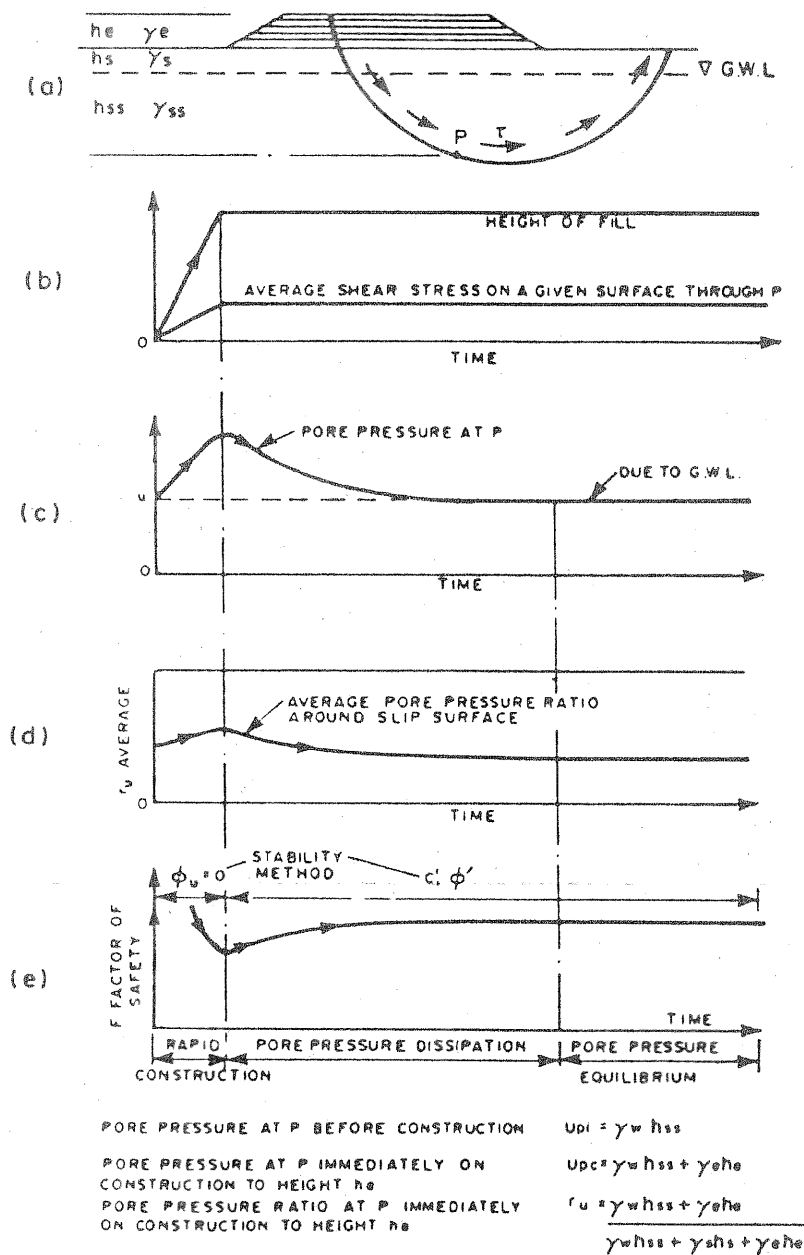


FIGURE 3
VARIATION OF FACTOR OF SAFETY WITH
TIME FOR EMBANKMENT ON SATURATED CLAY
U-DEPENDENT

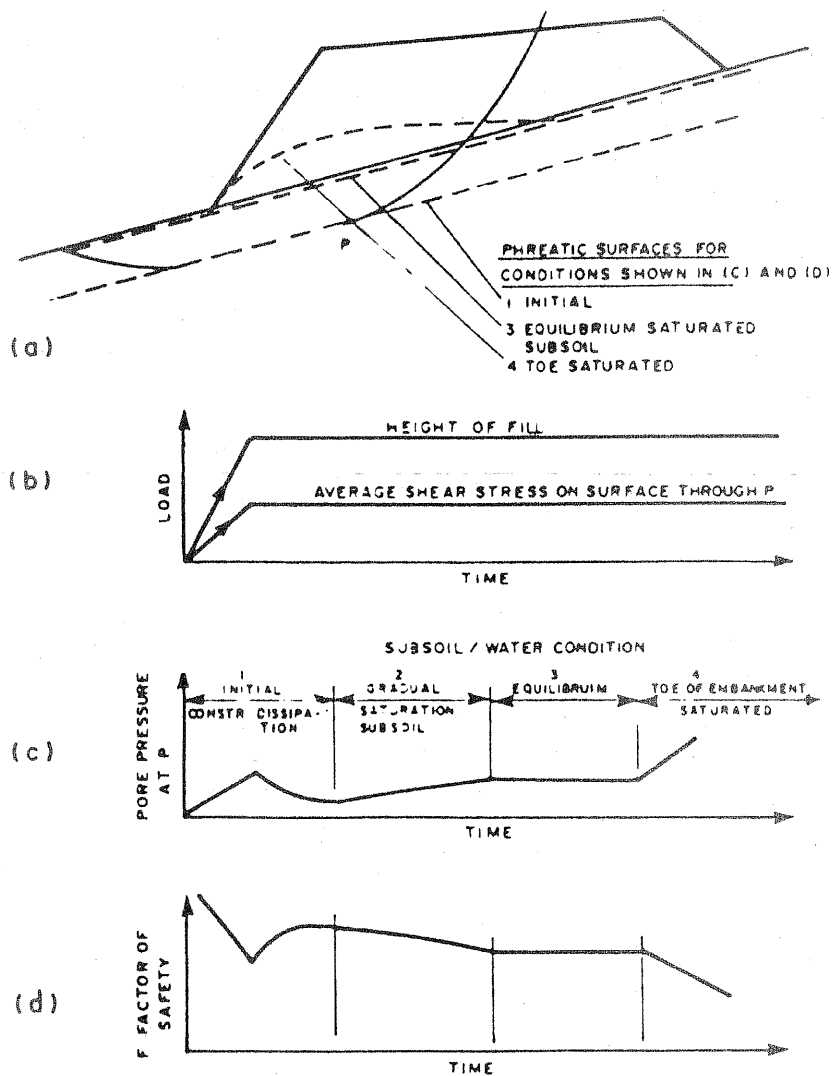


FIGURE 4

VARIATION OF FACTOR OF SAFETY WITH TIME
FOR EMBANKMENT ON SIDE SLOPE. U-INDEPENDENT

elimination of evapo-transpiration with the removal of vegetation, or due to direct interference with drainage, or even some unrelated change in moisture conditions. The example then assumes a period of equilibrium (3) takes place which could be for many years, before the drainage conditions again change to (4) when the toe of the embankment becomes saturated. This could be from many different indirect causes or more directly, for example, because a culvert through the embankment fails in some way.

For both u-dependent and u-independent causes it may be necessary to improve the stability of the embankments. Although the appropriate analyses for this would more often than not be carried out by a geotechnical engineer, and hence fall outside the scope of this document, it is worth noting that fill benches or berms may significantly increase the calculated factors of safety. For u-dependent cases, however, it should be remembered that although the calculated factors of safety may be improved, it is possible that berms could impede drainage thus resulting in a worsening of the situation.

The potential stability of embankments is dominated by the pore pressures built up in the subsoil and to a certain extent in the embankments themselves.

In the u-dependent situation it is unlikely that high pore pressures will be developed in the embankment, but they will be highest directly under the embankment. In some cases it may be advantageous to provide a drainage layer at the interface to allow rapid dissipation of the excess pressures not only to improve stability, but also to accelerate settlement.

In u-independent situations high pore pressures can be developed within the embankment as well as in the subsoil and provision of drainage systems for both may be an absolute necessity. Rock toes, sand blankets, pipes and geofabrics are commonly used for this purpose (TRH 9⁵), and the difficulty lies not so much in the design of a particular system, but in deciding when and where it should be used. Through gulleys it is likely that embankment drainage systems will be required, but along side slopes the necessity will be less and the indications for drainage will almost certainly be more subtle. A thorough understanding of the geology, aided by air photo interpretation, are prerequisites for deciding this.

4.2.1 Pore pressure (u) dependent

If u is a dependent variable, ie the pore pressure developed is directly related to the height of the fill, a total stress analysis should give a factor of safety (F) of not less than 2.0 for the initial assessment. This initial-assessment stage is based on estimated values of shear strength from visual descriptions of the subsoil.

If the calculated factor of safety is less than 2,0, more detailed assessment will be necessary. This will probably entail relatively simple in-situ testing to establish the appropriate values for undrained shear strengths. The minimum acceptable factor of safety for this stage is 1,5.

If the calculated value is less than 1,5 it will be necessary to carry out further shear strength tests and analyses to evaluate the possible gain in strength due to consolidation during the construction process. An effective stress analysis is necessary for this and geotechnical expertise will be required. During construction a factor of safety of 1,3 may be acceptable, provided that there is a high level of confidence in the definition of the subsoil strata and of the shear strength parameters.

If monitoring is possible, and if the rate of construction could be controlled if necessary, the acceptable factor of safety may be reduced to 1,2.

The embankment stability should also be analyzed for the post-construction situation when the excess pore pressures have fully dissipated. The factor of safety should then be not less than 1,5. In practice, it will probably exceed this value, but it is strongly recommended that the check analyses should be made.

4.2.2 Pore pressure (u) Independent

If the pore pressure is an independent variable, ie it responds to ground-water level, it is much more difficult to be specific regarding acceptable values for the factor of safety. Such embankments should first be analyzed on a total stress basis for the known construction conditions. The acceptable factors of safety should be the same as for the u-dependent cases described above, ie 2,0 for the initial assessment, and 1,5 for the second-stage assessment. Stability should then be analyzed for possible long-term situations. For this, it is necessary to make assumptions regarding future moisture conditions. From these, pore pressures can be deduced, and used for effective stress analyses. The factor of safety should be not less than 1,2 even under most adverse conditions.

In practice, however, a different approach will probably be adopted for the design, and the objective will be to devise drainage measures to ensure that the adverse water conditions, which will lead to a factor of safety of 1,2, cannot actually be exceeded.

4.3 SETTLEMENT CRITERIA

Freeway embankments have settled more than 2 m without any problems arising. For example, one of these has no structures through it and is across a flood plain on a sag vertical curve. The sub soil

increased in thickness from both ends towards the centre. The settlement followed that pattern, with the result that the vertical curvature was increased. No problems regarding riding quality or aesthetics arose.

If this embankment had encompassed a major structure, the structure would almost certainly have been piled. There would have been a major problem of downdrag on the piles, due to consolidation of the surrounding subsoil, and there would have been the almost impossible task of correcting the levels at the bridge approaches. A post-construction settlement of as little as 100 mm, adjacent to the structure, would have caused considerable difficulties.

It is obvious that settlement per se, is not the problem, but it is the rate of settlement, in conjunction with the amount, that is significant. Furthermore, it is these, relative to the construction period and the presence, or otherwise, of structures, that is important.

It is not feasible to express these variables in the form of simple criteria, such as an allowable settlement equation and it is necessary to use judgement. In order to do this it is necessary to express the predicted settlement of any embankment, in terms of both amount and time. These may both be estimated, within very approximate limits, from the initial or second-stage assessment.

Typically, embankment settlement predictions could be stated as say, 100 ± 50 mm over one year, or $1,2 \pm 0,2$ m over two to five years. All such estimates are assessed on the basis of whether or not structures are involved and on the proposed construction time available. In most cases, it will be found that settlements are acceptable; in some they will not be and it then becomes necessary to refine the estimates. The next stage of more sophisticated investigation and design then comes into effect.

A very approximate rule for this third stage is that if the predicted settlement is larger than 200 mm over a period exceeding two years, and no structures are to pass through the embankment, further investigation is needed. If there are structures, this criterion should be more stringent. It is suggested that if the predicted settlement exceeds 100 mm over two years, further investigation is required. The third-stage assessment may confirm initial estimates and the problem then becomes one of designing the works to allow for settlement.

Occasionally it becomes necessary to reduce the potential embankment settlement by compaction of the subsoil. This is expensive and should rarely be required and then only after extensive investigation and consideration of alternative solutions. Similarly, accelerating the rate of settlement could be necessary through the use of drainage systems, but these too are expensive and should be avoided if possible.

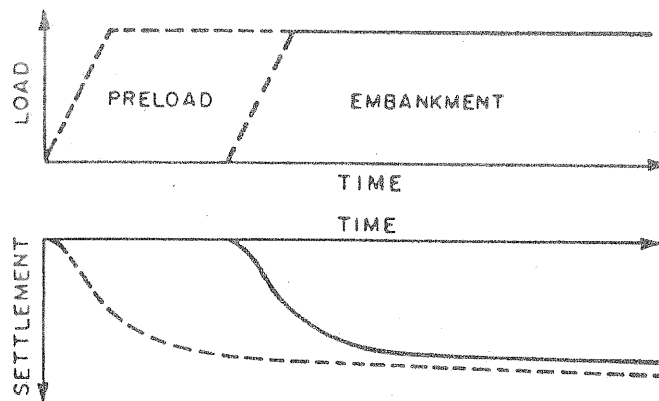
A more usual problem is that predicted settlements could be tolerated, if they occurred before pavement layer construction. Every effort should be made at the planning stage to programme construction to eliminate these problems. It could be advisable to arrange separate pre-loading or surcharging contracts, or, in other cases, to define the construction sequence to maximize the time for settlement before building of structures or final paving. Figure 5 illustrates the benefits of preloading and surcharging.

Preloading means constructing the embankment before it is required, so that time is available for settlement to occur. In some cases, it may be necessary to place fill in position without compaction and accept that it is temporarily stockpiled for subsequent removal and reconstruction. It will, in any case, almost certainly be necessary to remove the fill at structures to allow the building of these. Where possible, it is advantageous to preload not only up to the proposed structure position, but across it as well, since this will be much more effective in achieving the required early settlement.

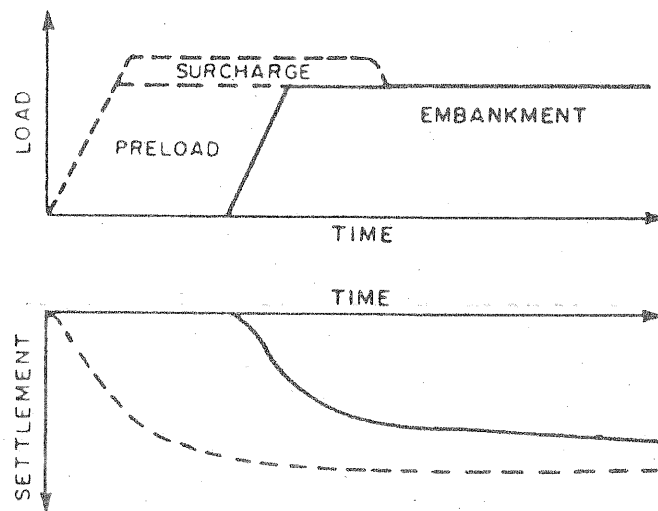
Surcharging may sometimes be beneficial. This is the temporary placing of a higher fill than will eventually be necessary, so that the extra load promotes more settlement in the time available than would have been the case with the designed height embankment.

It can be seen from Figure 5 that the warrant for requiring surcharge is primarily dependent on the rates of settlement after construction stops. The figure illustrates a general situation only and each case must be considered individually, which requires fairly sophisticated investigation and analysis. For embankments on soft soils it is probable that the optimum surcharge height will be in the range of about 20 to 30 per cent of the embankment height, lower will be barely worth the additional problems and higher will overstress the subsoil. The length of surcharge will depend on the particular purpose for which it is intended, but for the common case of minimising post-construction settlements at a structure, the surcharge length along the embankment from the structure position should be about twice the compressible soil depth. These are only very rough rules of thumb, not design recommendations and it is most strongly emphasised that although the potential benefits of surcharging are considerable, it is necessary to pay careful attention to the stability criteria. Decreasing the factor of safety by adding surcharge could give rise to large shear strains, or even stability failure, and nullify any advantages which may have been gained. Berms to increase stability, as described in the previous section, may then be required to avoid this problem.

It is almost inevitable that preloading and surcharging will cause double handling and even spoiling of material. It may necessitate separate contracts and will result in expenditure and a substantial time before the road is finalized. In addition it may even cause some public reaction to an apparently unnecessarily long construction period. In many cases, however, the benefits of minimizing post-construction settlements far outweigh these disadvantages.



(a) PRELOADING ONLY



(b) PRELOADING AND SURCHARGING

FIGURE 5
PRELOADING AND SURCHARGING

Where large settlements are predicted, and this would only be expected over soft deposits, the concomitant large settlements of culverts through the embankments may require attention both to the structural design and the functional design, since the centre settlement will be about twice that at the portals. Each case has to be addressed on its merits, but it should be appreciated that where large settlements are anticipated the function of the culvert may be prejudiced unless appropriate measures are taken. These will be dependent on, inter alia, the construction programme and may include preloading, surcharging, construction of the culvert by excavating through the embankment after allowing time for settlement, precambering of the culvert or even enlarging the culvert to allow for differential settlement.

The above discussion of acceptable settlement primarily concerns embankments over soft deposits. Large settlements may also occur at embankments over side slopes and gulleys. Such embankments may be high, so that, although the subsoil may be relatively firm, the large pressures cause significant settlements. The resulting problem is exacerbated because the cross-fall, and thus height differential, causes much larger settlements on the down slope side.

It is tentatively suggested that more stringent criteria for acceptable settlement should be applied for settlements on side slopes, viz if the predicted settlement from the preliminary and second stages of investigation and analysis exceed 100 mm for any time period, it is necessary to proceed to a further design stage.

4.4 SENSITIVITY ANALYSES

Although sensitivity and probabilistic analyses would usually be considered to be amongst the more sophisticated approaches utilized by experienced geotechnical engineers, some discussion of these techniques is relevant, because it provides guidance on investigation requirements.

Sensitivity analysis is merely the repetition of the calculations with variations of the parameters, to check the influence of changes in these parameters on the final outcome.

For simple stability analysis using undrained shear strengths, the factor of safety is practically a linear function of the embankment height divided by the shear strength. It is self-evident that errors in selecting the appropriate shear strength have a directly proportionate influence on the assessed factor of safety. Confidence in the latter can only be based on a reliable estimate of shear strengths. In general, this is best achieved, not by a few sophisticated measurements, but by good modelling of the strata and a number of simple strength tests.

More complex effective stress stability analyses for both the pore-pressure dependent and independent cases, require more involved sensitivity analyses. In these cases the variables are the pore pressure parameters and the shear strength, with the latter consisting of both apparent cohesion and friction angle components. Figure 6 is an example of the result of a sensitivity analysis for a particular embankment. It is emphasized that the detailed results cannot be generally applied; they do, however, demonstrate valid general trends.

It is clear that the factor of safety is extremely dependent on the angle of friction and on apparent cohesion. The former, however, has a fairly small range for a large variety of soils and can be readily determined; the latter practically always should be assumed to be zero.

The analysis is therefore most sensitive to variations in the pore pressure ratio, r_u . It is this which is most difficult to assess for all but the most simple slope-stability analyses. This again emphasizes the necessity of realistically modelling the geology and the most probable moisture conditions.

A problem that can occur quite frequently in southern Africa, however, and that perhaps would not be included in the usual sensitivity analyses, is that of large changes in undrained shear strengths of partially saturated in-situ subsoils. These may be stiff at the time of investigation, but will almost certainly become wetter and weaker under an embankment. In these cases it is necessary either to measure undrained shear strengths of the wetted soils (and this is difficult because assessment of a realistic final moisture content is difficult) or to analyze the embankment on the basis of the effective stress parameters, which is preferable.

Sensitivity analyses for settlement are simple because compressibility is the only variable of any significance. However, as opposed to stability analyses which will usually be confined to one or two very similar analytical methods, there are numerous settlement equations based on a variety of different methods of assessing compressibility. The sensitivity analyses should include not only comparisons of settlements using one particular method with variations in the compressibility, but also comparisons using different estimation methods based on different and varying input parameters. In short, it is recommended that a number of estimation methods be used and no excessive reliance be placed on any one apparently sophisticated analysis.

Probabilistic slope-stability analyses are beyond the scope of this document. It may be noted that an essential element of such analyses is the taking into account of the natural variability of the soil parameters and expressing these in terms of either normal, or more complex distributions. Most road engineers are already familiar with the application of probability concepts to the control of earthworks and pavement construction, so the advantages of these methods will be readily appreciated.

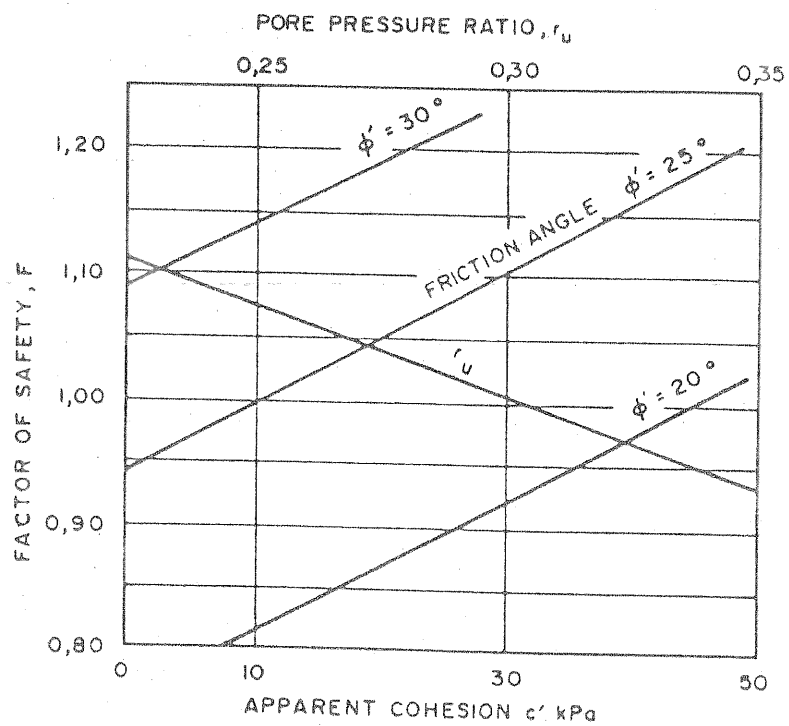


FIGURE 6
RELATIONSHIP BETWEEN FACTOR OF SAFETY F ,
 c' , ϕ' AND r_u

5 EMBANKMENT ANALYSES

5.1 INTRODUCTION

It has already been emphasized that embankment design is a process consisting of a number of stages from investigation, testing, analysis, decision-making and construction, to performance monitoring. In this process it is possible to overemphasize the role of analysis at the expense of common sense or engineering judgement. Nevertheless, frequently some analysis, albeit simple, is necessary. The following sections deal with simple analytical methods for stability and settlement estimates. The purpose of these analyses is to establish, with reasonable confidence, the severity of the potential problem and thus to permit a decision to be made on the necessity, or otherwise, for further specialist geotechnical input.

5.2 STABILITY ANALYSES

All embankments settle, but very few have stability problems. Despite this, it is still necessary to check the potential stability of many embankments. The overall process can conveniently be divided into initial and second stages, of increasing levels of sophistication, both for measuring subsoil parameters and for subsequent analyses.

5.2.1 Initial stability assessment

In the case of most road embankments where stability is a potential problem, subsoil strength is much more important than embankment strength. For the purpose of initial assessments of stability, the strength component of the fill can be conservatively ignored and the assessment based on subsoil description. Ranges of shear strengths for the standardized soil descriptions are given in Appendix C. It has been found that with experience, the appropriate soil description, such as soft, firm or stiff, for a particular stratum can be readily designated, hence a shear strength can be ascribed.

It is essential to model the stratification of the subsoils under the embankment realistically. However, for the initial assessment it is usually assumed that the worst subsoil conditions encountered continue throughout the subsoil.

Very few embankments founded on sands have failed. Embankments on clays require most attention. This does not imply that it is only soft alluvial clays that should be investigated; since many embankment problems that occur, are at the less obvious situations on side slopes, or gulleys, where worse loading conditions and far worse water conditions may be encountered.

For embankments on clay where undrained shear strength is the appropriate choice of soil parameter, and hence of analytical method, the following rule, which allows a conservative factor of safety, may be used:

$$H_{\max} = \frac{c_u}{8}$$

where $H_{\max}$ = maximum allowable height of embankment, metres
 c_u = undrained shear strength of subsoil, kPa

For example, if a 10 m high embankment is to be constructed over a layer of clay, described as a stiff material, for which the table in Appendix C assesses the shear strength as about 100 kPa, then:

$$\begin{aligned} H &= \frac{c_u}{8} \\ &= 12 \text{ m} \end{aligned}$$

Therefore the embankment would be considered as satisfactory.

However, if the clay was also described as fissured and partly saturated, the overall post-construction strength could be somewhat less than the initial intact strength (see 4.4). Therefore, the allowable height of 12 m does not provide sufficient margin over the proposed height of 10 m.

A further stage of investigation would then be necessary to determine the shear strength with more reliability, after first ascertaining through the use of a less conservative model and analysis, that the situation is marginal.

The two most common ways of checking stability are either by using charts, or by using computer programs. These are generally both based on the same analytical methods and the choice is largely governed by convenience. Whichever method is adopted, it is important to carry out sufficient trials so that an idea can be formed of the sensitivity of the analysis to the different input parameters.

Many programs are available eg SLOP4⁶, PC STABL5, PC SLOPE, STABR, and the charts of Hoek and Bray⁷ and Pilot and Moreau⁸. The latter are particularly useful for the u -dependent case of an embankment on flat soft deposits and have the advantage that the influence of berms on stability can be accommodated.

An example of the Pilot and Moreau charts is given in Figure 7 and described below.

The parameters required for the chart are the geometry, the strength and density of the embankment material, and the shear strength of the subsoil. If it is conservatively assumed that the embankment consists of cohesionless material, ie $c_u = 0$ with an angle of friction of 34° and that the embankment slope is 1:1.5, (ie slope = 2/3 in the chart notation) and the density of the embankment material γ is 20 kN/m^3 then using the charts consists of first selecting the correct group of charts, ie embankment with cohesionless material, $c/c_u = 0$ and slope 2/3 and then calculating N to select the appropriate chart within the group. For the example above:

$$\begin{aligned} N &= \frac{c_u}{\gamma H} \\ &= \frac{100}{20 \times 10} \\ &= 0.5 \end{aligned}$$

If the depth of the clay layer is assumed to be large, ie $D/H = 1$, then the factor of safety (F) can be read off the chart for $\phi = 34^\circ$, ie $F = 2.57$.

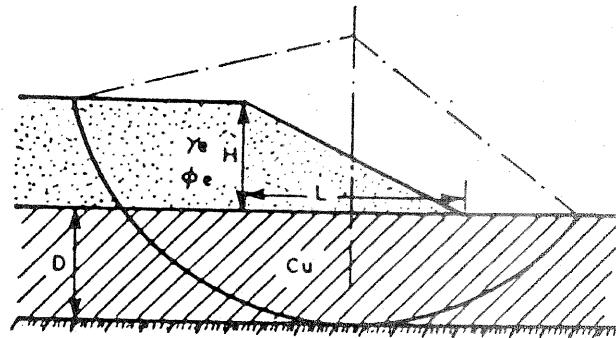
A simple sensitivity analysis can very easily be conducted. If, for example, the undrained shear strength was estimated to average 100 kPa, but varied by 20 kPa about this average, the appropriate chart would be that for $N = 0.4$ and the factor of safety would then decrease to 2.10.

Similarly, the charts could be used to check the influence of changing the embankment slopes and also the benefits of using berms.

The initial assessment considered, primarily, the case of embankments on level clay deposits. These are the simpler cases and are so obviously a problem that they would be checked.

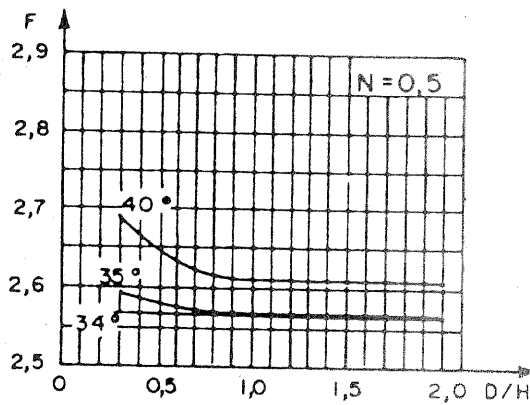
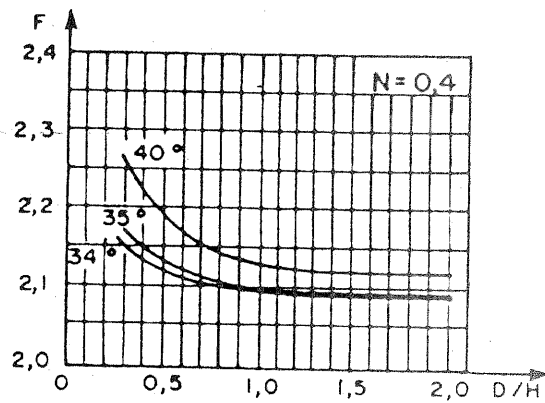
Problems of embankment stability on gulleys and side slopes are more complex and are not amenable to chart solutions.

In these cases, simple circular-arc stability analyses can be carried out using undrained shear strength concepts.



$$\text{SLOPE} = \frac{H}{L}$$

$$N = \frac{C_u}{\gamma_e H}$$



TYPICAL CHARTS FOR SLOPE = $\frac{2}{3}$ AND COHESIONLESS EMBANKMENT $C_e = 0$

FIGURE 7
STABILITY CHARTS - PILOT AND MOREAU

If these indicate a factor of safety less than 2.0, it will be necessary to carry out analyses based on effective stress concepts.

For this purpose, the Mohr-Coulomb shear strength model is used, together with the Terzaghi principle of effective stress. These result in shear strength being defined by the following equation:

$$\tau = c' + (\sigma - u)\tan \phi'$$

where c' = effective cohesion
 σ = normal pressure
 u = pore pressure
 ϕ' = angle of friction

For normally consolidated clays, c' is taken as zero and ϕ' varies within a relatively small range. The major contribution to the strength is therefore dependent on $(\sigma - u)$, the effective normal stress. The normal stress, σ , at the failure surface is defined by the geometry of the situation and is simply the pressure due to the overlying material. The pore pressure, u , may or may not be dependent on the pressure due to the embankment and, as previously described (section 4.2), embankments may therefore be considered as falling into two categories; either those at which the pore pressure is a dependent variable, or those at which it is independent.

Where the pore pressure is a dependent variable (See section 4.2.1) it is often convenient in calculations to use the concept of pore pressure ratio, r_u , which is defined as follows:

$$r_u = \frac{u}{\gamma H}$$

where u = pore pressure at a point
 H = height of material above the point
 γ = density of material

In other words, the pore pressure is a function of the overburden pressure.

(a) Pore pressure (u) dependent

The stability criteria for pore pressure dependent cases are given in section 4.2.1 and typical examples are embankments over saturated clays. As shown in Figure 3, as the embankment increases in height, both σ and u increase at the same rate if no strains and no drainage occur. Therefore there is no gain in

strength under these conditions, but there is an increase in the stresses tending to cause failure. The strength is then the undrained shear strength, c_u , and is generally used in the preliminary assessment of embankment stability. It is the strength measured by in-situ tests such as the vane shear, or the CPT, and by laboratory quick undrained triaxial or shear box tests, since none of these allow sufficient time for drainage of the pore pressures to occur. If this strength parameter is used for analysis, it implies that the embankment is built at such a rate that no drainage occurs. This is sometimes called the $\phi = 0$ condition.

It is clear that if dissipation of pore pressure occurs, either because the permeability is sufficiently high, or because the rate of construction is deliberately slowed to allow for this, u decreases relative to σ and there is a gain of strength which is proportional to the rate of consolidation. Figure 3 illustrates these conditions and shows how the factor of safety changes with pore pressure dissipation.

In some cases, this forms the basis of the construction technique in which the rate of construction is tailored to the rate of dissipation of pore pressure.

Alternatively, if the rate of construction cannot be delayed, the natural rate of drainage can be increased by inserting artificial drains into the slower consolidating layers. It may be noted that the least safe period for embankments, where u is a dependent variable, is during construction.

(b) Pore pressure (u)-independent

The stability criteria for these cases are given in section 4.2.2 and typical examples are embankments at gulleys or side slopes, where there are external sources of water. The subsoil may be either saturated, or partly saturated, during construction and allow construction without difficulty. However, pore pressures may increase after construction, either because the embankment has impeded the natural drainage, or even because the embankment effectively acts as a dam. This means that the effective shear strength decreases, because the normal pressure σ remains the same and the pore pressure u increases. These are extremely difficult situations to predict, because prediction of the possible pore pressure, u , is a matter of judgement, and depends on the geology and climate and even on the vegetation, as well as on the drainage of the embankment and subsoil. Figure 4 illustrates a typical example where the pore pressure increases due to a buildup of moisture under an embankment.

5.2.2 Second-stage stability assessment

The initial stage consisted of assessment of shear strength, and hence stability, from the visual description of the subsoil, using either the simple equation ($H_{max} = c_u/8$) or appropriate stability charts or computer programs.

The second stage uses the same methods of stability analysis, but relies on obtaining more reliable estimates of shear strength. These would be generally based on in-situ tests. Brief descriptions of shear strength assessment from insitu tests are given in the following sections.

- (a) SPT: Undrained shear strengths may be estimated from the SPT N values. The table in Appendix C shows the usual correlation and this may be written as:

$$c_u \text{ kPa} = 10 N$$

It must be emphasized that this is a very crude estimate and rates as little more reliable than the visual assessment by an experienced person.

- (b) CPT: Although this test was originally devised for the investigation of sands, it has been found that good correlations between cone pressures, q_c , and undrained shear strength, c_u , can be obtained. These can be expressed by the following equation:

$$c_u = \frac{q_c - \gamma D}{N_q}$$

where N_q = Cone Factor and varies from 10 to 30

γ = density of the subsoil

D = depth of subsoil

For recent alluvial deposits N_q may be taken as 14 for the standard Dutch type mechanical cone. This should be increased to 16 if a continuous electrical cone system is used. The value of the γD term is usually small compared with q_c and can be ignored. It is conservatively recommended that for preliminary analyses, the following relationship be used:

$$c_u = \frac{q_c}{20}$$

For overconsolidated deposits, N_q is higher and a value of 25 should be taken for preliminary analysis.

It should be noted that a significant advantage of the CPT is that a continuous, or near continuous, record of strength against depth is obtained.

This may be particularly relevant in many stability problems because a distribution of shear strength is obtained. This may be valuable for sensitivity or probabilistic analyses. A further advantage is that the CPTs, particularly the recently developed types which measure pore pressures (CPTU), allow a good definition of the nature and stratification of the soil.

The CPT also allows for the detection of thin layers. At many alluvial deposits, the presence of even relatively thin sandy layers may permit drainage of excess pore pressures. This may be sufficient to prevent stability problems which could occur otherwise. On the other hand, the presence of a thin layer of much softer clay could control stability.

- (c) Vane Shear: The vane shear test takes many forms. It may be performed at the bottom of a borehole by using long extension rods; the vane may be pushed directly into the subsoil in a similar manner to the CPT, or tests may be carried out using hand-held vanes in trial pits or in large diameter auger holes. There are various degrees of sophistication of the vane-rotation and torque-measuring devices. All these factors must be taken into account when assessing the reliability of the results.

The interpretation of the results is generally accepted as being straightforward, in that shear strength is calculated from the measured torque and the geometry of the vane, ie:

$$c_u = \frac{T}{\pi d^2 (h/2 + d/6)}$$

where T = torque

d = diameter of vane

h = height of vane

Typically vanes are made so that the height is twice the diameter, ie $h = 2d$, so the above equation can be approximated to:

$$c_u = \frac{T}{4d^3}$$

Experience has shown that for various reasons the shear strength measured by the vane shear test in plastic clays may give values which are too high. It is therefore recommended that the correction given by Bjerrum⁹ be used (Figure 8).

- (d) Pocket Penetrometer: These should be seen as confirmatory of visual descriptions, rather than as independent measurements of shear strengths. The test is generally carried out in pits or auger holes which have been relatively disturbed by excavation and it is therefore necessary to clean the surface to expose a zone that is neither dried out, nor disturbed. Many tests can be carried out in a short time and a good indication of the variability obtained.

It should be noted that most pocket penetrometers are calibrated using the unconfined compressive strength, q_u , which is double the value of the undrained shear strength, c_u , used for preliminary stability assessments.

- (e) Dynamic Cone Penetrometer - DCP: This equipment is familiar to road engineers since it is used for measuring in-situ CBRs and also for the assessment of earthworks densities. Since the California Bearing Ratio is itself a measurement of the bearing capacity of a circular plunger and hence of the shear strength, the CBR percentage value can be related to shear strength by the following:

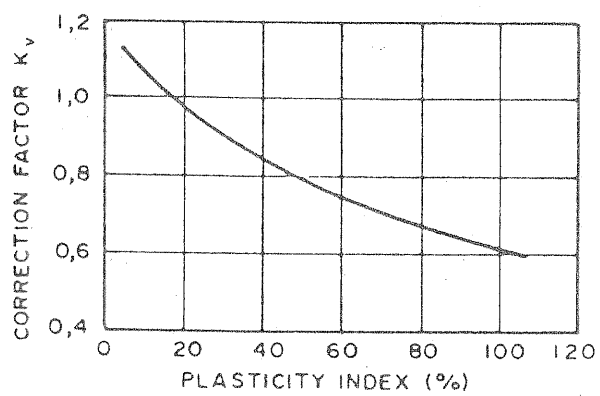
$$c_u \text{ kPa} = 12.5 \text{ CBR per cent}$$

Descriptions of the DCP test and the assessment of the CBR are given by Kleyn¹⁰. Figure 9 shows the relationship between DCP number (DN) and CBR.

- (f) Laboratory Tests: Provided good undisturbed samples have been obtained, it may be convenient and appropriate to carry out simple laboratory shear strength tests.

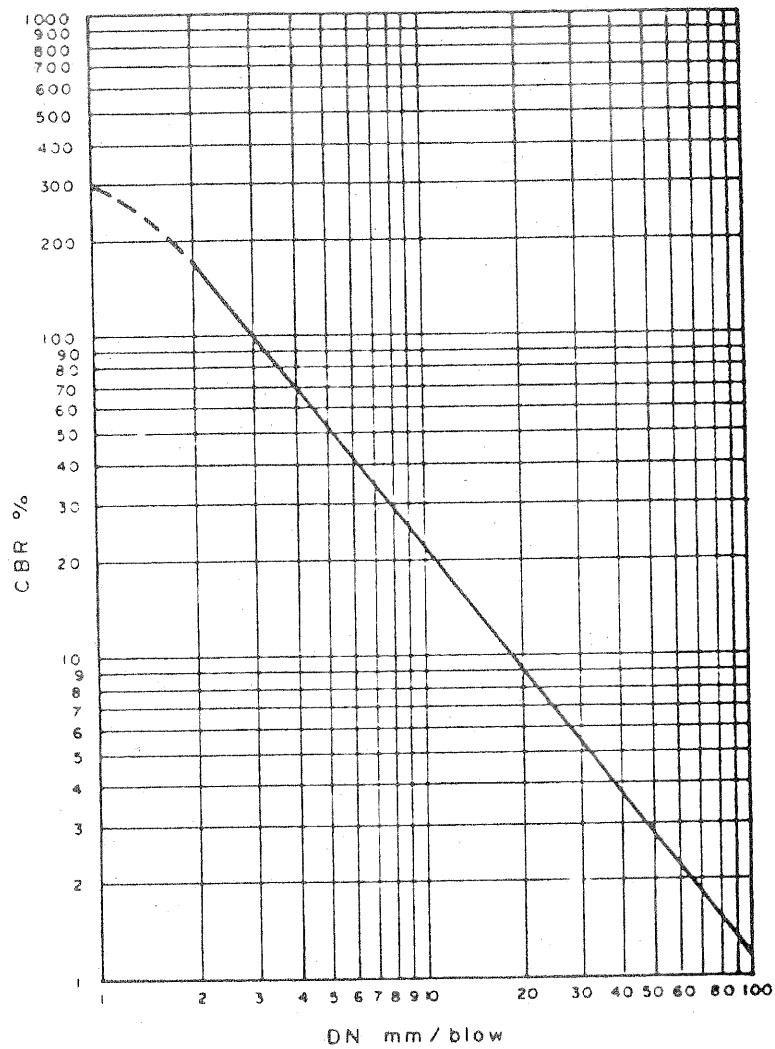
At this second stage assessment, quick undrained shear box or triaxial tests are sufficient. It is essential to check that the samples tested have not dried out. This should be done by checking moisture contents at the time of sampling and at the time of testing.

It is strongly emphasized that cognizance must be taken of moisture conditions at the time of strength assessment. If the subsoil is saturated, the visual assessment or in-situ tests will give appropriate strength values. If, however, the material is dry, the measured strength may be considerably higher than the strength that will eventually occur under the embankment.



$$S_u (\text{FIELD}) = K_v S_u (\text{VANE})$$

FIGURE 8
VANE STRENGTH CORRECTION FACTOR
FOR SOFT CLAYS



$$\text{CBR (\%)} = 410 (\text{DN})^{-1.27} \text{ FOR } \text{DN} > 2$$

FIGURE 9
EVALUATION OF CALIFORNIA BEARING RATIO (CBR)
FROM DYNAMIC CONE PENETROMETER (DCP)

5.2.3 Further stability assessment

It is strongly recommended that any further stability assessment, which implies more sophisticated testing and analysis, should only be conducted by experienced geotechnical engineers. No guidance is therefore given on these methods, other than to emphasize yet again that the most important, and difficult part of the process is realistically modelling the actual conditions.

By this it is meant that proper delineation of the strata is essential for all embankment assessments, and at situations where the pore pressure is an independent variable, a realistic assessment of the future moisture conditions is vital.

5.3 SETTLEMENT ESTIMATION

The estimation of embankment settlement should be seen as establishing, with a high degree of confidence, the order of magnitude of the predicted settlement, viz 10 mm, 100 mm or 1 m. In other words, in the majority of cases it is the development of realistic subsoil-embankment models that is essential, not the application of sophisticated analytical methods using the results of complex laboratory tests.

The methods of estimating settlements are discussed in the following subsections in the order in which they would normally be applied.

5.3.1 Initial settlement estimation

In accordance with the AIMED design sequence described previously, the first action is to define the purpose of the settlement assessment. This means it is necessary to appreciate the required reliability of the estimate for a particular embankment. To a large extent this is dependent on the magnitude of the settlement, but it is nevertheless usually possible to form some judgement of the other considerations.

It may be known that the construction programme will result in the pavement layers being built a year or more after embankment construction. If there are no structures through the particular embankment, settlement will not be a problem, provided that whatever does take place, occurs within a year or so. A modest quality estimate will then be sufficient. Alternatively, the embankment may be across an alluvial deposit and may encompass a piled river bridge. These conditions, coupled with a constrained construction programme, would make it obvious that high quality estimates of the amount and rate of settlement are essential.

The second part of the design process is to consider the information available. At the earliest phase of the investigation this will be the engineering geology report. In some cases, this will provide sufficient information to judge whether or not settlement is likely to be a problem.

In the majority of cases, the results of the simplest field investigation, consisting of trial pits and boreholes, possibly with some in-situ tests, such as standard penetration tests, will be sufficient to judge, without formal analysis, the order of magnitude of the settlements. This will determine whether further investigation is required. Simple standard material descriptions such as loose, medium dense and dense, or soft, firm and stiff often provide a satisfactory estimate of compressibility and hence of settlement. If, however, there is doubt, a basic settlement equation may be used as an aid in judgement, viz:

$$\delta = \frac{\sigma \times H}{E}$$

where δ = settlement

σ = increase in stress due to embankment

H = thickness of compressible stratum

E = modulus = $1/m_v$

m_v = coefficient of compressibility

For example, if the subsoil is described as medium dense sand and is 10 m thick underlying a 10 m high embankment. E may be about 20 MPa, σ is about 200 kPa since the embankment density will be about 20 kN/m³, therefore:

$$\begin{aligned}\delta &= \frac{200 \times 10}{20 \times 1000} \\ &= 100 \text{ mm}\end{aligned}$$

It would then be necessary to evaluate the consequences of this estimated settlement.

If no structures pass through the embankment, and it can be constructed some time before the pavement is built, the situation is almost certainly satisfactory, leading to the decision that no further investigation is required.

On the other hand, if a large box culvert passed through the fill, pressure under the culvert would be less than that under the adjacent fill, and differential settlements between the embankment and the culvert would be expected. In addition, there would be differential settlements between the ends of the culvert and the centre. Although the estimated settlement in this case is the same, ie 100 mm, the evaluation would be that insufficient information was available to be confident that the estimate was reliable enough.

The decision should then be made to measure subsoil compressibility parameters with a higher degree of reliability. With a compressible subsoil thickness of only 5 m, the estimated settlement would have been 50 mm, and it is unlikely that any further investigation for a sandy subsoil would have to be carried out.

A stiff clay could give about the same modulus value, resulting in the same settlement. The evaluation would then have to consider the rate of settlement, to assess how much would occur after pavement completion.

Estimation of amount of settlement is more reliable than estimation of settlement rate. The initial investigation should concentrate on assessing the amount. If this shows that a real problem exists, it will be necessary to estimate settlement rate. This is calculated by using the subsoil coefficient of consolidation, c_v . First estimates of c_v can be made from the subsoil descriptions, and typical values are given in Appendix C.

The time for 90 per cent of settlement to occur can be estimated using the following equation:

$$t = \frac{d^2}{c_v}$$

t = time in years

d = drainage path length, metres

c_v = coefficient of consolidation, m^2/year

The drainage path length is the distance along which drainage, or dissipation, of the excess pore pressures in the compressible layer can occur. Figure 10 illustrates typical drainage path situations.

If, for example, the thickness of the clay layer is 10 m, and double drainage occurs, then d is 5 m. If the coefficient of consolidation is estimated from the data in the appendices as $5 \text{ m}^2/\text{year}$; then:

$$\begin{aligned} t &= \frac{5^2}{5} \\ &= 5 \text{ years} \end{aligned}$$

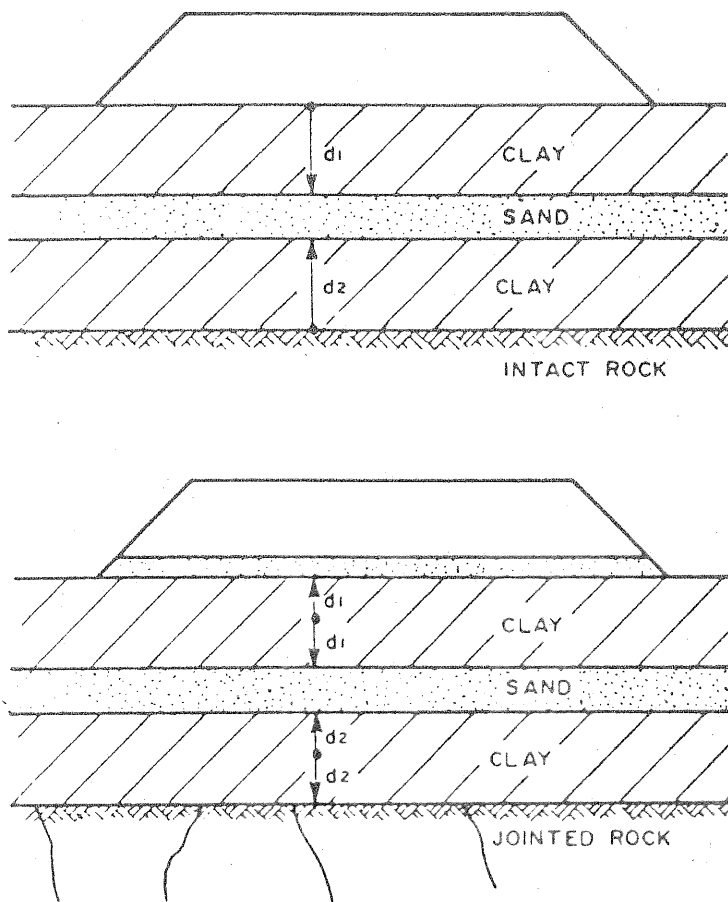


FIGURE 10
DRAINAGE PATHS DEFINITION

If single drainage is appropriate then d is 10 m, and the time for settlement is:

$$t = \frac{10^2}{5} \\ = 20 \text{ years}$$

It is immediately obvious that elaborate means of estimating the coefficient of consolidation are totally inappropriate if a reliable estimate of the drainage path length has not been obtained. The importance of understanding the engineering geology, and hence stratification, is emphasized again.

It should not, however, be construed from the above that a good estimate of the coefficient of consolidation is never required. There are many cases where the stratification of clay and sand layers can be readily established and modelled. If this is so, a reliable estimate of the coefficient of consolidation may be justified, as is illustrated below.

If, as in the previous example, it is assumed that the upper and lower boundaries of the 10 m thick clay are defined by sand and fractured rock, ie d is 5 m, the time for complete consolidation is 5 years if $c_v = 5 \text{ m}^2/\text{year}$. If, however, the clay had been described as having few fissures, the estimate of c_v might have been $1 \text{ m}^2/\text{year}$. The time for consolidation would then have been 25 years, which may or may not be acceptable, depending on the particular circumstances.

The example could be extended further by supposing that the clay layer was very fissured, with frequent sand lenses so that the estimate of the coefficient of consolidation was $10 \text{ m}^2/\text{year}$. In this case the time for complete consolidation would be 2.5 years, and construction time may be sufficient, say 2 years before paving. Remaining settlement after this time would then be negligible.

It may be necessary to estimate the time for intermediate proportions of settlement. For example, the time for half settlement, t_{50} , is given by the following simplified equation:

$$t_{50} = \frac{0.2 d^2}{c_v}$$

In the above example t_{50} therefore is 0.5 years, and only 50 mm settlement will occur after this period. This may be regarded as negligible and no further investigation will then be required.

It is emphasized that the above examples demonstrate initial estimates based on no more than reliable assessments of stratification, and on good visual descriptions of the subsoil. The equations given are

conservatively simplified versions of those conventionally used in more sophisticated analyses. They can be used during investigations and decisions taken in the field as to whether more detailed investigation is required.

Clearly, it is of considerable advantage if the engineer responsible for the investigation not only knows the details of the embankments, but also knows the proposed construction programme. It is then possible to direct the investigation to obtain specific information and also, with adequate planning, to adjust the construction programme, if necessary, to suit subsoil conditions. The financial implications of doing this may be considerable and the reliability of estimating the amount and rate of settlement will then have to be correspondingly high.

5.3.2 Second-stage settlement estimation

If, as a result of the initial assessment of settlement, it has been decided that further investigation is required, this will generally take the form of in-situ testing so that values of the subsoil moduli can be obtained and hence settlements estimated.

Reliable estimates of the subsoil moduli from in-situ testing can be obtained from:

Standard Penetration Tests - SPT

Cone Penetration Tests - CPT

- (a) SPT: The tables in the appendices, and the following equations, can be used to assess the order of magnitude of the moduli from the SPT-N value:

- (i) for sands

$$E = 530 (N + 15) \text{ kPa}$$

- (ii) for clayey sands

$$E = 350 (N + 5) \text{ kPa}$$

Alternatively:

- (iii) for sands

$$E = 1\,000 N$$

- (iv) for clayey sands

$$E = 500 N$$

- (v) for clays

$$E = 250 N$$

- (b) CPT: The moduli can be estimated from the CPT cone pressures, q_c . It is generally accepted that the CPTs give more reliable estimates than SPTs, particularly in soft materials, because the resolution of the measurement of q_c is much better than that of N .

(i) for sands and highly plastic clays

$$E = 1.5 q_c$$

(ii) for clayey sands

$$E = 3 q_c$$

(iii) for stiff clays

$$E = 6 q_c$$

Estimated settlements can be obtained using the moduli from the SPT and CPT values given above and the following methods of calculation.

- (a) Settlement from SPT: The same equation is used as that described in Section 5.3.1.

$$\delta = \frac{\sigma H}{E}$$

It should be noted that σ refers to the mean increase in stress due to the embankment load at the centre of the compressible layer.

Road embankments are generally wide compared with the thickness of underlying compressible strata. The reduction in stress due to depth is therefore not large for moderate thicknesses of compressible subsoil and can be conservatively ignored in preliminary estimates. If the thickness of subsoil is large, this simplification will be over-conservative. Any of the standard stress distribution charts or diagrams should then be used to estimate stresses.

If the same parameters are used as for the embankment in Section 5.3.1 (viz 10 m high at a density of 20 kN/m³ with an underlying clay layer 10 m thick, giving a consolidation settlement of 100 mm), then a similar, but narrow embankment, with a half width less than the thickness of subsoil, would induce a mean stress in the subsoil of only 70 per cent of that at the surface with the result:

$$\begin{aligned} \delta &= 0.7 \times 100 \\ &= 70 \text{ mm} \end{aligned}$$

- (b) Settlement from CPT: Settlements of embankments using CPT results are generally calculated using the modified Buisman-De Beer method.

The settlement δ is given by the following equation:

$$\delta = \frac{1,5 H \sigma_{vo} \log_{10} (\sigma_{vo} + \delta\sigma)}{q_c}$$

where

H = thickness of compressible stratum

σ_{vo} = initial vertical effective stress at centre of stratum

$\delta\sigma$ = increase in vertical effective stress at centre of stratum

q_c = cone pressure at centre of stratum

Since the CPT gives continuous readings of cone pressure, it is common to subdivide the subsoil on the basis of the magnitude of cone pressures, as well as on geological description of the strata. The calculation is then carried out in a tabular form and may also include a factor, α , which is dependent on soil type.

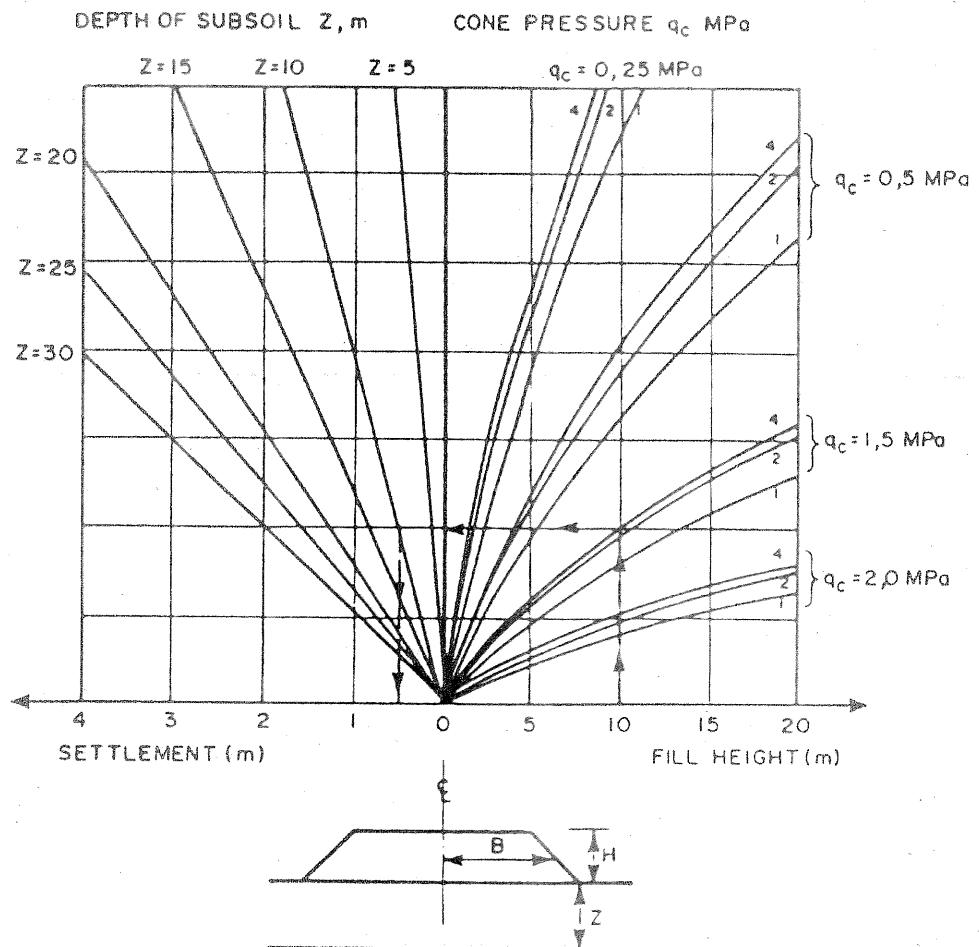
An alternative method is the use of a settlement chart (Figure 11) which is simply a graphical representation of the Buisman-de Beer equation given above.

Settlements can be estimated from the chart with sufficient accuracy for most purposes by dividing the subsoil into one or two layers and estimating the average cone pressures within each layer.

For example, if in the example given in Section 5.3.1 of a 10 m high embankment of density 20 kN/m³, the 10 m thick medium dense sand had an average q_c value of 1,5 MPa and the embankment was 20 m wide, ie $2B/z = 2$, then the chart would give a settlement of about 0,5 m. Note that this chart estimate has already taken into account the decrease in stress with depth.

It is often found that the Buisman-De Beer method gives somewhat higher embankment settlements than other methods.

It is recommended that if good visual descriptions, SPTs and CPTs are available, all the methods given above should be used to obtain an average value, particularly if the estimated settlement exceeds that considered to be acceptable for the embankment.



EXAMPLE : $H=10$ m ; $B=20$; $Z=10$ m ; $q_c = 1,5$ MPa

∴ SETTLEMENT = 0,60 m

NOTE : CONE PRESSURE LINES 4, 2 AND 1 INDICATE $\frac{Z}{B}$ VALUES

FIGURE II
CPT SETTLEMENT CHART

5.3.3 Further settlement estimation

The second-stage settlement analysis methods, based on visual assessment and in-situ tests, allow adequate analyses and evaluations for the majority of potential embankment settlement problems. There will be a few embankments that will require further investigation to measure the appropriate subsoil parameters, and more sophisticated analyses to estimate settlement.

In some of these cases more sophisticated in-situ testing, such as the piezometer cone penetration test (CPTU) and the self-boring pressuremeter (SBPT) will be needed.

In other cases, undisturbed sampling, followed by laboratory testing, will be required.

It is not the purpose of this document to describe either this testing, or subsequent analyses, since these are covered in the geotechnical literature on the subject.

Should the stage be reached where more sophisticated testing methods are required, it would be necessary to utilize geotechnical expertise.

In view of the general approach suggested in this document, it is likely that such expertise would have been involved at the planning stage of the investigation, since the planning is itself a function requiring considerable experience.

In whichever manner the investigation is organized, it should never result in the geotechnical advisor being presented with a few borehole logs and consolidation test results, and asked to predict settlements, with the proviso that no further investigation would be permitted.

6 CONCLUSION

This document sets out a recommended method for the assessment of stability and settlement of embankments. The emphasis is placed on the development of well-planned investigations and the use of simple techniques and analyses to obtain realistic engineering results.

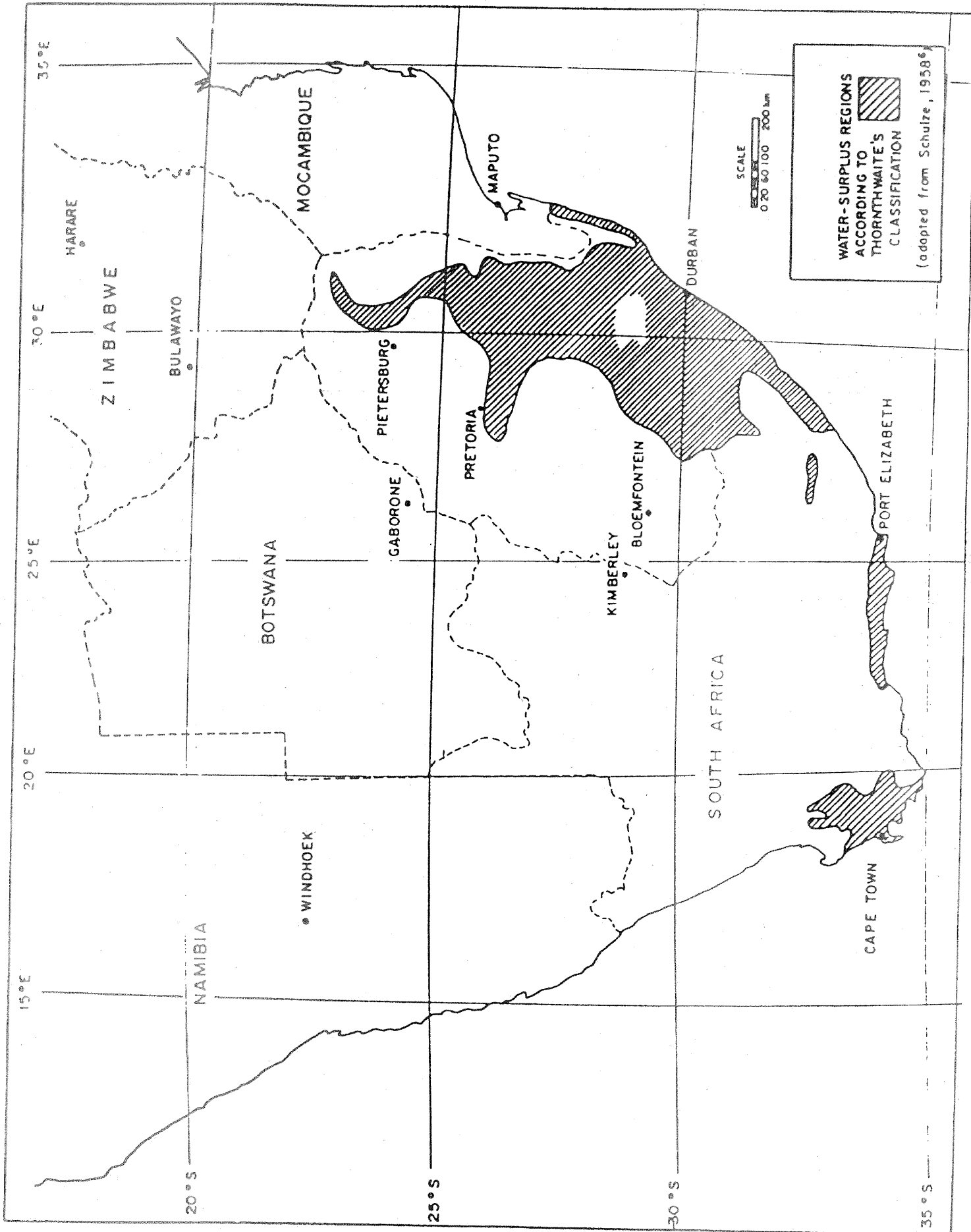
As a corollary, the recognition of problems which should fall within the category of those requiring specialist input is stressed. These will almost certainly be relatively few, but their early definition is important if appropriate solutions are to be found.

Finally, it is strongly recommended that the design of embankments will benefit from the discipline of some formal design procedures which clearly define responsibilities and recommended construction methods.

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APPENDIX A: Water surplus regions of southern Africa



APPENDIX B

METHODS OF SITE INVESTIGATION

The more common investigation techniques are briefly described in the following sections.

- (a) **Geophysical:** The most usual geophysical technique is the hammer seismic refraction method. For roadworks this is primarily used for the examination of the depth to rock and excavation characteristics for cuttings. It is equally applicable for estimating the depths of compressible strata, and particularly if correlated with boreholes or probes. An extremely valuable use is in detecting anomalies such as old infilled channels, faults or intrusions.

One limitation of the seismic technique is that the relatively small energy input from a hammer blow precludes reliable investigation at depths in excess of about 10 m. This limitation can, to a certain extent, be overcome by using more sophisticated equipment or by using explosives. It is strongly recommended that the work should only be carried out by personnel with professional geophysical training.

Resistivity methods may also be used. These require specialist input, since, although in common with seismic surveys the field operation techniques appear to be simple, the interpretation can be difficult. The method is very useful for detecting anomalies such as intrusions and delineating the extent of marked changes in materials and for detecting water.

- (b) **Sample Holes:** In many cases sample holes, excavated either with a backactor or a large diameter auger, can provide sufficient information for the first-stage investigation. These techniques allow direct examination of the subsoil in a relatively undisturbed state.

It must be remembered, however, that there are stringent safety regulations concerning men working in excavations. Working in an excavation deeper than 1.5 m is prohibited unless the excavation is supported. Exemptions are permitted in the case of large diameter augered holes, provided that the relevant Code of Practice is followed.

More use could probably be made of sample holes, particularly for embankments at gulleys or on side slopes, where trenches may provide all the information that is necessary and, additionally, show the soil profile over a significant length.

Backactor and large diameter auger holes are usually about a quarter of the cost of small diameter boreholes in terms of rate per metre. Undisturbed and disturbed samples can be taken, and in-situ testing of the relatively undisturbed material is possible.

This testing can consist of shear vane, or hand held penetrometer tests, to assess the shear strength, or plate jacking tests to measure strength and compressibility.

Small plate jacking tests, usually carried out by crossjacking 200 mm diameter plates in sample pits or auger holes, has recently become more common. The technique is relatively inexpensive, suitable for a wide range of materials and provides immediate and easily interpreted results.

A similar but less sophisticated class of test is the Dynamic Cone Penetrometer (DCP) which was developed for pavement testing. From the results, California Bearing Ratios (CBR) can be deduced and from these reasonable estimates of undrained shear strength and compressibility can be made.

The disadvantages of sample pits are that the depth is limited if backactors are used. If auger holes are favoured, the site access for large machines may be a limiting factor. In both cases working below the water table is virtually impossible.

- (c) Penetration Testing: There are many penetration testing techniques, but the most commonly used in southern Africa is variously called Dutch Probing, Deep Sounding, Quasi Static Penetration Testing, or to use the recently adopted international designation, Cone Penetration Testing (CPT). The method consists of pushing a standard size cone (35 mm diameter) into the subsoil at a constant rate of penetration (20 mm/sec), and measuring the load required to achieve this. The compressibility and strength or density of subsoils are deduced from the cone pressures.

A recent development of the CPT is the measurement of pore water pressures during penetration. This allows indirect identification of the subsoil and estimates to be made of the in-situ consolidation characteristics. This technique is usually designated CPTU to indicate that measurements of the pore pressure (u) are made simultaneously with cone pressures.

The advantage of cone penetration testing is that a practically continuous record is obtained, usually at a cost of about a quarter of that of conventional boreholes. Nevertheless, it would rarely be recommended that CPTs should be carried out without at least some boreholes being put down to obtain direct subsoil identification.

CPT is used primarily on saturated alluvial deposits or at tailings dams. There is little experience available on interpretation of CPT results in unsaturated materials. The latter would, in any case, probably be best investigated by direct examination of sample holes.

- (d) **Boreholes:** Small diameter boreholes are often considered to be the standard investigation technique.

There is a tendency to overspecify the use of boreholes, where simple sample holes following engineering geological mapping would have sufficed. Boreholes are relatively expensive - the cost of drilling and sampling a hole about 10 m deep, which is about one day's work, is approximately four times that of large diameter augering or backactor trial holes. It follows that good quality drilling must be achieved if the cost is to be justified. A variety of contractual systems have been tried to produce high quality borehole results at a reasonable cost. The most common, competitive tendering on the basis of rate per metre, may lead to diametrically opposed objectives for the driller and the geotechnical engineer. An alternative system is the hiring of drilling machines and operators on a day-work basis. There is much to commend this method. Whichever system is used, close supervision by experienced staff is essential, if the investigation is to be conducted efficiently and economically.

There are many variations in the methods of making boreholes and only a brief mention is made of the more usual methods.

The techniques can be divided into soft-ground boring and rock drilling. The first can be carried out with a number of different machines, whereas the latter is done primarily by rotary coring using diamond bits.

At the initial stage of the investigation, it is likely that undisturbed sampling will not be carried out and the size of the boreholes will be determined by the casing to be used during the diamond drilling. This casing is usually NX size and is large enough to allow sampling in the overburden, using Shelby samplers. These are thin-walled metal tubes, often about 50 mm in diameter, which are pushed into the subsoil. Larger diameter casing through the overburden would be preferable, since large diameter undisturbed samples - say 75 mm - could then be taken. These are much more satisfactory for laboratory testing.

Standard Penetration Tests (SPTs) which consist of driving a special sampler by using a standard hammer, are also commonly carried out. In this test the number of blows, N , that are required to drive the sampler a distance of 300 mm into the subsoil, are recorded. Numerous correlations are

available between N and the compressibility and strength of the subsoil. Some of these are given in the appendices. The SPT results are largely dependent on the technique used. It is most important that the test should be carried out in accordance with an approved method.

If sophisticated sampling is required to provide high quality undisturbed samples for laboratory testing, then larger diameter boreholes are necessary. One of the largest sizes used allows piston samples of up to 150 mm in diameter to be taken. This is not common, other than where extensive deposits of soft clays occur.

Occasionally it may be necessary to obtain a better estimate of the elastic compressibility than is available from SPTs or CPTs. This is unusual for embankment design, since elastic settlement occurs during construction of the embankment and causes no problems other than at interfaces with structures. If better estimates are required, these can be obtained by in-situ testing such as the Menard pressuremeter or the self-boring pressuremeter.

Settlement is usually a much more significant problem if it is long-term or consolidation-dependent, since deformations may occur after construction has been completed. This characteristic can be measured by laboratory consolidation tests as well as by field tests, such as in-situ permeability measurements.

There are two important points to note regarding laboratory consolidation tests:

- (i) The samples are very small compared with the total volume of soil under the embankment. There is a tendency, particularly with alluvial deposits, to select material which can be sampled most easily, with the result that the more clayey samples are tested. This can give an unnecessarily pessimistic picture of the deposit, particularly with regard to the predicted consolidation rate.
- (ii) It is necessary to take account of sample disturbance and stress history in the interpretation of laboratory test results. Preconsolidation is possible, even in recent alluvial deposits, and failure to correct for this could lead to very large overestimation of settlement.

In some cases, embankment-stability problems may arise on materials that cannot be tested with any of the techniques suitable for soils listed above. Failure could occur through an unfavourable joint system in a rock and diamond drilling may be required. Although this may be useful and necessary, it must be borne in mind that a very small sample is obtained, compared with the overall subsoil mass. It is probably the gouge material in the joints which gives rise to problems, and this is difficult to recover in the drilling. In order to appreciate the problems, much broader interpretation of the strata on an engineering geological basis is needed.

Where diamond drilling is carried out emphasis must be on very good core recovery, therefore the size used should be at least NX.

The table below gives a guide as to the costs of typical site investigation field work. It must be noted that costs are at 1991 levels, and also that there may be a considerable variation between projects. The times reflect an overall average assessment of what may be expected for the duration of a project.

Costs and time for site investigation

Method	Relative cost R/m	Metres/day
Sample holes	40	50
Large diameter auger	40	100
CPT	40 - 80	50
Soft ground boring	120	10
Diamond Drilling	300	6

- Notes:
- (i) Diamond drilling taken as base at R300/m at 1991 prices.
 - (ii) Large variations may occur due to quantities in contract, access problems and distances between holes.
 - (iii) Soft-ground boring rate includes sampling and SPT.
 - (iv) Range for CPT reflects sophistication from simple mechanical to electrical with pore pressure measurements.
 - (v) Costs do not include supervision, logging or in-situ testing in sample holes or in large diameter auger holes.

APPENDIX C

SOIL PARAMETERS

Numerous tables are published that give guidance on strength and compressibility parameters. Such tables are extremely useful but it is prudent to give some warning on their use. Perhaps the first warning is that where tables relate a parameter to a physical description of the material, it is absolutely essential that the description must be in accordance with the recommended method and must be carried out by a competent person.

Similarly, if relationships derived from in-situ testing, such as SPT or CPT, are used to obtain soil parameters, then these tests must be conducted in the standard manner.

Finally, it is essential to relate the state of the material at the time of observation, or test, with what it may be at some future time, ie in the wet season, or due to gradual moisture content increases to an equilibrium under an embankment.

It is also emphasized that values in the tables must be seen as not significantly better than estimates of the order of magnitude. This does not indicate that decisions cannot be based on them, but only that some conservatism is appropriate.

In many instances it is possible to have in-situ test results, as well as good visual descriptions. If these correlate, then more confidence can be placed in any estimates based on the parameters.

(a) Shear strength-Clays

Description	SPT N	CPT q_c MPa	Undrained shear strength c_u kPa
Very soft	2	0,3	20
Soft	2-4	0,3 - 0,6	20-40
Firm	4-8	0,6 - 1,4	40-80
Stiff	8-15	1,4 - 3,0	80-150

(b) Compressibility

(i) All materials

Soil	Modulus E MPa			
Granular	loose	medium	dense	
Sandy gravel	30-80	80-100	100-200	
Sand	10-30	80-50	50-80	
Non-plastic silt	8-12	12-20	20-30	
Cohesive	soft	plastic	hard	very hard
Silty sand	5-8	10-15	15-20	20-40
Silt	3-6	6-10	10-15	15-30
Lean clay	2-5	5-8	8-12	12-20
Fat clay	1-4	4-7	7-12	12-30

Note that the consistency descriptions in this table are not the same as those in Tables (ii) and (iii). The latter descriptions are internationally accepted.

(ii) Sands

Description	SPT N	CPT q _c MPa	Modulus E MPa
Very loose	4	2	4
Loose	4-10	2-5	4-10
Medium dense	10-30	5-15	10-30
Dense	30-50	15-20	30-40
Very dense	50 >	20 >	40 >

(iii) Clays

Description	SPT N	CPT q_c MPa	Coefficient of Compressibility m_v m ² /MN
Very soft	2	0,3	1,5
Soft	2-4	0,3-0,6	1,5-0,3
Firm	4-8	0,6-1,4	0,3-0,1
Stiff	8-15	1,4-3,0	0,1-0,05

(c) Coefficient of consolidation

Subsoil description	Permeability	Coefficient of consolidation c_v m ² /year
Intact clays	very low	0,1-1,0
Fissured clay, alluvial silty clay	low	1,0-10
Clayey sand, sandy	medium	10-100
Silty sands	high	100-1 000
Sands	very high	1 000 >