Ron Munro

DEALING WITH BEARING CAPACITY PROBLEMS ON LOW VOLUME ROADS CONSTRUCTED ON PEAT

Including case histories from roads projects within the ROADEX Partner Districts

THIS PROJECT IS BEING PART- FINANCED
BY THE EUROPEAN UNION
European Regional Development Fund

ROADEX II
NORTHERN PERIPHERY

DEALING WITH BEARING CAPACITY PROBLEMS ON LOW VOLUME ROADS CONSTRUCTED ON PEAT

INCLUDING CASE HISTORIES FROM ROADS PROJECTS WITHIN THE ROADEX PARTNER DISTRICTS

December 2004

Ron Munro

The Highland Council

PREFACE

This Report is a final report of the Phase 2 'Understanding and Analysis' section of the EU ROADEX II Project and aims to give an insight into the state of the art of road construction and road maintenance for roads constructed across peat in the Northern Periphery of Europe.

It will concentrate particularly on the current practices for road construction over peatlands in the four specific Partner Areas of the EU Roadex Project i.e. Troms County in Norway, the Districts of Keski-Suomi and Lapland in Finland, the Region Norr of Sweden and the Highland Area of Scotland.

In addition, where appropriate, the report will also refer to relevant peat related experiences being carried out elsewhere in the world where these are considered apposite to the report.

The report was written following discussions and meetings with engineers across the Partner areas. The author would like to thank Aarno Valkeisenmäki, Hannu Keralampi, Timo Saarenketo, Jorma Immonen and Martti Eerola from Finland, Karl Melby, Stein Stokkebø, Tor Erik Frydenlund, Roald Aabøe, Kjetil Vollan, Even Øiseth and Inge Hoff from Norway, Peter Carlsten, Johan Ullberg and Gunnar Zweifel from Sweden and Alistair Gilchrist, Frank MacCulloch, Hugh Mackay, Ken Wiseman, Richard Evans, Douglas Bremner and Willie Watt from Scotland for their help and input into the report.

The report was written by Ron Munro of The Highland Council Transport, Environmental & Community Services. Mika Pyhähuhta of Laboratorio Uleåborg designed the graphic layout and Garry Smith of The Highland Council prepared many of the graphic images. Virpi Halttu, Teen Mackay and Janet Mackintosh assisted in the production of the report by typing, drafting and editing the various versions.

Finally the author would like to thank the ROADEX II Steering Committee for its encouragement and guidance in this work.

Roadex II Lead Partner: The Highland Council, Transport, Environmental & Community Service, HQ, Glenurquhart Road, Inverness IV3 5NX Scotland, Project co-ordinator: Mr. Richard Evans.

CONTENTS

ABSTRACT

Dealing with bearing capacity problems on low volume roads constructed on peat

The Roadex II project is an EU funded trans-national technical co-operation between the northern European roads districts of Finland, Norway, Sweden and Scotland.

The goal of sub-project 2.5, ‘Roads over Peat’ was to gather together existing and past practices for dealing with bearing capacity problems for road construction on peat in the Partner roads districts. As part of the exercise interviews were held with practising roads and geotechnical engineers in the Partner areas to gain as full an insight as possible into their current thinking. The result of the research is a snapshot of the Partner area practices in dealing with bearing capacity problems in roads constructed over peat that covers such topics as the classification and engineering properties of peat, local field survey methods, testing, design considerations, risk management, methods of construction supplemented by local case studies.

The paper will present the results of the research carried out within the Partner areas and will give a ‘snapshot in time’ of local thinking within the Northern Periphery for dealing with bearing capacity problems on low volume roads constructed on peat.

KEY WORDS: Roads, Peat, Construction, Roadex, Survey, Geotechnical.

1 The Roadex II Project

1.1 The Roadex II Project

The ROADEX II Project is a co-operation aimed at developing ways for interactive and innovative road management of low traffic volume roads. The high level aim of the Project is to strengthen and reinforce the first ROADEX technical exchange co-operation across the Northern Periphery region that took place from 1998 to 2001.

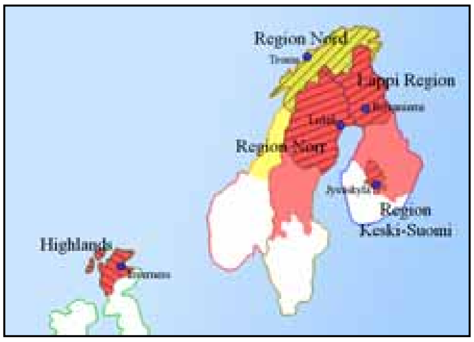


Figure 1. Northern Periphery Area and Roadex II partners.

Within this overall strategy the particular objective of ROADEX II was to develop ways for interactive and innovative road condition management of low traffic volume roads integrating the needs of local industry, Roads Districts and society at large. This goal involved developing models, assessment methods and tools to improve local Road District road condition management taking into account the views of road users.

The partners within the Project comprised public road administrations, forestry organizations, forest companies and haulage organizations from the following regions in the Northern Periphery of Europe: The Scottish Highlands and the Western Isles, the northern regions of Norway and Sweden, and the regions of Central Finland and Lapland in Finland. The Roadex cooperation maintains a web site at www.roadex.org.

The Roadex II project was conducted in three phases during 2002-2005: (I) Problem identification, (II) Understanding and Analysis, and (III) Innovation and Testing.

The goal for the phase I work was to provide a road user's perspective of the condition of the road networks in each test area. These areas were chosen to be representative of each partner road district. The survey focused on road users´ transportation needs and opinions on the general condition and trend of the road network in summer and winter, traffic safety issues, types of problem encountered with transportation industries as well as opinions regarding the level of cooperation with local road authorities.

Phase II focused mainly on the technical details of the shared road condition problems across the areas. These problems, identified in the Roadex I project, included the permanent deformation of low volume roads, material treatment techniques, drainage problems, spring thaw weakening and its management, and managing road sections resting on peat. The phase also included a subproject that focused on the problems that would arise if low volume roads were allowed to continue to deteriorate. A final subproject evaluated current environmental guidelines for low volume roads across the partner districts and produced a common environmental checklist.

The final phase of the Project, Phase III, will focus on preparing proposals for a basis of new low volume road condition management policies suitable for Northern Periphery areas. It will also summarize the finding of the phase I and II results in the form of new structural innovations and best practise methods. Finally phase III will briefly review the possibilities that modern information technology can provide for low volume road condition management.

1.2 Dealing with bearing capacity problems on low volume roads constructed on peat

This report presents the results of one of the surveys carried out in Phase II during 2002-2004. The goal for Sub-project 2_5, “Dealing with bearing capacity problems on low volume roads constructed on peat”, was to gather together existing and past practices for dealing with bearing capacity problems for road construction on peat in the Partner roads districts. As part of the exercise interviews were held with practising roads and geotechnical engineers in the Partner areas to gain as full an insight as possible into their current thinking. The result of the research is a snapshot of the Partner area practices in dealing with bearing capacity problems in roads constructed over peat that covers such topics as the classification and engineering properties of peat, local field survey methods, testing, design considerations, risk management, methods of construction supplemented by local case studies.

The paper presents the results of the research carried out within the Partner areas and gives a ‘snapshot in time’ of local thinking within the Northern Periphery for dealing with bearing capacity problems on low volume roads constructed on peat.

The Roadex II project has carried out extensive data collection from the project test sites and reports, and detailed analyses will be published in scientific symposiums and publications. Data is also available on the Roadex web pages.

2 Background

2.1 Introduction

Road construction over peat presents great challenges to the intending road builder not only in the landscapes and terrain that have to be crossed but also in the management of the engineering properties of peat; high water content, high compressibility and low strength.

The roads engineer has to overcome these engineering obstacles and considerations of low bearing capacity and excessive settlement in order to be able to construct safe, stable and serviceable road embankments.

2.2 History

Roads over peat have been proven to have been around since c.4,000 B.C. The earliest of these appear to have been simple rafted tracks, probably constructed as rights of way between primitive communities or as accesses to local peat workings. An example of one dating from 4,000 B.C., recently unearthed in a blanket bog in southern Ireland, is shown below for interest.

Figure 2. Source: B. Raftery "Trackways through Time, Archaeological Investigations on Irish Bog Roads".

A great deal has happened of course since these first tracks were laid done in 4000 BC. Traffic flows have certainly increased, carriageway loadings have intensified and road users expectations of comfort and standards of carriageways have risen also.

Construction technology has advanced too to meet these challenges and many innovative methods of construction have been developed for peatland projects over the intervening years such that it is still very possible to build acceptable roads over peat at reasonable cost to meet modern expectations. In Scotland probably the most noteworthy of these were the very successful reinforced concrete slab roads of the A9 "Great North Road" improvements through Inverness-shire in the 1920's. These peat roads were still in satisfactory condition serving modern traffic flows when they were uncovered and removed in the late 1970's to permit faster alignments to be constructed.

2.3 The present situation

Road construction by its nature tends to be a 'conservative' science, particularly in the planning, design and construction of major public roads. These highways are substantial elements of social infrastructure, involving significant sums of public money, and are required to give good service for lengthy periods with increasing use.

Engineers dealing with these roads rightly shy clear of construction risk and where possible usually opt for the safer and more conservative forms of construction whereby the peat is totally removed and replaced with sound road foundation material. This is of course an expensive solution and a primary user of scarce natural resources from local environments, and only really appropriate for the construction of national high speed roads.

For lower classes of road, an awareness of the "usability of peat" as an engineering foundation is more common, particularly in those geotechnical communities in countries with large peatland areas, and especially in the Northern Periphery where the "green issues" of earthworks construction are becoming increasingly important within the public domain.

These issues, such as the proliferation of quarries, loss of agricultural land and recreational space and the amenity of landscape, have gained a much higher public profile whilst budgets for roads have reduced commensurately. The old budgets that permitted the building of new roads as a first choice solution to infrastructural problems are no longer provided and as a consequence engineers are being encouraged to turn their focus instead to maximizing the strength, potential and capacity of the existing road networks.

The 4 partner roads districts of the ROADEX II project can be seen to be typical examples of local roads authorities undergoing this change of focus, and how this new reality of roads funding is being managed. Their available roads budgets, like many other roads authorities, are spent by necessity on keeping their main roads networks serviceable, at the expense of their secondary and minor roads networks. As a result of this the condition of main roads are being held constant as budgets decline whilst older, lower classes of roads become progressively worse.

These minor rural road networks are important. The journeys to markets of the major indigenous timber, fish and quarry industries of the Partner areas invariably start on these networks and regularly involve passage over low strength public and private roads before accessing the main strategic routes. Not surprisingly therefore the conditions of these minor rural road networks are critical for business, community and personal life. In particular the preservation or improvement of the bearing capacity of the carriageways on these routes is paramount for local business users.

For these reasons an increasing number of geotechnical engineers are actively pursuing cost effective and innovative solutions for improving the bearing capacity of minor rural roads constructed over peatlands. Through their efforts peat is no longer dismissed as an engineering material.

3 Peat

3.1 Introduction

The word ‘peatland’ can be defined very simply as ‘an area of land where peat is found’ but this definition takes no account of the great range of interpretations used for ‘peatland’, and ‘peat’, across the world. In an effort to clarify terminology for the purposes of this report the words ‘peatlands’, ‘mires’, ‘fens’ and ‘bogs’, will be used in the manner that they are defined in the Irish Environment and Heritage Service “Peatlands” website www.peatlandsni.gov.uk.

The definitions set out in the “Peatlands” website are:

- **Peatland** - an area with a naturally accumulated peat layer at the surface
- **Mire** - a peatland where peat is currently forming and accumulating
- **Fen** - a peatland which receives its water and nutrients from the soil, rock and groundwater as well as from rain and/or snow
- **Bog** - a peatland which receives its water solely from rain and/or snow falling on its surface

The section that follows summarises of the formation of peatlands, mires, fens and bogs to give an understanding of some of the characteristics and engineering properties that arise as a consequence.

3.2 Formation of peat

‘Peat’ forms in a landscape when the natural decay processes fail to keep up with the amount of vegetation being produced. This usually happens on waterlogged land starved of oxygen, such as is found in mires, fens and bogs where the lack of oxygen prevents natural micro-organisms from decomposing the dead plant material. Where these conditions occur the dying vegetation does not decay at the end of the growing season as normal but instead accumulates year on year as a peat layer. Peat forms slowly in this way, involving an accumulation of organic material in water, and taking approximately 10 years for 1cm of peat to form.

The most important feature in this simple scenario is water and in particular the water balance within the peat. For a peatland to survive, the water balance cannot be negative, ie the water input must keep up with the water loss.

But peat is not the only soil to have an organic content. Organic soils can occur in many ways and in many landscapes. The organic material can be deposited insitu, like peat, by dying vegetation and it can also be washed into place by inundation, flood, rivers, etc. These latter soils that have had their organic material washed into them inevitably have a higher mineral content due to the minerals carried by the incoming water flows. These high mineral content soils are usually considered to be outwith the classification of a peatland and for simplicity this report will concentrate mainly on the mires, fens and bogs that have formed insitu through dead material collapsing in place year on year.

3.2.1 "The Wetland Succession"

Mires, fens and bogs normally arise through a process that is commonly called the 'wetland succession' as explained by Hobbs in 1986. This wetland succession has 3 stages set out by Hobbs:

- The '**rheotrophic**' stage in which the mire develops in a body of water such as a lake, pool or flooded basin and gets its nutrients through the feeding streams, ground water and seepage. Initially the mire process starts with inorganic sedimentation, such as silts and clays, but this becomes increasingly more organic as the detritus from plant communities builds up in the basin floor. The eventual product of this build up is a marsh-like mass known as a '**fen**';

- The '**transitional**' stage characterized by a steady growth of the fen upwards and out of the standing water and into a 'raised bog'. During this stage the bog is still influenced by local water levels but is beginning to rely on rainwater for sustenance;

- The '**ombrotrophic**' stage where the mire has grown fully out of the standing water and out of the influence of the local water table. At this stage the bog relies totally on rainwater for its survival and holds its own survival water reservoir within its mass above the local groundwater table. This type of bog is termed a '**raised bog**' for obvious reasons and is generally acidic in character.

An ombrotrophic bog can also form directly on a suitable surface under favourable wet climatic conditions without the need for standing water. These bogs are known as '**blanket bogs**' and are discussed within the body of the report.

3.2.2 Mires, Fens and Bogs in the Northern Periphery

Mires, fens and bogs are normally classified according to their topographical and hydrological features, otherwise known as their 'morphology' and within the Northern Periphery three common types of peatland morphology are common:

- palsa mires
- fens (or 'aapa' mires in Finnish)
- raised bogs including blanket bogs

The distribution of these mires, fens and bogs across the Northern Periphery is shown in the map below prepared by Succow & Jeschte in 1990. 'Zone II' indicates the range of palsa mires, 'Zone III' indicates the range of fens or aapa mires and 'Zone IV' indicates the normal range of raised bogs.

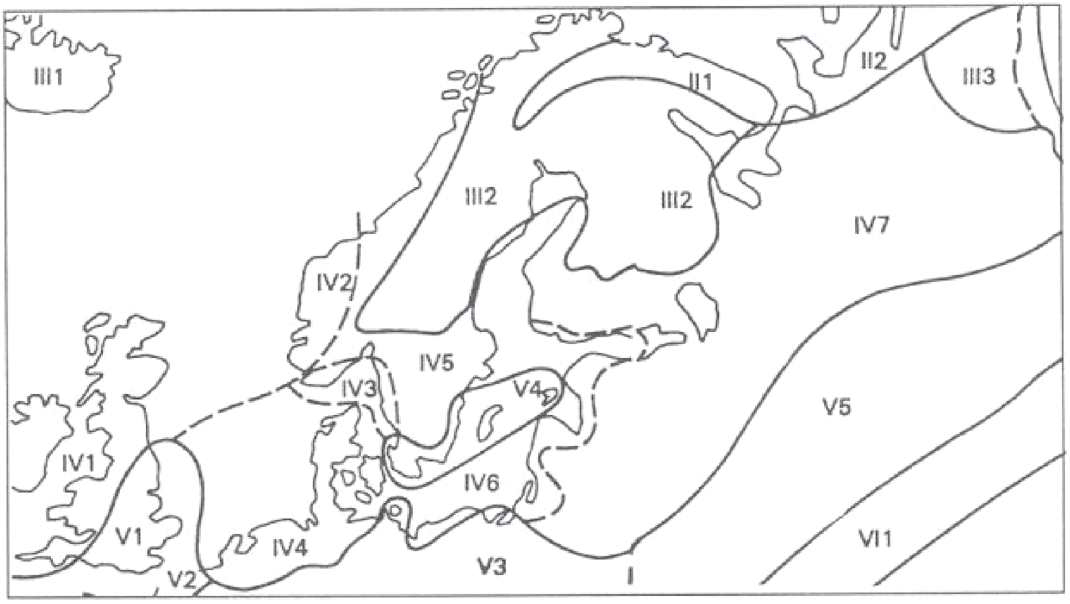


Figure 3. Mire zones across northern Europe, Succow & Jeschte 1990. ('Zone II' indicates the range of palsa mires, 'Zone III' indicates the range of fens or aapa mires and 'Zone IV' indicates the normal range of raised bogs.)

Missing from this classification is the intermediate 'transition bog' phase between an aapa mire and a raised bog. This phase has been omitted from the report for simplicity.

Palsa Mires

Palsa mires form in the sub-artic areas of the Northern Periphery where conditions exist that permit frozen ice cores to develop and grow within the peat mass insulated from the summer thaw. The water supply for the survival of these bogs comes from the annual snow melt waters and this gives rise to the significant mineral contents in the peat layer formed. Palsa mires are characterised by a 'palsa mound' as below:

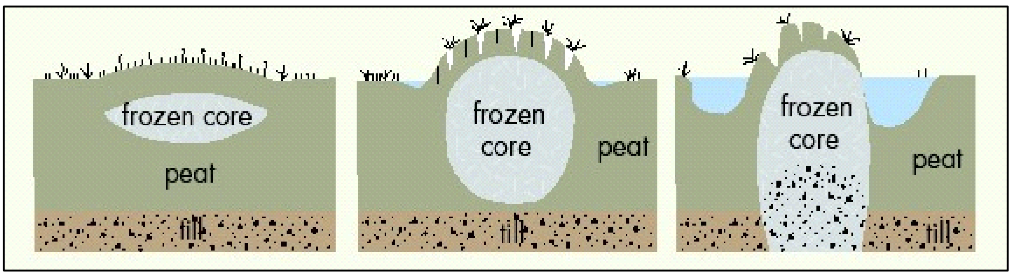


Figure 4. Formation of a palsa within a peat mass, Source: "Peatlands in Finland, Finnish Peatland Society.

In the first stage (left diagram), a frozen core developes within the peat mass. This frozen core grows and the 'palsa' starts to rise up through the mire (middle diagram). Finally, the outer peat layer of the palsa begins to dry and crack (right diagram) at which point the palsa breaks down and collapses.

Palsa mires are limited to the far north of Finland and adjacent areas and are a separate consideration in their on right. Road engineering in such areas requires particular geotechnical solutions and specialised pavement structures to deal with the extremes of environment ("Arctic road construction – problems and modern solutions"). Road construction over palsa mires is not discussed in this report.

Aapa Mires

Aapa mires (or fens) extend over most of the Nordic countries as can be seen in Figure 3. These fens receive the bulk of their survival water from snow melt or from seepage flows from adjacent wet areas and are mostly flat or slightly concave such that the centre of the bog does not rise above the surrounding mineral ground (a major difference from raised bogs.)

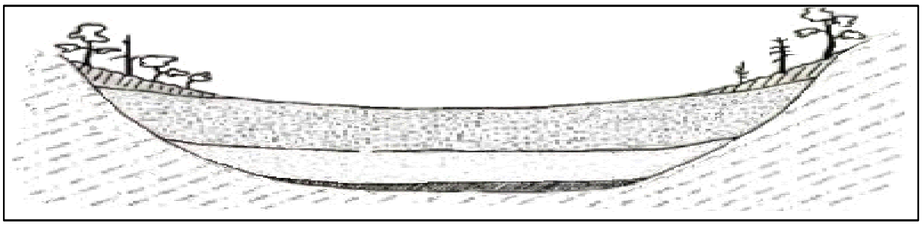


Figure 5. Cross-section through an aapa mire.
Source: "The World of Plants, Aapa Mires", North Ostrobothnia Regional Centre

Since aapa mires are usually flooded in late spring by snow melt, the characteristic hummocks of raised bogs are not formed. Instead narrow ridges are created in the form of a ladder across the line of the flow of water with wide pools between the 'rungs' leading to their common name of 'string bogs'. Such aapa mires can extend over wide areas. They tend to have a higher mineral content, lower water content and be more 'humified' (Von Post) than raised bogs.

Raised bogs

'Raised bogs' get their sustenance from rainwater. They are found in the cold and cool temperate regions (like the Northern Periphery) and thrive on a moderate rainfall with a cold winter. They are usually formed where lakes or basins have accumulated sufficient organic material to create a dome of peat and typically the bog surface and water table of a raised bog are higher than those of the surrounding vegetation.

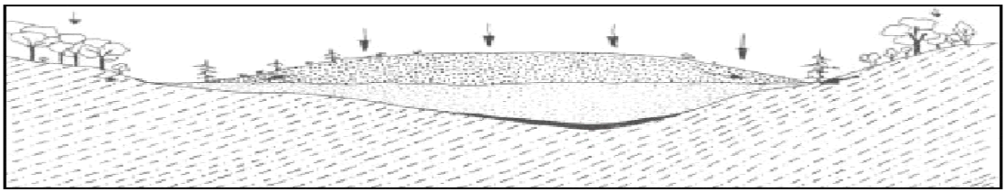


Figure 6. Cross-section through a raised bog (arrows indicate the flow of water through the bog) Source: "Conserving mires in the European Union".

Blanket bogs

'Blanket bogs' get their name from the peat layer covering the landscape like a blanket. They generally occur on flat or undulating soils with poor surface drainage in oceanic climates with heavy rainfall. The distribution map to the right shows the distribution of blanket bog in the Northern Periphery. Typically these bogs need a climate with an annual rainfall of at least 1000mm and a minimum of 160 rainy days per year. Their characteristics are similar to raised bogs with a more even and less deep peat layer over the landscape.

Figure 7. Distribution of Blanket Bog.
Source: "Conserving mires in the European Union"

An interesting feature of blanket bogs is their groundwater flow. For a blanket bog to survive it must have rain, as this brings most of its minerals and nutrients, but unlike other types of bogs there can be an appreciable groundwater flow and this has to be catered for in any engineering works.

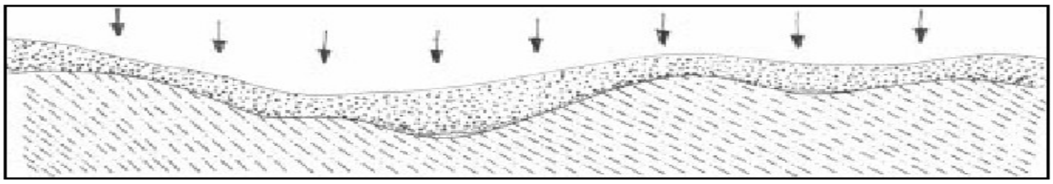


Figure 8. Cross-section through a blanket bog, (arrows indicate the rainfall on to the bog)
Source: "Conserving mires in the European Union".

3.3 The Classification of Peat

At the initial engineering level peats may be considered to fall into three basic groups as proposed by N W Radforth in 1969. These are 'amorphous-granular peat', 'coarse fibrous peat' and 'fine fibrous peat'. Within this simple hierarchy 17 sub-groups can be listed:

Table 1. Classification of Peat Structure, Source, NW Radforth, Muskeg Engineering Handbook, 1969

Predominant characteristic	Category	Name
Amorphous-granular	1	Amorphous-granular peat
	2	Non-woody, fine fibrous peat
	3	Amorphous-granular peat containing non woody fine fibres
	4	Amorphous-granular peat containing woody fine fibres
	5	Peat, predominantly amorphous-granular containing non woody fine fibres, held in a woody, fine-fibrous framework
	6	Peat,predominantly amorphous-granular containing woody fine fibres, held in a woody, coarse-fibrous framework
	7	Alternate layering of non-woody, fine-fibrous peat and amorphous-granular peat containing non woody fine fibres
Fine-fibrous	8	Non-woody, fine-fibrous peat containing a mound of coarse fibres
	9	Woody, fine-fibrous peat held in a woody, coarse fibrous framework
	10	Woody particles held in non-woody, fine-fibrous peat
	11	Woody and non-woody particles held in fine-fibrous peat
Coarse-fibrous	12	Woody, coarse fibrous peat
	13	Coarse fibres criss-crossing fine-fibrous peat
	14	Non-woody and woody, fine-fibrous peat held in a coarse fibrous framework
	15	Woody mesh of fibres and particles enclosing amorphous-granular peat containing fine fibres
	16	Woody, coarse-fibrous peat containing scattered woody chunks
	17	Mesh of closely applied logs and roots enclosing woody coarse-fibrous peat with woody chunks

The 'amorphous-granular' peats comprise those peats with a high colloidal mineral component which tend to hold the contained water in an adsorbed state around the grain structure. The two fibrous peat types, 'fine-fibrous' and coarse-fibrous', are woodier and hold most of their water within the peat mass as free water. These categories reflect the morphology of the parent peat deposit and give rise to many of the important engineering properties.

This simple basic classification can be further subdivided by physical description or 'degree of humification' based on the hand squeezing of samples as set out by Von Post in 1925 shown in Table 2.

Table 2. Degree of Humification of Peat. Source: L Von Post & E Granlund, 1926.

Degree of Humification	Identification Guide
H1	Completely unconverted and mud-free peat which when pressed in the hand only gives off clear water. Plant remains are still easily indentifiable.
H2	Practically unconverted and mud-free peat which when pressed in the hand gives off almost clear colourless water. Plant remains are still easily indentifiable
H3	Very slightly decomposed or very slightly muddy peat which when pressed in the hand gives off marked muddy water, but no peat substance passes through the fingers. The pressed residue is thickish. Plant remains have lost some of their identifiable features.
H4	Slightly decomposed or slightly muddy peat which when pressed in the hand gives off marked muddy water. The pressed residue is thick. Plant remains have lost more of their identifiable features.
H5	Moderately decomposed or muddy peat. Growth structure evident but slightly obliterated. Some amorphous peat substance passes through the fingers when pressed but mostly muddy water. The pressed residue is very thick.
H6	Moderately decomposed or very muddy peat with indistinct growth structure. When pressed approximately 1/3 of the peat substance passes through the fingers. The remainder extremely thick but with more obvious growth structure than in the case of unpressed peat.
H7	Fairly well decomposed or markedly muddy peat but the growth structure can just be seen. When pressed about half the peat substance passes through the fingers. If water is also released this is dark and peaty.
H8	Well decomposed or very muddy peat with very indistinct growth structure. When pressed about 2/3 of the peat substance passes through the fingers and at times a thick liquid. The remainder consists mainly of more resistant fibres and roots.
H9	Practically completely decomposed or mud-like peat in which almost no growth structure is evident. Almost all the peat substance passes through the fingers as a uniform paste when pressed
H10	Completely decomposed or mud peat where no growth structure can be seen. The entire peat substance passes through the fingers when pressed

3.4 Peat Characteristics & Properties

3.4.1 Introduction

It will be seen from the previous chapters that peat can be a highly variable material and the engineering properties of a peat deposit will be a consequence of the formation and morphology of the peat. At one end of the scale fibrous peats will have a visible plant structure with little humification almost resembling a mat at times. Amorphous peats, at the other end of the scale, will have a highly decayed structure with no vegetable fragments whatsoever in many ways resembling a clay.

This variability occurs throughout the deposit, both horizontally and vertically, again as a direct result of the deposit's morphology. Significant variation can happen within 10 metres horizontally and even less vertically and great care has to be taken as a result in selecting areas for sampling to ensure that as representative samples as possible can be made available for testing.

3.4.2 Peat characteristics and properties

The most distinctive characteristic of a virgin peat deposit is probably its high **water content** and many of the characteristics of peat of interest to the engineer as a foundation material result from this basic property. Water contents of peat in the Northern Periphery generally range from 500% to 2000% but can reach as high as 2,500% for some coarse fibrous peats. Water content values of less than 500% are usually an indicator of high mineral fractions within the peat sample.

The **ash content** (or non organic content) of a peat sample is the percentage of dry material that remains as ash after controlled combustion. Peat that has grown insitu (the subject of this report) normally has an ash content of somewhere between 2% and 20% of its insitu volume and this range of ash contents can be an indicator of this type of peat.

The **insitu bulk density** of a peat bog depends predominantly on its moisture content. Amorphous granular peats can have insitu undrained bulk densities of up to 1200 kg/m^3 whilst at the other end of the scale very woody fibrous peats can have insitu densties of as low as 600 kg/m^3.

The **dry density** of peat is also dependent on the natural moisture content and mineral content of the particular deposit. This density is an important characteristic for the engineer concerned with road construction over peat as it influences the behaviour of the peat under load. Dry densities of peat can typically vary between 60 kg/m^3 to 120 kg/m^3. Higher values are possible however where the deposit has a high mineral content.

The **specific gravity** of peat typically varies from 1.5 to 1.8 with the higher ranges again reflecting a higher mineral content.

The **void ratio** of peat varies with the type of peat and moisture content. As an example a peat with a moisture content of 1,000% is likely to have a void ratio of approximately 18. Void ratios as high as 25 can be found in fibrous peats and void ratios as low as 9 are possible for the denser amorphous granular peats. The void ratio of a particular peat bog normally tends to decrease with depth but as always there can be exceptions to this general rule.

The **permeability** of peat in the field is highly variable depending on its morphology and reduces dramatically when subjected to loading. The permeability of virgin peat usually ranges from 10^{-2} to 10^{-4} cm/sec but when loaded with a low embankment it can quickly reduce to 10^{-6} cm/sec and with a higher embankment construction to as low as 10^{-8} to 10^{-9} cm/sec.

Peat compresses significantly under embankment loads as the free water within the pores is squeezed out into the adjacent unloaded bog. As the load is applied the voids within the loaded peat reduce and the inter-colloidal particle attractions increase with a consequent rapid reduction in the permeability through the peat.

The **shear strength** of a peat deposit depends on its moisture content, degree of humification and mineral content. This is a key parameter for the engineer intending to load a peatland. The higher the moisture content of the peat the lower its shear strength. The higher the degree of humification and mineral content of the peat the higher its shear strength. To date it has not been easy to accurately determine insitu shear strengths of specific peat deposits in the laboratory due to difficulties in obtaining good representative samples from the field, getting them quickly to the laboratory and then trimming them to size without disturbance. As a consequence of this simple insitu field tests such as the vane test have been developed to give an indication of insitu shear strengths. But these have limitations.

The strength of a peat in a particular deposit is seldom dependent on depth. This is not surprising because the peat is normally unloaded in its virgin state and has a low submerged unit weight. Frequently a peat bog will show a peat strength decrease with depth due to the changing character of the peat particularly where it becomes less fibrous and more amorphous with depth.

3.4.3 Summary of properties

A summary of the above peat properties can be seen in the following table.

Table 3. Some Comparative Engineering Properties of Peat. Source: LS Amaryan, GV Sorokin & LV Ostroumova, 1973.

	Type of Peat			
Property	**Fibrous peat**	**Medium decayed peat**	**Decayed peat**	***(Peaty soil for comparison)***
Water content %	1400 - 2500	900 - 1400	500 - 900	*(200 – 500)*
Ash content %	1.5 - 3.0	3 - 8	8 - 30	*(30 - 90)*
Insitu bulk density (kg/m^3)	900 - 1100	900 - 1100	900 - 1100	*(1000 - 1500)*
Specific gravity	1.6 - 1.7	1.5 - 1.55	1.45 - 1.5	*(1.5 - 2.0)*
Void ratio	22 - 40	13 - 22	syys.13	*(3 – 9)*
Permeability (cm/sec)	10^{-3} - 10^{-4}	10^{-4} - 10^{-5}	10^{-5} - 10^{-6}	

4 Behaviour of Peat

4.1 Introduction

The consolidation and settlement of peat under load has long been recognised to be an extremely complex process and the intending road builder planning a construction on a peatland is faced with a major practical problem when he tries to predict and quantify the magnitude and rate of consolidation and settlement of a particular peat deposit.

The construction of an embankment, when carried out slowly, loads the peatland and causes the underlying peat to be squeezed and compressed as it responds to the load. In the normal case of a saturated peat this new load is first taken up by the free water within the peat and the local pore water pressures rise in response. (In the case of an unsaturated peat any gaseous phase present is expelled first.)

These localised increased pore water pressures then seek release through the expulsion of free water into the adjacent unloaded bog causing a reduction of the volume of the peat under load (giving rise to settlement) and a transfer of further load onto the peat skeleton.

This reduction of water pressure within the peat over time through the expulsion of internal water is termed ‘consolidation’.

4.2 Consolidation & Settlement

A peat deposit can settle and consolidate in 2 ways under the application of a load.

- Slow settlement with a change in volume, ie with gradual compression and consolidation allowing time for the peat mass to respond to the load. This is the desired method for improving the strength and bearing capacity of a peat deposit. Peat is highly vulnerable to ‘shear overstress’ during embankment construction and loading phases need to be carefully controlled to keep any stresses induced in the peat to within the strength of the peat at the time.

- Rapid settlement without a change in volume, ie with rapid spread and shear of the peat causing failure. This scenario is generally to be avoided but it can work to the advantage of the engineer where an embankment has to be constructed by the ‘displacement’ method (see Section 8.5).

In the normal course of events the consolidation and settlement of a peat may be separated into two main phases, ‘primary consolidation’ and ‘secondary compression’ as shown on the ‘time v settlement’ graph of Figure 9.

4.2.1 Primary consolidation

Peat is a very permeable material in its natural state and the magnitude of the initial settlement (the primary consolidation) under a controlled load is normally large and the period of settlement short, usually days.

During this initial phase the new load is taken up the free water and peat skeleton within the loaded peat. As the peat resists the load the vegetal structure compresses and strengthens and at the same time load is transferred back to the free water causing localised increases in the pore water pressures. This pressurised water in turn finds release into the adjacent unloaded peat causing loads to be taken up again by the loaded peat with further settlement, strength improvement and load transfer.

Normally this 'primary consolidation' takes place within the time it takes to build the embankment and its magnitude is usually dependant on the weight of the embankment and the thickness of the peat deposit and any other compressible layers. Once this initial phase has passed and any excess water pressures dissipated the settlement under load continues at a much slower 'secondary compression' rate which is generally accepted to be linear with the logarithm of time as shown on the following 'time v settlement' graph:

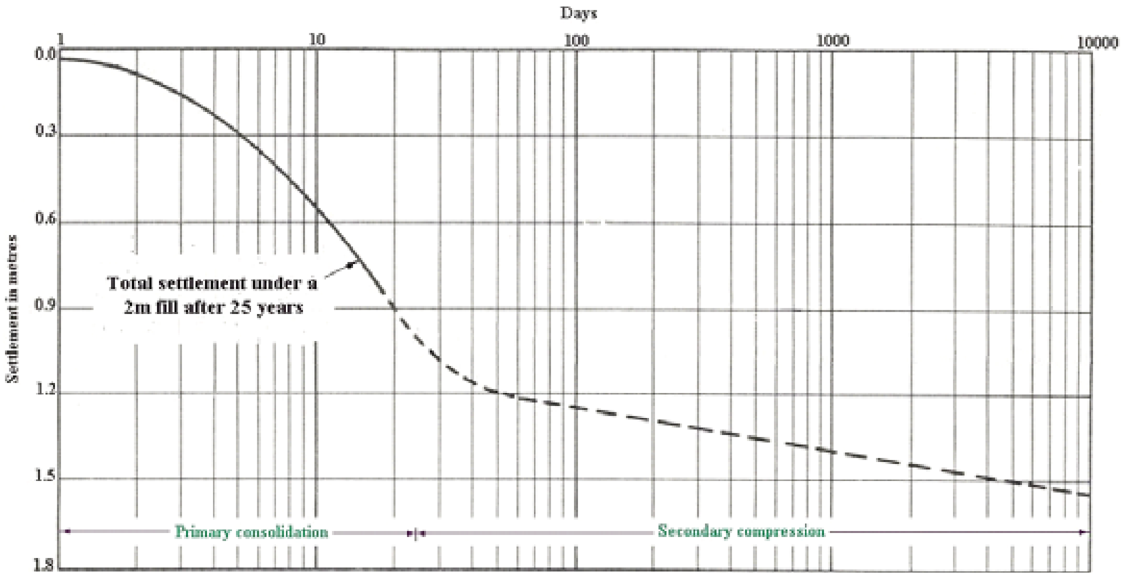


Figure 9. 'Time v Settlement' graph of an embankment on peat. (Chapter 6, The Muskeg Engineering Handbook. National Research Council of Canada)

4.2.2 Secondary compression

In the 'secondary compression' period the load on the peat is further transferred from the water within the peat to the internal peat skeleton as the peat continues to respond to the applied load. This secondary phase is now generally accepted to be the result of the loaded vegetable fragments within the peat mass slipping and re-organising to form a denser matrix. As the peat fragments come together and pore sizes close up the permeability through the peat reduces in response.

This simple scenario of course does not give a full picture of the complex processes at work in peat consolidation and strength improvement. The descriptions of 'primary consolidation' and 'secondary compression' are two parts of a continuous dynamic consolidation process at work within the loaded peat mass. The amount of primary consolidation incurred in any location will vary with type of peat but it can be generally approximated to around 50 per cent of the total settlement over time. Secondary compression is normally accepted to take place over a period of 30 years (or 10,000 days in Figure 8).

In all of the above it has been stressed that the peat should be loaded slowly enough for the underlying peat to respond to the increasing load and be permitted sufficient time to consolidate and gain strength rather than shear. If the embankment load is applied too quickly to such an extent as to approach or exceed the insitu strength of the underlying peat then catastrophic failure can follow. The construction of an embankment on peat is only possible if the ‘drained strength’ of the peat is utilised. If loaded quickly without allowing time for water pressures to be released the loaded peat will effectively have the shear strength of water, ie 0kPa. This has to be avoided in the construction of an embankment and designers should be aware that serious shear stresses can be induced even by moderate fills if loaded too quickly.

5 Ground Investigations & Laboratory Tests

5.1 Introduction

Peat is an interesting engineering soil with many unique characteristics. As such as much information as possible should be obtained on a particular deposit before embarking on any construction proposal. This usually involves carrying out appropriate site investigations and laboratory tests but local records of previous engineering works can also give very useful insights into likely performance characteristics and these should not be ignored.

5.2 Ground Investigations

Ground investigation for major projects over peat can be a very elaborate and time consuming exercise involving, amongst other issues, desk top investigations of current and historical geotechnical information, aerial photo-interpretations of surface ground features, site visits, field explorations and inspections (see Section 7.2).

All of course are very valuable in producing geotechnical information on which to base engineering decisions but generally practicing engineers on low volume road networks are presented with budgetary and time constraints that effectively rule out extensive ground investigations and only those investigations required to establish the main parameters of the peat deposits are usually commissioned. This means finding out as much information as possible within the working constraints and acting accordingly on the information gathered.

One of the most important elements of this ground investigation is the site visit and 'walkover'. This practical stage produces the very real benefit of an understanding of the surface features of the peatland such as ditches, watercourses, subsurface pipes, surface topography, peat workings, waterlogged areas, areas of free water, etc and this early understanding can provide a definite aid to the design and interpretation of the subsequent invasive ground investigation.

Some physical ground investigation is essential in all works to determine the type, depth and properties of the peat deposit in question. Short summaries of some of the most common methods for ground investigations suited for peat are set out on the following pages.

5.2.1 Ground investigation methods in peat

Probing

Probing has but one function in ground investigation surveys in peat and that is to establish the depth of the peat layer. It is usually carried out with proprietary steel probing rods available in sets of rods approximately 1m long with bayoneted or threaded connections to permit the rods to be assembled to a suitable length for the depth of peat. Where the peat overlies a hard surface such as moraine or rock a simple probing survey generally produces a good indication of the thickness of the peat. This is not the case where the peat is underlain by a soft layer such as clay, gyttia (organic clay) or silt, or peat deposits containing gravel layers from flooding episodes or chunks of woody materials. Here probing alone cannot differentiate between the differing materials. What can be said with some authority is that some form of probing exercise is essential in every project over peat, either as the main method for determining the depth of peat or as a calibration exercise for a non probe-based method.

Penetration testing

Penetration testing can be likened to the probing method outlined above but the method differs significantly in the sophistication of the probe equipment used. The advantage of penetration testing over simple probing is that the penetrometer probe can measure the relative stiffness of the layers that it passes through. There are a wide variety of types of penetrometers available on the market today with varying degrees of sophistication. Some of these in order of sophistication are:

Weight probing (or ‘sounding’) – This method involves pushing the probe into the soil under a constant standard load of 1kN. The head of the probe has a screw point and if the penetrometer does not sink into the soil under this load the penetrometer is rotated (either manually or mechanically) and the number of ‘half turns’ per 0.2m is recorded. By this means an indication of the relative stiffness of the soil layers can be presented in a soil profile.

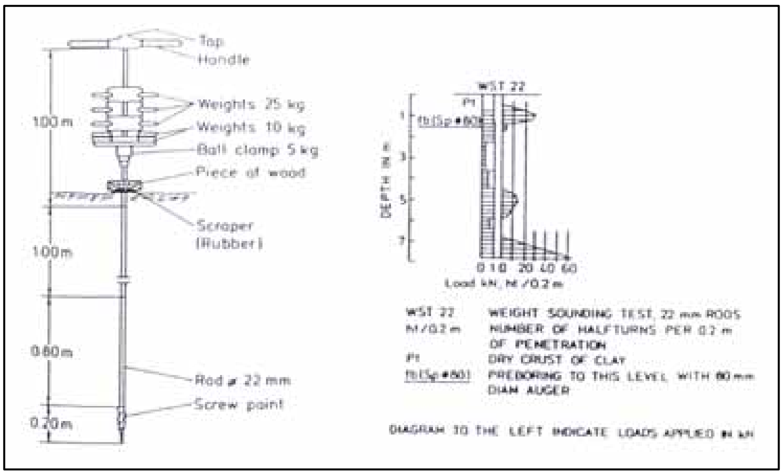


Figure 10. Weight probing Source: ‘Embankments on Organic Soils’, 1996.

Dynamic probing – In these methods the probe is driven through the soil by standard blows from a standard hammer falling a standard height (either manually or mechanically) and the number of blows recorded give an indication of the stiffness of the various soil layers. This test can be used for the full range of engineering soils and normally the lightest hammer is used for peat and organic soils.

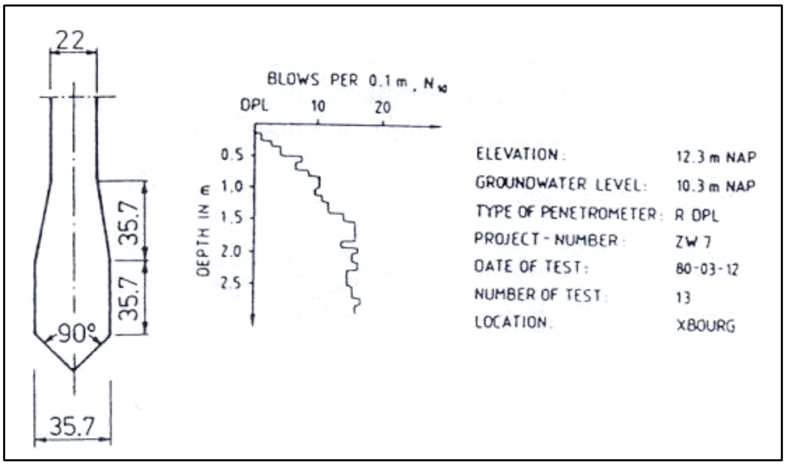


Figure 11. Light dynamic probe, Source: ‘Embankments on Organic Soils’, 1996.

The standard penetration test is the most common test of all for general engineering soils but the test uses a 63.5 kg hammer and is considered too heavy for peat soils.

Cone penetrometer testing – These modern probes have tips that can measure the cone resistance through the soil, the skin friction on the side of the probe and, in the case of the piezocone, the pore pressure generated in the soil by the probe. This is the most accurate penetration method at present and the readings obtained by this form of testing can be used to determine the soil stratigraphy, type and soil density. It may be possible to estimate the undrained shear strength of very humified peats on occasion using this method but not in fibrous peats.

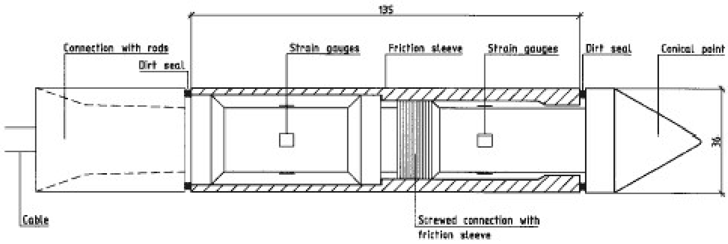


Figure 12. Electrical friction cone, Source: Guide to cone penetration, J J M Brouwer, Lankelma

All of methods listed above have their own advantages and all require specific corrections to be made to the data gathered during the test to reflect the particular behaviour of the peat at failure.

Ground penetrating radar (GPR)

Ground penetrating radar or 'soil radar' is a growing ground investigation method for peatland surveys and is particularly useful for establishing the thickness of existing roads and soils layers prior to widening and strengthening. With skilled interpretation the radargrams produced by the survey can show clear boundaries between the road and the underlying peat and can be used to monitor the longterm behaviour of the road, or test embankment, with good accuracy.

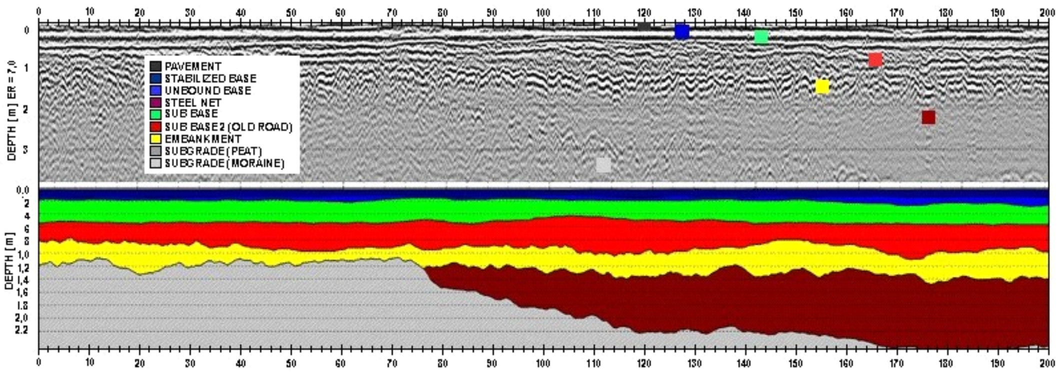


Figure 13. Radargram of a road embankment approaching peatland (top) and the resulting interpretation (below), Source: Roadscanners Oy, Rovaniemi, Finland.

Sampling

Physical site sampling of soils is the only truly definitive way of identifying a buried soil. Unfortunately, like many tests, the act of obtaining a sample can affect the sample being recovered and this is particularly important when sampling in peatlands. Depending on the quality of the sample, samples are variously termed ‘undisturbed’ where the sample is effectively similar to the material in the ground, ‘disturbed’ where the sample has lost some of the attributes of the soil in the ground and ‘remoulded’ where the sample no longer has the structure of the material in the ground. All types of sample have a value depending on the type of laboratory test planned.

Site sampling of shallow peat areas is generally carried out by means of a screw auger, post hole auger or Hiller auger and the samples obtained (disturbed samples) used to determine the classification and stratigraphy of the peat.

Sampling of deeper peat deposits is usually carried out with a ‘thin-walled piston sampler’ or similar sampler to extract undisturbed samples at depth and the samples obtained used in the determination of the index properties and settlement parameters of the peat. The piston sampler cuts a sample of soil by being pushed in closed mode down through the deposit to the test level at which point a piston slowly pushes a sample tube into the soil to be extracted. The technique aims to minimise edge effects on the sample but some disturbance such as smear is inevitable as the sampler is inserted.. Once the cut sample has stabilized the complete assembly is withdrawn and the test sample recovered.

The importance of obtaining good quality large size undisturbed samples capable of adequately represent the insitu inhomogenous nature of the peat deposit is stressed by the Swedish Geotechnical Institute (SGI) who have developed their own 100mm diameter peat sampler for recovering such undisturbed samples at depth with good results.

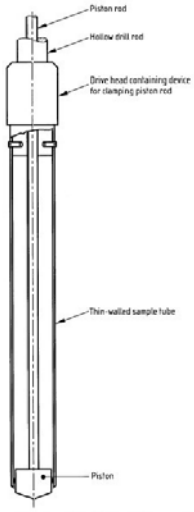


Figure 14. Thin walled piston sampler
Source: BS5930 Site Investigations

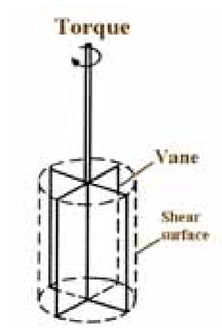
The Swedish Geotechnical Institute Sampler

The "SGI sampler" has a sharp circular wave-toothed cutting edge mounted on 100mm diameter plastic tube capped with a robust driving head on top. The length of the tube is variable and dictates the length of sample recovered but normally a 1.0m long sampler is used. The extent of disturbance in the sample largely depends on the method used to drive the sampler into the ground and following testing it has been established that the best results are usually achieved when the sampler is driven down into the peat by means of a lightweight percussive machine.

Figure 15. Photographs of The Swedish Geotechnical Institute Sampler in use.
Source: Swedish Geotechnical Institute.

Field vane testing

The field vane test is commonly used to give an indication of the in situ undrained shear strength of peat deposits but the results obtained should be treated cautiously. The test involves a probe with four orthogonal vanes being pushed into the peat to the test depth and rotated under a measured torque until a failure is caused in the peat at the edge of the vane. The test is relatively simple to perform and understand but research by Landva (1980) has shown that the failure in peat does not occur at the periphery of the vane but on a cylinder 7mm to 10mm outside the vane and results from field vane tests require correction for this reason. In the standard field test the measured torque at failure is used as a basis to calculate the vane shear strength of the peat.



Picture 16. Shear vane
Source: SGI

Vane testing of fibrous peats is no longer standard practice in Sweden as it is considered that the test does not give good results due to the tearing effect of fibres away from the vanes and the position of the relative failure surface. If and when the test is used in fibrous peat the results are only taken as an indication of the relative strength of the peat.

5.2.2 Ground investigation methods in the ROADEX partner areas

A summary of the ground investigation methods used by the Roadex partner areas is presented below.

Table 4. Ground Investigations: In-situ test methods in the Roadex partner areas.

Technique	Norway	Finland	Sweden	Scotland
Borehole	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
Probing	Used regularly	Used regularly	Used regularly	Used regularly
Undisturbed sampling	Used regularly	Used Occasionally	Used Occasionally	Used Occasionally
Shear vane	Used regularly	Used regularly	Used regularly	Used Occasionally
SPT/CPT	Used Occasionally	Used regularly	Used Occasionally	Used Occasionally
Swedish Weight Sounding	Used Occasionally	Used regularly	Used Occasionally	Not used
Georadar	Used Occasionally	Used regularly	Used Occasionally	Used Occasionally

From this it will be seen that probing is the most common method used for ground investigation in peatlands in the Northern Periphery roads districts and that this is closely followed by shear vane testing, undisturbed sampling, penetration testing and, increasingly, the use of ground penetrating radar.

5.3 Laboratory testing

Laboratory tests, apart from those involving pure research, are generally designed to try to replicate the conditions expected to be encountered on site. This modelling is difficult to produce for peat samples particularly those of fibrous peats with very high water contents and permeability. Special large samples can be taken and tested in especially large testing apparatus in an effort to create the site conditions in the laboratory but this facility is not usually available to the non-research based engineer charged with constructing, or maintaining, a low volume road over peat.

In these circumstances the simpler 'classification tests' outlined in Tables 3.1 and 3.2 are normally used together with empirical relationships to produce some guidance on the likely behaviour of the peat in situ.

Table 5. Laboratory testing methods used in the Roadex partner areas.

Technique	Norway	Finland	Sweden	Scotland
Classification	Used regularly	Used regularly	Used regularly	Used regularly
Moisture content test	Used regularly	Used regularly	Used regularly	Used regularly
Oedometer test (includes Rowe Cell and compressiometer	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
Triaxial test	Research technique only	Not used	Not used	Research technique only
Organic content test	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
Ash content test	Used Occasionally	Not used	Used Occasionally	Used Occasionally
Dry density test	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
Bulk density test	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
Direct shear test	Used Occasionally	Not used	Used Occasionally	Not used

From this Table it will be seen that Classification and Moisture Content tests regularly feature in laboratory investigations for road construction over peat and these 2 tests can probably be considered to be the minimum amount of testing required for best practice.

6 Embankments over peat

6.1 Initial considerations

The selection of a method for the construction or improvement of a road over peat will normally be based on economic considerations coupled with the performance requirements expected of the new carriageway. Most public roads, even relatively high speed roads, can stand fairly large settlements if they are long and uniform particularly if the ride quality is not significantly affected. Short differential settlements across the carriageway on the other hand can pose quite dramatic traffic hazards for fast moving vehicles and will usually require to be designed out if at all possible.

High speed national strategic networks will therefore normally require conservative forms methods to be employed to satisfy the tight carriageway tolerances involved. Lightly trafficked lower class roads on the other hand may be able to exploit the less expensive, less rigorous solutions available, particularly where vehicle speeds are likely to be lower.

Irrespective of which end of the performance spectrum a particular road embankment lies it will have to be designed to meet the two main engineering criteria of embankment stability and settlement.

6.2 Bearing Capacity

The words 'bearing capacity' and 'peatland' do not immediately sit well together. Peat in its natural state consists of water and decomposing plant fragments with virtually no measurable bearing strength. It can of course be transformed under suitable circumstances and methods into an acceptable engineering material but its extensive range of morphologies and types does not permit a single definition of peat 'bearing capacity' to be easily proposed. A broader description of bearing capacity in peat is therefore required

'Bearing capacity' in its classical soil mechanics sense can be defined as "the ability of a soil to safely carry the pressure placed on the soil from any engineering structure without undergoing a shear failure with accompanying large settlements' (U.S.Army Corps of Engineers, 1992). The same source defines 'ultimate bearing capacity' as the pressure to cause a 'critical plane of failure (slip path) in the soil'.

In applying these definitions to peatlands therefore it seems prudent to consider 'shear failure', 'settlement' and 'critical planes of failure' when discussing bearing capacity over peat. The "Centre for Civil Engineering Research and Codes" of Indonesia and The Netherlands certainly agree when they record their view that the main problems in constructing roads over peat and organic soils are 'stability and long term settlements' and develop their "Guideline road construction over pear and organic soils" accordingly. Their guideline names settlement and shearing/stability as 2 of the main geotechnical mechanisms involved in road construction over peat soils. This view is also strengthened by the US Transportation Research Board, Transportation Earthworks Committee (AFS10) who are promoting research in 2004 for 'the stability (bearing capacity) of embankments'.

This report will also take this wider view. For the purposes of this report 'bearing capacity' will be taken in its broad sense and include the consideration of stability and settlement.

6.3 Stability

All embankments should be designed to be stable and be constructed in such a fashion so as to produce a sufficient factor of safety against foundation and sideslope failure. A typical embankment over peat can fail in 2 ways:

- by failure of the underlying peat along a slip surface, normally in the form of an arc

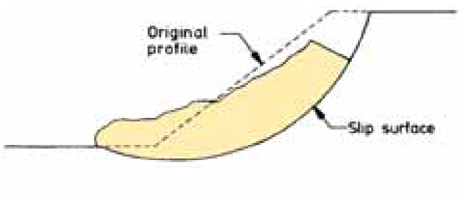


Figure 16. Circular slip surface
Source BS6031 "Earthworks"

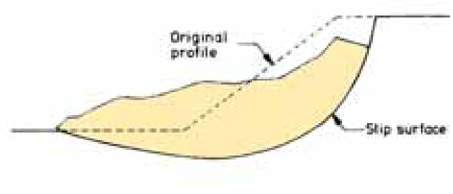


Figure 17. Non circular slip surface.
Source BS6031 "Earthworks"

- By punching shear into the underlying peat where the embankment settlement is accompanied by heave of the adjacent peat bog alongside the embankment

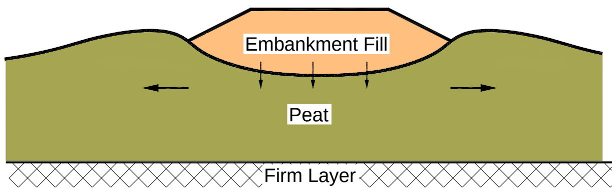


Figure 18. Punching shear cross-section (G Smith)

Appropriate analyses should be carried out ahead of the construction works to ensure that these failure conditions are avoided.

Various forms of proprietary stability analyses are available to the practising engineer in the geotechnical market and Internet such as PLAXIS, OASYS, FLAC, SAGE, etc. The selection of the most suitable method of analysis (spreadsheet, general analysis, finite difference/finite element analysis, 2 dimensional, 3 dimensional, etc) should be left to an engineer experienced in the field. As part of this work it will be necessary to examine the short term construction stability of the embankment, including the effects of the different phases of the embankment construction, as well as the long term stability of the chosen method of construction.

Stability is unlikely to pose a design problem on fibrous bog peats due to the reinforcing effect of the peat fibres but it can be a significant consideration in the design and performance of embankments over fen peats which tend to be more humified and less permeable.

6.4 Settlement

The settlement of an embankment on peat has two distinct considerations; magnitude and rate of settlement. The rate of settlement, and the time needed for the embankment to settle, is normally the more important consideration of the 2 parameters for a road construction project if future post-construction maintenance is to be minimized. Post-construction repairs invariably require closures, incur cost and delays to traffic and these are best resolved within the original works with better design. The early estimation of the magnitude and rate of settlement is therefore a significant factor in a successful embankment over peat.

Peat exhibits an immediate ‘elastic’ settlement as soon as it is loaded, during the application of the load in fact, and ‘consolidation’ settlement thereafter. It is possible to estimate the ‘immediate’ settlement element but most authorities in the Northern Periphery partner areas normally choose to ignore this elastic element and concentrate their efforts instead in assessing the magnitude of the ‘consolidation’ settlement as this has a far greater effect on the serviceability of the finished road.

As mentioned in Chapter 4 an embankment on peat consolidates (settles) in 2 stages; the ‘primary’ consolidation stage as the pore water is squeezed out of the peat mass and the ‘secondary compression’ stage as the internal peat matrix slowly takes an increasing share of the embankment load as it increases in strength.

These phases can be estimated by a number of means but all methods can only produce general predictions as the actual conditions on site will invariably differ from the laboratory test conditions. Site instrumentation is normally considered essential to check that site settlements are proceeding as predicted.

6.4.1 Methods of prediction of settlement

The OECD Report “Construction of roads on compressible soils” (1979) reports that the settlement of embankments on peat on major roads in northern Europe is generally assessed using the methods listed in Table 5 below. Interviews with practising engineers in the Roadex partner areas in 2003-04 confirm that this is still the case.

Table 5. Table of Assessment of Embankment Settlement in northern European States. Source: OECD, “Construction of roads on compressible soils.

	Calculation Method		**Test for parameters**		**Correction**
	Magnitude	**Rate**	**Magnitude**	**Rate**	
Norway	Janbu non linear theory	One dimensional theory	Oedometer	Oedometer	Skempton - Bjerrum
Finland	Non linear theory	One dimensional theory	Oedometer or water content	Oedometer	Skempton - Bjerrum
Sweden	Non linear theory	One dimensional theory	Oedometer	Oedometer	Skempton - Bjerrum
Scotland	Non linear theory	One dimensional theory	Oedometer	Oedometer or insitu tests	Skempton - Bjerrum

The assessment of settlement for low volume roads does not however usually warrant such comprehensive and detailed analyses and normally a simpler assessment for primary consolidation as set out by Carlsten (1989) can suffice. The writer visited a number of design offices in the Partner areas during his research for this report and saw Carlsten’s method being used as a reference in all.

Carlsten’s method for the design of Preload

Carlsten’s method is based on his experience of a number of road construction projects over peatlands in Sweden and his process results in an estimation of the settlement likely during the primary consolidation phase. (If secondary compression predictions are required Carlsten recommends that these be carried out using proprietary computer software programs.)

His method, set out in a series of research papers, concludes with the presentation of a number of settlement diagrams that can be used in the absence of undisturbed samples of the insitu peat. These diagrams, shown below, bring together 4 of the main parameters governing settlement in peat: the thickness of peat, its water content, the applied load and the time elapsed. The diagrams assume that the peat is normally consolidated. For a previously loaded peat a correction is suggested.

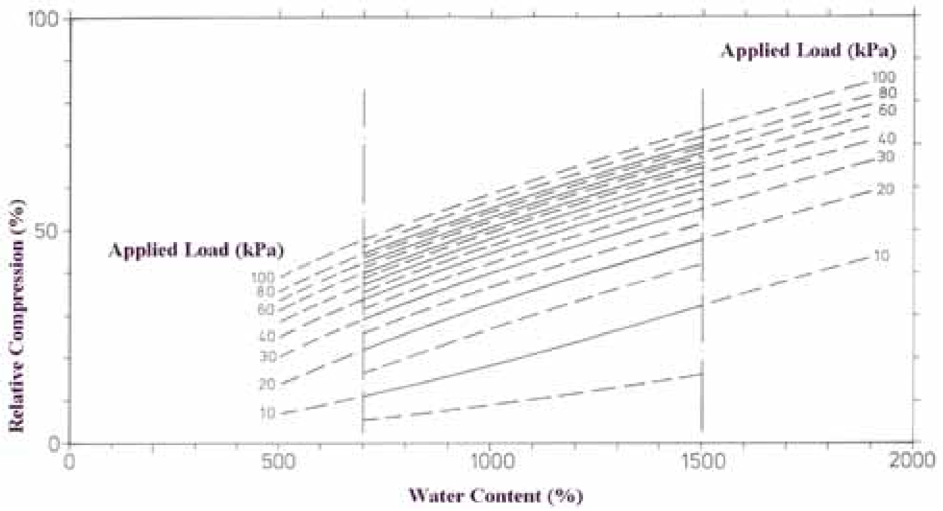


Figure 19. Carlsten's diagram 1: Relationship of deformation v water content for different loadings.

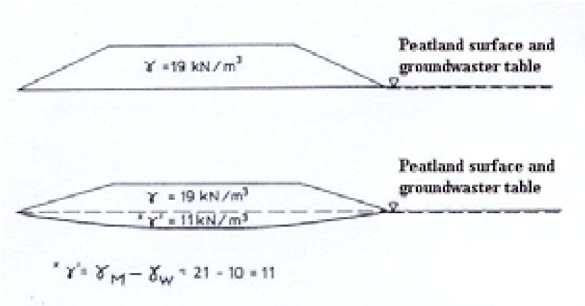
Carlsten recommends that the calculation of the settlement of an embankment using this chart is carried out as a series of calculations to better simulate the embankment loading sequence making allowance for any buoyancy effects as the embankment settles into the water table. This buoyancy effect can be significant in the loading sequence. A typical embankment fill of 19 tonnes/m³ can be reduced to 11 tonnes/m³ under water (Figure 20) with consequent reduction in load on the underlying peat.

Table 6 below gives an illustration of the suggested calculation process for the settlement of an embankment on 4.5m of peat modelled as 4 layers of 1.0m, 1.0m, 1.0m and 1.4m with layer water contents of 1200%, 1200%, 1300% and 1000% respectively.

Table 6. Settlement v Loading Sequence.

Soil Layer No	Thickness of layer m	Water content %	Embankment loading sequence									
			q =10 kPa		q =20 kPa		q =30 kPa		q =40 kPa		q =50 kPa	
			ε %	δ m	ε %	δ m	ε %	δ m	ε %	δ m	ε %	δ m
1	1.0	1200	23	0.23	38	0.38	46	0.46	50	0.50	54	0.54
2	1.0	1200	23	0.23	38	0.38	46	0.46	50	0.50	54	0.54
3	1.0	1300	27	0.27	42	0.42	48	0.48	53	0.53	56	0.56
4	1.4	1000	18	0.25	32	0.45	39	0.55	44	0.62	48	0.67
∑				**0.98**		**1.63**		**1.95**		**2.15**		**2.31**

These results can be used in turn to prepare a 'settlement v load relationship' curve for the particular embankment construction as shown in Figure 21 following.



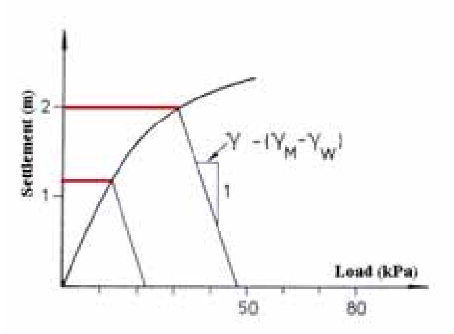


Figure 20. Effect of buoyancy. *Figure 21. Settlement v load relationship'.*

This chart is also used by Carlsten for the estimation of settlement in peatland preloading operations as below. The 4.5m embankment arrangement in Table 6 can be used to illustrate the principle.

In line with normal practice the embankment is to be constructed in 2 stages. The first layer is planned to be 1.2m thick (Σq = 22.8 kPa) and a second layer of 1.3m (1.2m + 1.3m = 2.5m, Σq = 47.5 kPa) placed when the underlying peat has consolidated sufficiently to support the additional load (taken as when 70% of the primary consolidation of the first layer has been reached.) These 2 loading stages can be seen in Figure 22.

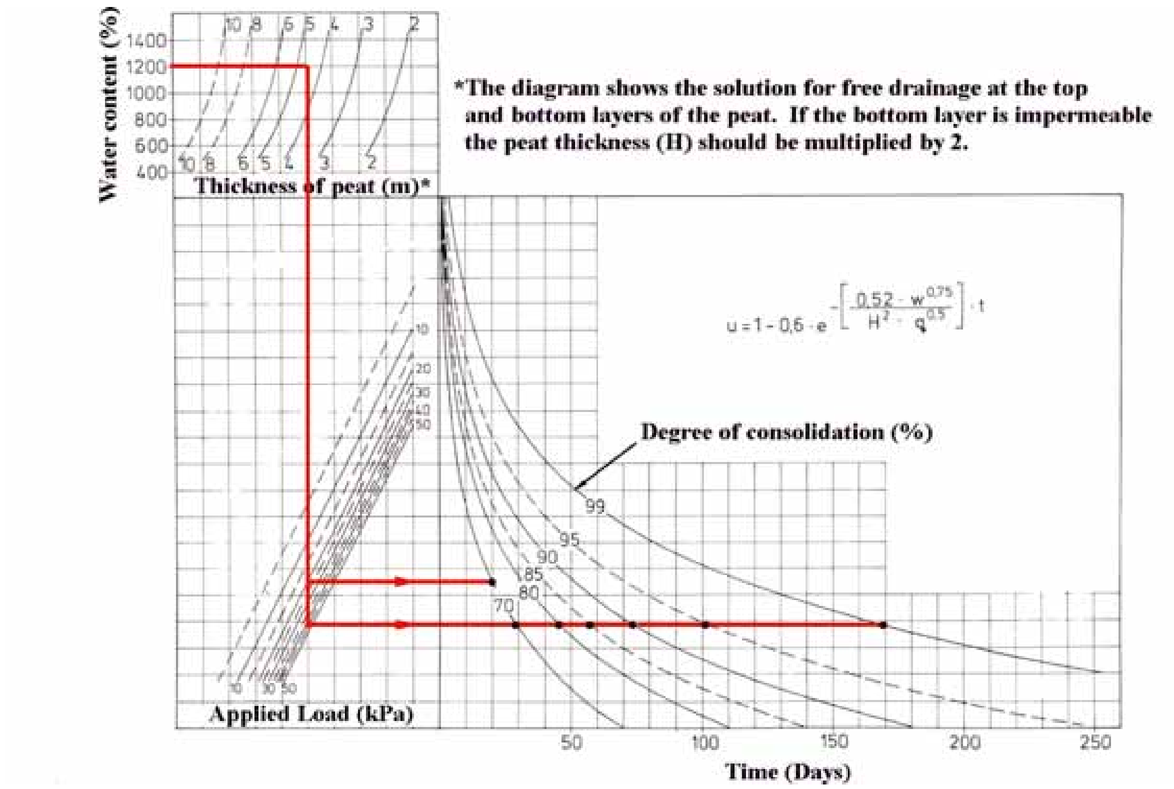


Figure 22. Carlsten's diagram 2: Calculation of consolidation in peat (after Carlsten, 1989)

The use of Carsten's method can be illustrated in Table 7 where predictions of the stage settlements from Figure 22 are presented alongside the expected embankment settlement with time from Diagram 2.

Table 7. Table of estimated settlement, Source: P Carlsten, Vägbyggnad på Torv', SGI Vägledning 2, 1989.

Layer	**Load Σq (kPa)**	**Consolidation (%)**	**Time from Diag 2 (Days)**	**Predicted final settlement from Fig 19 (m)**	**Settlement with time (m)**
Stage 1	22.8	70	20	1.18	0.83
Stage 2	47.5	70	29	2.00	1.40
		80	46		1.60
		85	58		1.70
		90	75		1.80
		95	103		1.90
		99	170		1.98

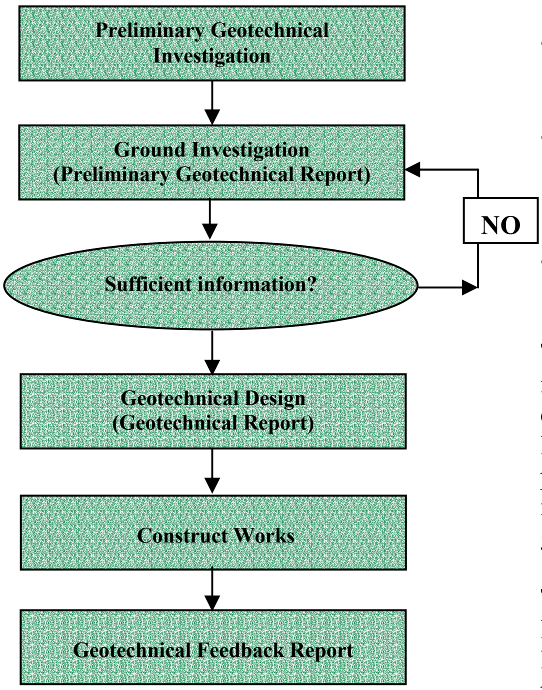
This table only gives an indication of the settlement in a peat layer of course. If the peat layer is part of a series of compressible layers the settlement in the other layers must be estimated also to arrive at an overall prediction figure for settlement of the embankment.

7 Geotechnical Risk Management

7.1 Introduction

Ground conditions for engineering works can never be totally certain and invariably constitute significant risk for projects. Some uncertainty will always remain even after the most rigorous design procedures. For road construction and improvement schemes to be successful therefore the accompanying ground conditions and geotechnical risks must be adequately identified and their effects managed if construction problems and serviceability difficulties are to be avoided.

In recognition of this Roadex Partner countries normally follow a formal geotechnical design and risk management process for their road construction and improvement projects, particularly for those involving peat, so that the geotechnical risks are identified ahead of the problems on site and correctly managed. Depending on the complexity of the proposed works and the degree of geotechnical risk identified the key stages in this risk management process normally comprise:



- a **preliminary geotechnical investigation**
- a **geotechnical design investigation**, that includes the ground investigation and produces the **geotechnical report**
- a **geotechnical feedback report** on completion of the Works

These 3 stages, although apparently independent at first glance, are part of a continuous management process over the life of the project. Reports and engineering positions are updated and amended as new information becomes available and geotechnical risk assessed.

The process can be applied to all projects, large or small, the only difference being the level of work considered necessary to meet the sophistication of the project.

Figure 23. Source "Eurocode 7: A commentary".

7.2 The Preliminary Geotechnical Investigation

The Preliminary Geotechnical Investigation is normally the first stage of the geotechnical design and risk management process. A good preliminary geotechnical investigation will set out to cover the geotechnical risks, implications and feasibility of all construction options for the project and make recommendations for the subsequent main project ground investigation where this is considered necessary.

In the UK the preliminary geotechnical investigation, or the 'Preliminary Sources Survey', generally takes a standard form as set out in the Design Manual for Roads and Bridges Volume 4 Section 1, HD22/02 'Managing Geotechnical Risk'. This volume recommends the following format:

- **An introduction**
- **Sources of information used** and desk studies – details of enquiries, site history, former uses, industries, mining, contamination
- **Field studies** – walkovers, geomorphological/geological mapping, probing, pitting and testing work, drainage/hydrological studies, geophysical or photographic surveys, etc.
- **Site description** – geography, hydrology, hydrogeology, geomorphology, man-made features, former history, etc
- **Ground conditions** – soils anticipated, known and predicted engineering properties, significance of geological formations, ground water conditions, etc.
- **Preliminary engineering assessment** – for each soil location and soil type. This section can be extensive and detailed touching on the geotechnical considerations of individual cuttings, embankments, subgrade sections, etc.
- **Comparison of project options** and risks – all geotechnical, geo-environmental, historical factors likely to influence the project (routes, alignment, health & safety, buildability).
- **The Geotechnical Risk Register** – The geotechnical risk register is normally started during the preliminary geotechnical investigation and once commenced is continued through to project completion.

For large projects the preliminary geotechnical investigation is usually accompanied by an appendix that details the further investigations that should be carried out including the objectives and format of the main ground investigation works. One of the benefits of this investigation is the identification of areas of the project needing more intensive ground investigation. Without this ground targeting it is possible that the subsequent ground investigation works could miss significant geotechnical hazards along the route.

7.3 The geotechnical report

The Geotechnical Report is the main geotechnical interpretation report for the project and includes details of all of the investigations carried out together with the design of the geotechnical structures. A typical report will include (from HD22/02 'Managing Geotechnical Risk'):

- **An introduction** outlining the scope and objective of the report and a description of the project including a site description
- **A review of all existing information**, eg topographical maps, geological maps and records, aerial photographs, records of mines and mineral workings, land use and history, previous ground investigations, flood records, contaminated land, etc.

- **Field and laboratory studies**, walkovers, geomorphological/geological mapping, ground investigations, drainage/hydrological studies, geophysical surveys, laboratory investigations, etc.
- **Ground summary**, the summary interpretation of the geography, topography, geology, hydrogeology, geomorphology, man-made features and historical development of the project.
- **Ground conditions and material properties**, the detailed interpretation of the ground conditions along the project route. This generally includes details and descriptions of the various materials to be encountered along the route together with a full justification for the parameters to be adopted for the geotechnical design.
- **The Geotechnical Risk Register**, highlighting the expected risks and consequences together with any mitigation measures
- **Geotechnical design criteria**, for each soil location and soil type. This section would normally be a significant section of the report and would be expected to include detailed interpretations and justifications for the geotechnical design. Elements considered generally include earthworks (cuttings & embankments), highway structures, reinforced earthworks, drainage, subgrade, etc.
- **Instrumentation and monitoring**, details of the recommended instrumentation and monitoring systems and frequency of readings.
- **References**

7.4 The geotechnical feedback report

The geotechnical feedback report is a record of the geotechnical matters encountered during the Works. It is normally commenced at the start of construction and comprises a full record of ground conditions, materials, structures and other issues for use by the client and future maintenance managers. A typical document would include sections on earthworks (cuttings and embankments), subgrades, capping layers, drainage, imported materials, structures, testing, instrumentation, monitoring and design changes or problems experienced during the Works.

7.5 The geotechnical risk register

The management of geotechnical risk, like all risk management, is a dynamic process that has to continuously monitor and review risks as they are discovered. The Risk Register assists this dynamic management process by systematically considering all identified risks in a structured fashion. This generally involves 4 phases

- The identification of the hazard
- Assessing the probability of it occurring and its impact if it did
- Managing the risk identified
- Allocating responsibility and action

Good communication between client, designer and contractor is needed however for this process to work. Where all parties can agree to sign up for co-operation the risks identified within the project have a better chance of being considered early and contingency planning put in place to meet the risk.

A typical Risk Register for a thin embankment over peat is shown in Table 9 following.

Table 8. Typical Geotechnical Risk Register criteria for Probability (P), Impacts (I) and Risk (R).

PROBABILITY (P)		
Very Likely	>75%	5
Likely	50-75%	4
Probable	25-50%	3
Unlikely	10-25%	2
Negligible	<10%	1

IMPACTS (can be amended to suit contract circumstances)		IMPACT (I)	
Either TIME dependent or	**COST dependent (€)**		
>10 weeks added to planned completion date	>€1M	Very high	5
>4 weeks added to planned completion date	€100K to €1M	High	4
>4 weeks<1wk added to planned completion date	€10k to €100k	Medium	3
1 to 4 weeks on activity: no change to planned completion date	€1k to €10k	Low	2
<1 week to activity: no change to planned completion date	<€1000	Very low	1

Calculated RISK R=PxI	Degree of Risk	Suggested Action
17 to 25	Unacceptable	If risk cannot be reduced project should not proceed
13 to 16	Unacceptable	Work must not start until risk has been reduced
9 to 12	Significant	Reduce risk. (Mitigate or transfer.)
5 to 8	Tolerable	Consider risk reduction measures
1 to 4	Trivial	Monitor work

Table 9. Simplified example of part of a risk register for a thin embankment over peat on a geotextile.

No	HAZARD/RISK	CAUSE	BEFORE CONTROLS			CONSEQUENCE	RESPONSE (avoid, transfer, mitigate, accept & manage)	AFTER CONTROLS		
			P	I	R=PxI			P	I	R=PxI
1	Unexpected ground conditions	Ground conditions encountered on site differ from those indicated in the project ground investigation.	3	3	9	Construction delayed. Design review required with possible changes in design. Project cost and timescale increased	Monitor works in progress. Use experienced staff on site. Ensure that site staff are aware of the results of the ground investigation and the basis of the design of the permanent Works	3	1	3
2	Flooding	Prolonged rain, Rise in groundwater levels within bog. Local watercourses break banks.	3	4	12	Permanent works damaged. Work stops. Increased costs for repair of the Works. Project delayed	Ensure that cut off drains are installed and serviceable. Monitor weather forecasts and take action in light of forecasted poor weather	2	2	4
3	Site clearance	Clearance of vegetation from within the site limits ahead of the permanent Works	4	3	12	Damage to fibrous surface of peatbog. Romoval of surface rootmat. Design of Works affected	Use low ground pressure construction plant. Ensure that site staff are aware that existing root mat has to be retained as reinforcement	2	1	2
4	Placing of fill on geotextile	Rupture, puncture or tearing of the permanent geotextile	4	3	16	Damage to permanent Works. Fill material laid directly on to bog surface. Failure of subgrade	Protect geotextile with layer of fine material.. Ensure that site staff are aware of need to protect geotextlie during installation	2	1	2

Risk rating (**R**) = Probability (**P**) x Impact (**I**)

8 Types of Construction

8.1 Introduction

This Chapter aims to give a brief introduction to the current range of techniques presently available to the practising road engineer and indicate some of the advantages and disadvantages associated with them.

Construction over peat can essentially be sub-divided into five broad classifications:

- 8.2 Avoidance
- 8.3 Peat excavation
- 8.4 Peat replacement
- 8.5 Peat displacement
- 8.6 Peat left in place

And these classifications can in turn be broken down into further derivatives designed to suit particular applications. Not all of course are currently being practised across the Northern Periphery but most have been trialled or tested by one or more Partner areas at some time and local practices have subsequently been developed based on experience.

8.2 Avoidance

The first method of dealing with peat is to avoid it. If circumstances permit (alignment, environment, economics, etc) any engineer faced with constructing or improving a road crossing a peatland has to consider going round the obstacle. This does not of course improve the bearing capacity of the construction but it is a sensible option, if available, for dealing with a peatland.

Table 10. Avoidance of peatland .

Avoidance of peatland	
Advantages	Avoids potential problems in dealing with peat and other soft soils. Should result in better long term road characteristics
Disadvantages	Requires alignment revision. Possible reduction in alignment quality.
Risks	None, other than normal construction risks.
Case Histories	None

8.3 Peat excavation

Excavation is the safest option for taking a road across peat or improving an existing road and many engineers consider it to be the only dependable way of making a permanent road across peatland other than avoidance altogether. With the excavation method all of the weak materials on the road line are excavated out to expose a firm layer of sufficient bearing capacity to accommodate the new construction. Thereafter an embankment of appropriate thickness is constructed on the exposed firm layer to enable the design to be fulfilled with a minimum threat of settlement or shear failure.

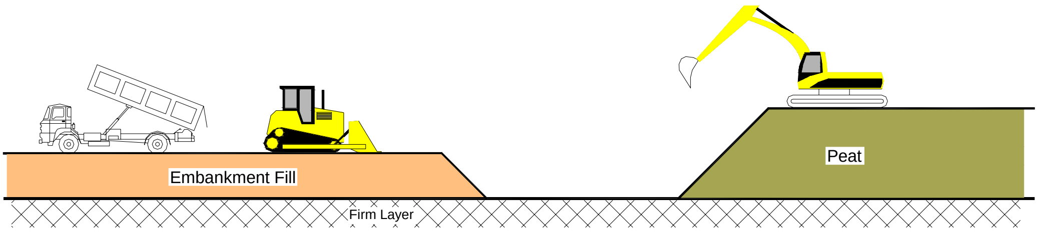


Figure 23. Long section along the excavation method. (P. Carlsten modified by G. Smith).

Where excavation can be used it is probably the most dependable way of constructing a durable road across a peatland provided that all of the peat can be excavated down to a sound load bearing layer. In these circumstances the bearing capacity of the new embankment becomes dependant on the method and materials of construction. What can be said is that where this can be done excavation will normally allow a stable platform to be constructed relatively quickly with negligible future settlements. For these reasons most new major roads across the Northern Periphery are built with the excavation method.

Excavation is however only generally economically feasible for the shallower depths of peatlands where quantities are relatively small. Experience in the Northern Periphery to date suggests that the economic limit for the excavation method normally lies somewhere between 3 and 4 metres. The actual economic depth will of course depend on local parameters, such as the type of peat, the area of the peatland, the cost of the backfill material, availability of spoil areas, etc. For 4m of excavation it will become increasingly more difficult to keep the peat excavation sides stable.

All partner areas across the Northern Periphery employ the excavation method with generally similar dimensioning parameters. A typical cross-section for measurement is shown below:

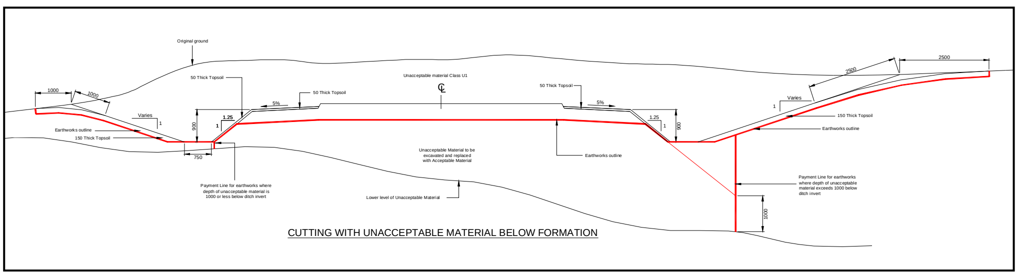


Figure 24. Cross-section of a cutting through deep peat. (The Highland Council).

The excavation method is not without its problems.

- In deeper bogs local pockets of peat can be left in place that can result in bearing capacity problems and settlements in the finished embankment if they are left uncorrected.
- If the peat has a low shear strength, the sideslopes of the excavations can be unstable and migrate into the excavations before they can be backfilled and this can result in significant increases in excavation quantities.

- Adjacent structures may be adversely affected by the removal of side support if not adequately protected.
- Suitable areas must be identified locally for the disposal of excavated materials, usually classed as "landscaping areas".
- The new embankment can act as a linear drain or drainage corridor through the peatland and affect the drainage regime and overall hydrology of the area.

Despite these apparent pitfalls excavation is still generally the preferred method for most engineers for high speed main roads.

Table 11. Summary of the peat excavation method.

Summary of the peat excavation method	
Advantages	Proven technology. Should achieve a good bearing capacity using a standard embankment construction on a sound layer. Limited consolidation and settlement over the lifetime of the road. No additional time required for surcharge effects.
Disadvantages	Significant quantities of excavated materials created. Land required for formation of sideslopes in peat and disposal of excavated materials. Difficulties in excavation and placing fill below water table. Normally demands high quality of fill material (low percentage of fines). Deep excavations may have effects on adjacent lands and structures. Unexcavated soft material below embankment may cause future settlements.
Risks	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.
Case Histories	F4, Sc4, Sc8

8.4 Peat replacement

The peat replacement method essentially involves taking out the weak peat material from along the line of the new road and replacing it with a suitable fill material to form the foundation for the new embankment as shown below. The method is very similar to the Excavation method but in the peat replacement method the grade line of the new embankment is at or above the level of the adjacent peatland.

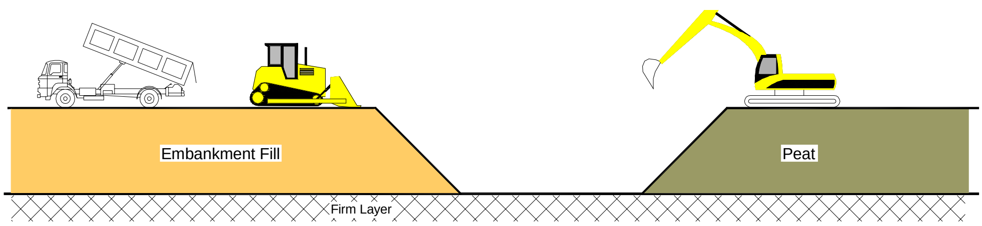


Figure 25. Long-section of Peat Replacement Method. (P. Carlsten modified by G. Smith).

Like the excavation method, the weak peat material is excavated down to an acceptable firm layer and the void created backfilled with normal embankment construction methods up to the required grade level with something more suitable, preferably with non-cohesive material locally won on site. All partner areas across the Northern Periphery employ peat replacement methods with generally similar construction practices. A typical cross-section for measurement (from The Highland Council) is shown in Figure 26.

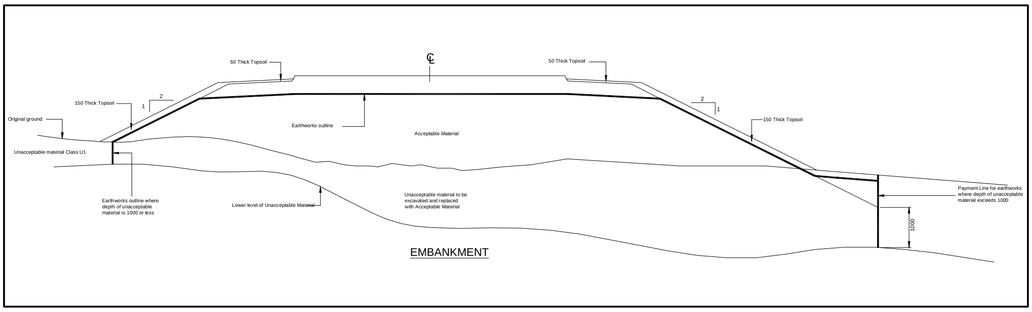


Figure 26. Cross-section through an embankment through deep peat. (The Highland Council).

Replacement is generally the preferred method of construction and improvement for high speed roads that cross peatlands. If constructed well the method should produce a serviceable embankment with minor settlement requiring minimal future maintenance. Great care has to be taken however to ensure that all of the peat material is removed from below the new embankment as any soft pockets that remain could give rise to differential settlement in the finished structure.

'Partial excavation' techniques can extend this method for uniformly deep peat deposits where the weight of the new road embankment is expected to be sufficient to displace the type of peat below. The method is discussed in the "Peat Displacement" section.

Table 12. Summary of the replacement method.

Summary of the replacement method	
Advantages	Proven technology. Should achieve a good bearing capacity using a standard embankment construction on a sound layer. Limited consolidation and settlement over the lifetime of the road. No additional time required for surcharge effects.
Disadvantages	Significant quantities of excavated materials created. Land required for formation of sideslopes in peat and disposal of excavated materials. Difficulties in excavation and placing fill below water table. Normally demands high quality of fill material (low percentage of fines). Deep excavations may have effects on adjacent lands and structures. Unexcavated soft material below embankment may cause future settlements.
Risks	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.
Case Histories	F4, F6, N5, Sc4, Sc8, Sw6

8.5 Peat displacement

A number of related methods fall within this category and the following will be discussed within the section.

- 8.5.1 Progressive displacement
- 8.5.2 Partial excavation
- 8.5.3 Assisted displacement

8.5.1 Progressive displacement

A derivative of the standard replacement method is 'progressive displacement' or 'displacement' which has been carried out very successfully on many projects across the Northern Periphery in recent years and is acknowledged to produce similar results. The method is normally used where depth of peat to be replaced is beyond the economic limit of excavation and the weight of the intended road embankment is expected to be sufficient to displace the type of peat below.

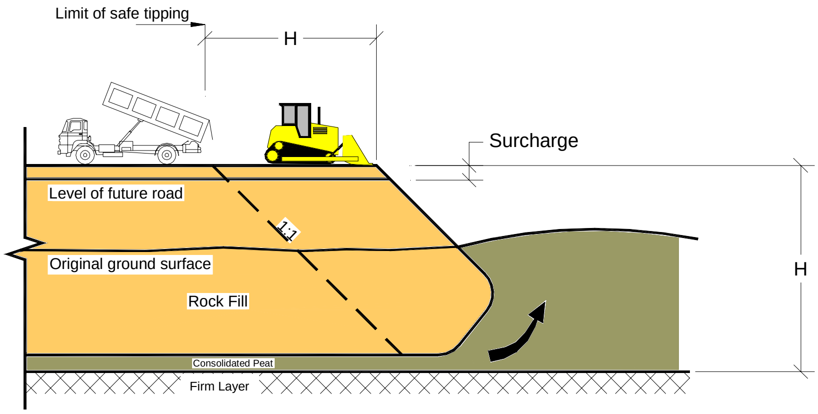


Figure 27. Long section through the Progressive Displacement Method. (P. Carlsten modified by G. Smith).

Essentially the displacement method involves the construction of a standard embankment up to the edge of the peatland and then an embankment drive across the peatland by end tipping normally aided by a surcharge. Some practitioners advocate an additional surcharge, 'a raised end', at the point of the advancing embankment to maximize the local displacement weight but this is not always used.

The action of the combined weight of the embankment and surcharge causes a shear failure in the peat ahead of the embankment and this results in the affected peat being displaced laterally, i.e. pushed to the side by the nose of the advancing embankment.

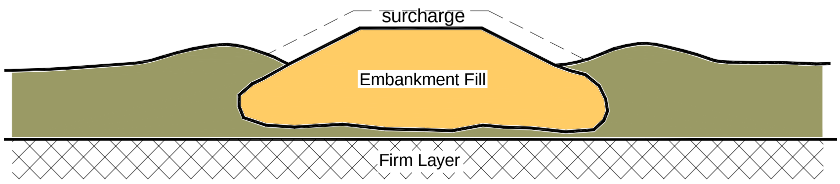


Figure 28. Typical cross section through an embankment formed by displacement. (P. Carlsten modified by G. Smith)

'Waves' of displaced peat are formed at the sides and front of the advancing embankment during the progressive displacement process and these can on occasion act passively to prevent the displacement continuing. They can also have an affect on adjacent structures and buildings even at some distance from the main axis of the displacement (effects at distances of up to 5 times the depth of the soft soil have been recorded) and these structures should be identified and considered before commencing a drive.

Once a progressive displacement is started in a peatland it will usually continue provided that the embankment height differential above the surface of the peatland is held constant through the addition of further fill material. It may however be necessary in some marginal locations to remove the developing wave in front of the embankment and deposit the material off the displacement line to ensure that the displacement can continue. The waves of heaved peat at the sides of the displaced embankment can aid the overall stability of the embankment by acting as pressure berms (see 8.6.2).

On completion of the displacement the surcharge is left in place for a period (normally months) to aid the consolidation of any trapped pockets of peat and to ensure that the completed embankment is 'bedded down' before the final road construction layers are placed.

The amount of displacement achieved during an embankment drive will be a consequence of a number of factors, all interdependent. The weight of the imposed embankment against the strength of the underlying peat, the volume and shape of the imposed embankment against the depth and character of peat to be displaced, the topography of the harder layers below the peat, as well as other the local environmental effects of each particular scheme. These conflicts need to be known and quantified before the quality of the displacement can be assured.

As with the replacement method, care has to be taken to avoid trapping pockets of peat below the embankment during the drive. Progressive displacement is best used when it is known that the topography of the underlying hard layer can permit the embankment to move forward downhill without trapping pockets of peat as shown in Figure 29. If the direction of advance of the embankment can be controlled 'downhill' in this fashion it is possible to prevent situations that would cause peat or other soft material to be trapped under the embankment on the 'uphill' side of the direction of travel.

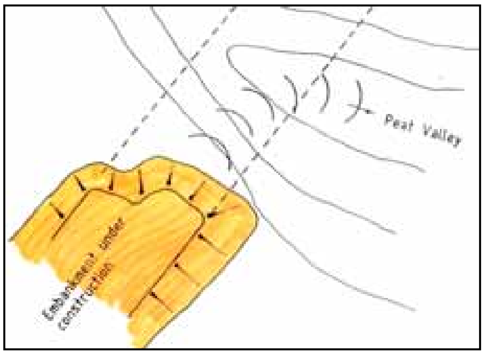


Figure 29. Downhill displacement .

It is normal practice to take proving cores through the completed embankment after displacement to check if the displacement has been successful. Where peat pockets are detected it is usual to either allow time for the trapped peat to consolidate under surcharge or to blast the material out from below the embankment by strategically placed explosives.

Table 13. Summary of the progressive displacement method.

Summary of the progressive displacement method	
Advantages	Well tried intermediate technologies. Should achieve a good bearing capacity on the displaced embankment construction. The displaced peat to the sides of the embankment can enhance the embankment stability. Good method for constructing a high embankment above a peatland.
Disadvantages	Better suited to amorphous peats. Fibrous peats may prove resistant to shear failure without assistance. Requires substantial quantities of fill material for the buried embankment. Requires longer construction time for displacement and surcharge affects to be effective. Normally demands high quality of fill material (low percentage of fines). Some limited consolidation and differential settlement can be expected over the lifetime of the road if peat pockets remain trapped below the embankment. The peat displaced during the procedure can cause heave effects on adjacent land and structures. Wide embankments may require significant materials to be displaced. Possible problem with culvert locations.
Risks	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.
Case Histories	F3

8.5.2 Partial excavation

Within the peat displacement method there is the derivative technique normally called "Partial Excavation" which can be mentioned in passing. This method is similar to the progressive displacement method in that it requires the embankment to progressively move across the bog under its own weight without mechanical compaction and is generally assisted by surcharge. It differs from the main progressive displacement method however in that it is assisted by excavating a manageable depth of peat in front of the nose of the embankment to reduce the amount of material to be displaced.

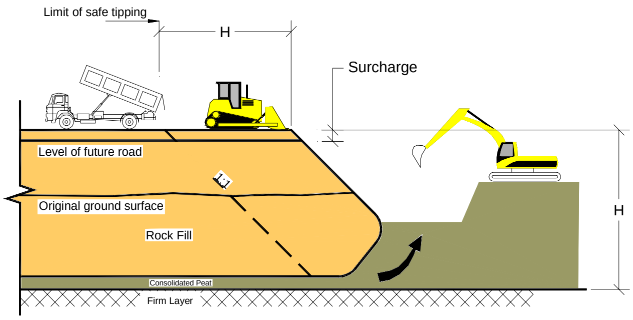


Figure 30. Long section for Partial Excavation. (P. Carlsten modified by G. Smith)

Partial displacement is normally used for the construction of wide embankments where the underlying peat is deep and beyond the limit of economic excavation. Under these circumstances standard progressive displacement techniques may not be fully effective due to the need to place sufficient weight across the full cross-section to achieve uniform displacement.

The partial displacement method is particularly useful where the top layers of the peat deposit are very fibrous or woody and underlain by a more amorphous peat. Where these layers exist they can act as a surface reinforcement to the peatland and resist the displacing forces induced by the imposed embankment. In these circumstances the fibrous layers can be excavated out and the lower levels thereafter displaced by the embankment assisted by a surcharge to suit the characteristics of the underlying peat. This method has been regularly and successfully used in Finland for replacement depths of 10-12 metres.

Table 14. Summary of the partial excavation method.

Summary of the partial excavation method	
Advantages	Well tried intermediate technologies. Should achieve a good bearing capacity on the displaced embankment construction. The displaced peat to the sides of the embankment can enhance the embankment stability. Good method for constructing a high embankment above a peatland..
Disadvantages	Better suited to amorphous peats. Fibrous peats may prove resistant to shear failure without assistance. Requires substantial quantities of fill material for the buried embankment. Requires longer construction time for displacement and surcharge affects to be effective. Normally demands high quality of fill material (low percentage of fines). Some limited consolidation and differential settlement can be expected over the lifetime of the road if peat pockets remain trapped below the embankment. The peat displaced during the procedure can cause heave effects on adjacent land and structures. Wide embankments may require significant materials to be displaced. Possible problem with culvert locations.
Risks	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.
Case Histories	F3

8.5.3 Assisted displacement

There are currently 2 main methods, water jetting and blasting, that can be used to assist a displacement to take place:

Displacement assisted by water jetting

As already mentioned fibrous peat bogs can prove to be more resistant to the standard form of displacement. The organic fibres act as reinforcements against the displacement and they have on some occasions prevented the required shear taking place. The reinforcement effect of the fibres can however be overcome by increasing the water content of the bog immediately ahead of the embankment by water jetting. This causes a reduction in the peat's shear strength so enabling it to be displaced more easily.

In this method water jet lances are pushed into the base of the peat ahead of the embankment front. The lances are then slowly withdrawn whilst water is pumped into the ground so maximising the volume of peat treated.

Table 15. Summary of displacement assisted by water jetting.

Summary of displacement assisted by water jetting	
Advantages	Used with progressive displacement and partial excavation methods. Established intermediate technology. Does not require peat excavation. Should achieve a good bearing capacity on the displaced embankment construction.
Disadvantages	As mentioned for progressive displacement and partial excavation methods.
Risks	Excavation in peatland. Peat displacement. Effects on adjacent structures. Possible trapped peat below embankment.
Case Histories	None

Displacement assisted by blasting

Displacement can also be assisted through controlled blasting of the peat ahead of the advancing front. The method was used extensively in the bog-blasted motorway schemes in Northern Ireland in the 1960's and "toeshooting" is still occasionally used in the Northern Periphery.

In recent years other methods of embankment construction have proved more cost effective than blasting but the method still remains a useful tool for site specific applications. Where it has been used blast assisted displacement has normally only been used in open areas that are free of structures and utilities apparatus where explosives can be used safely.

There are essentially 3 ways in which blasting can assist displacement, trench shooting, toe shooting and underfill blasting, but there are many derivatives to these basic methods that use particular explosive charge patterns for specific end results. This report will confine its remarks to the 3 main methods.

Trench shooting

This method has been successfully used where peat has been less than 6m deep and stiff enough to have stable side slopes. It is not a true displacement method but rather a blast induced excavation. One or more rows of explosive charges are pushed down into the base of the peat deposit (sometimes assisted by jetting) and fired to produce an open trench in which fill can be placed immediately. Experience indicates that to be effective the charges should be placed at centres equal to approximately half the depth of the peat.

Toe shooting

This method has been used in peatlands considered suitable for displacement. A standard progressive displacement exercise is executed until the advancing embankment starts to build up a peat wave ahead of the embankment front. Rather than excavate this mass of material out and dispose of it a series of charges are pushed into the peat ahead of the embankment toe and fired. The progressive displacement is then continued until circumstances again require further use of blast assistance.

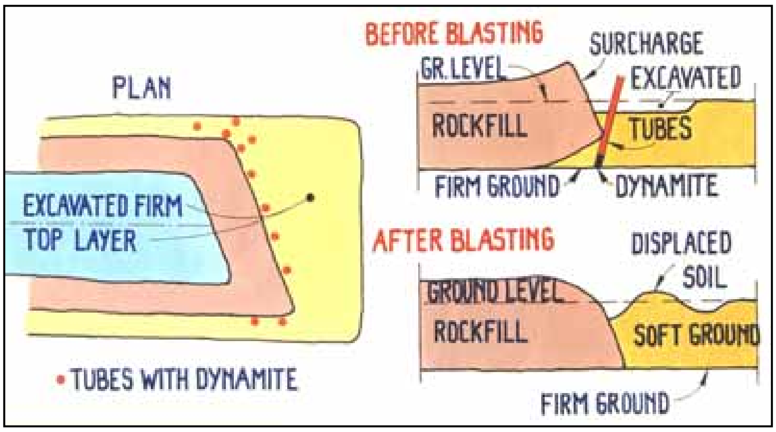


Figure 31. Displacement assisted by blasting, Norwegian Road Research Laboratory 1990.

In Norway for example a typical "blast assistance" for an embankment advance of 5m would involve a charge of around 5kg of explosive being installed in tubes every 2-3m along the leading edge of the fill. When the full displacement is completed the finished longitudinal edges would also be 'blast assisted' to make sure that they were in firm contact with the hard strata below.

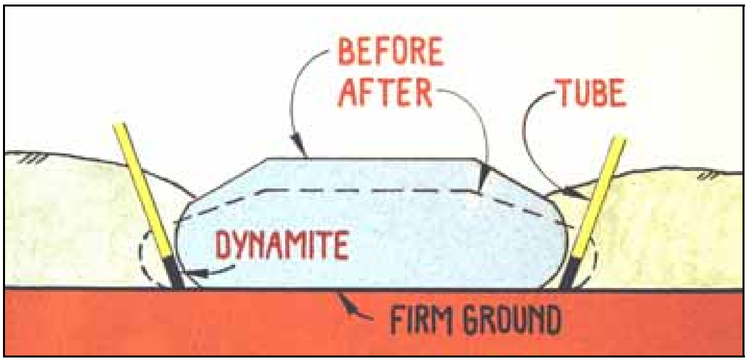
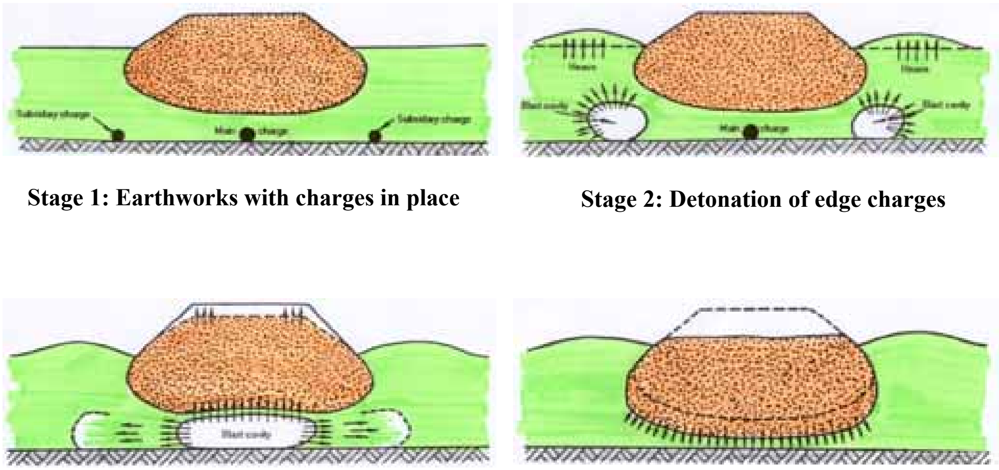


Figure 32. Blasting after completion of filling, Norwegian Road Research Laboratory 1990.

Underfill blasting

Underfill blasting has been used where the peat deposit has had a very fibrous top mat resistant to normal displacement methods or where the peat has been very deep. The method involves pushing charges into the base of the peat and constructing a floating embankment on top of the peat deposit. At an appropriate time in the settlement of the imposed embankment the charges are fired and the peat below the embankment forced laterally out from below the embankment allowing the embankment to drop onto the sound base layer.



Stage 1: Earthworks with charges in place **Stage 2: Detonation of edge charges**

Stage 3: Detonation of main charges **Stage 4: Resultant settlement of earthworks**

Figure 33. Sequence of underfill blasting.
Source: Road Research Laboratory, Road Research Technical Paper No 40

A sequence of delayed charges are employed to ensure that the designed earthworks movements take place. At least one row of charges is placed directly below the centreline of the new embankment (by jetting or by driving casings) to produce the void for the embankment to drop into. A further series of rows are placed along the edges of the fill to form the voids into which the peat can be displaced. In practice the edge charges are fired first to produce the space for the displaced peat and then the underfill charges fired immediately after to create the void for the embankment to drop into.

Table 16. Summary of assisted displacement by blasting.

Summary of assisted displacement by blasting	
Advantages	Used with progressive displacement and partial excavation methods. Established intermediate technology. Does not require peat excavation. Should achieve a good bearing capacity on the displaced embankment construction.
Disadvantages	As mentioned for progressive displacement and partial excavation methods. Use of explosives. Can only be used in clear open sites with no utilities, etc.
Risks	Use of explosives. Excavation in peatland. Peat displacement. Effects on adjacent structures. Possible trapped peat below embankment.
Case Histories	None

8.6 Peat left in place

This section aims to outline those methods of road construction over peat which use the strength of the in-situ peat to support the intended loads. The excavation, replacement and displacement methods previously discussed all rely on fill materials being readily accessible to create an embankment through the peatland area. On projects with large scale earthworks this could involve importing very large quantities of new material, the possibility of difficult logistics and construction sequencing, with consequent results of high costs.

Methods that leave the peat in place and avoid the disadvantages of bulk earthworks are now becoming increasingly more attractive to engineers as road construction budgets reduce and more cost effective solutions are sought. Environmental and waste minimisation considerations are also adding to the arguments for methods that build on the peat in place. At the date of writing the UK Government is levying a tax of 2.5 Euros on every tonne of new quarry materials used in construction works. This "Aggregate Levy" is applied to all raw quarry products or products processed from them; soils, sands, gravels, aggregates, rockfills, asphalts, cement, etc except those excavated on the road line or from recycled materials. It would appear likely that the principle will be extended to all Partner areas in the near future if only for environmental reasons.

Methods that leave the peat in place are therefore worth considering and this section will look at 5 groups of techniques under the heading of 'Peat left in place' that utilise the underlying peat as a load bearing layer. These are:

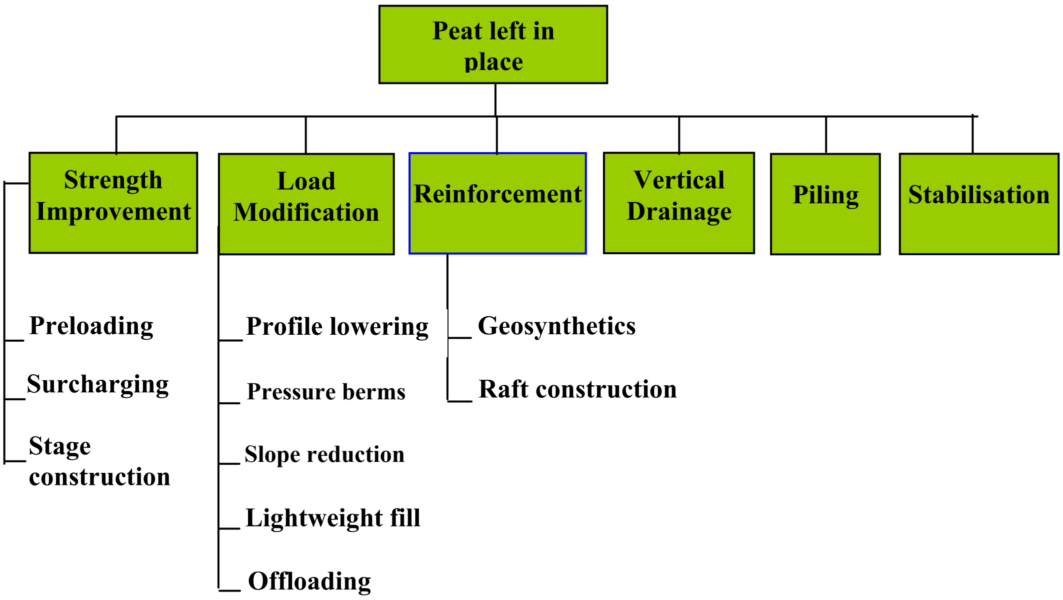


Figure 34. Peat left in place.

8.6.1 Strength improvement

Preloading

Preloading is a method of improving the strength of the in-situ peat subgrade (ie bearing capacity) ahead of the main works so that it can be capable of supporting the intended permanent embankment, pavement layers and traffic. Peat is well suited to the preloading method as it has a very high permeability in its natural state and compresses in a relatively short time under load when compared to other engineering soils. As the peat matrix deforms under load its permeability and compressibility decrease whilst its shear strength increases.

The principle of the preloading method is relatively simple. A load in excess of what is required is placed on the peat and allowed to settle until it reaches the predicted in-service settlement for the intended load. (This can be done with or without a surcharge or vertical drainage.) Once this settlement has been reached any excess load is removed and the service load left on the strengthened foundation at its final in-service settlement.

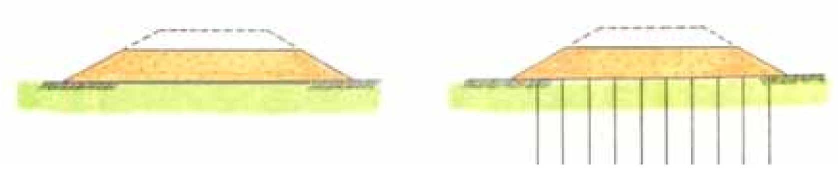


Figure 35. The use of preloading.

Table 17. Summary of strength improvement through preloading.

Summary of strength improvement through preloading	
Advantages	Minimises embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil.
Disadvantages	Embankment filling rate limited by soil strength increase. Time needed for preloading can extend construction time. Preloading materials may need to be brought on site earlier than required and require some double handling. Requires comprehensive site investigation and laboratory testing ahead of works and onsite monitoring system to ensure that the required settlements are being achieved. Is a 'floating' road method and is best suited to thin embankments.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	Sw3

The Preloading method is normally carried out accompanied by a temporary additional load (surcharge) to accelerate the rate of settlement and consolidation of the embankment.

Surcharging

The amount of surcharge needed to achieve the increased rate of settlement is a function of a number of things such as the type and depth of peat, its moisture content, ground water levels, distribution of load, etc. Each installation will invariably be unique requiring a geotechnical assessment of stability, settlement and increase in strength but a general Swedish 'rule of thumb' normally aims for an unloaded in-service embankment weight of 80% of the surcharged embankment after taking buoyancy effects into consideration. This equates to a nominal 25% surcharge over the weight of the final embankment ignoring the effects of buoyancy over time.

Its application to road construction can be illustrated simply in a time v consolidation graph:

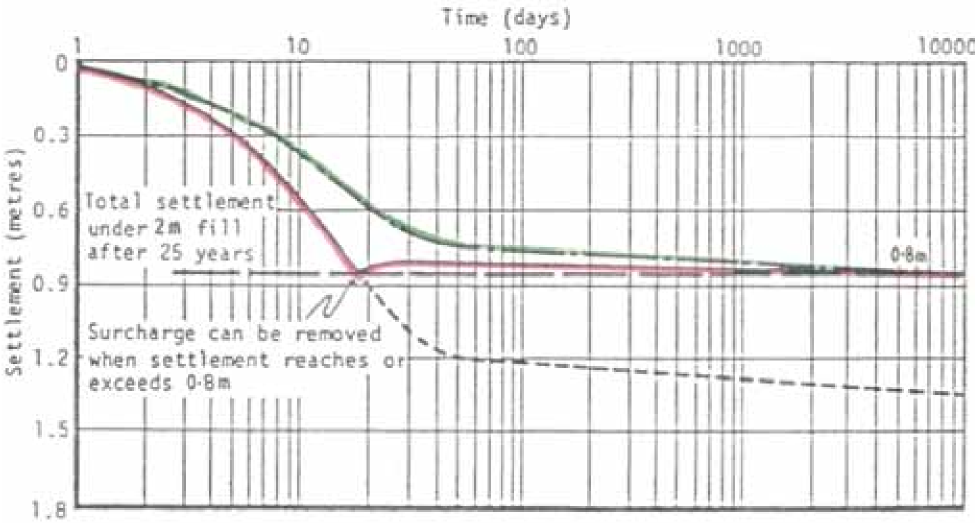


Figure 36. Time v consolidation curves showing the Application of surcharge.
Source: The Muskeg Engineering Handbook. National Research Council of Canada.

The top curve in **green** is a typical time-settlement curve for a 2m thick embankment placed over a peat deposit. Its settlement without surcharge would be approximately 0.8m over 25 years. This settlement however can be produced more quickly by placing an additional temporary 1.5m of fill on the service embankment during construction for about 20 days to force a faster rate of settlement – as shown by the dashes on the lower curve. After the required settlement of 0.8m has been reached the peat will have achieved a sufficient strength to support the normal embankment and the surcharge of 1.5m can be removed with the resulting future settlement of the road theoretically marginal i.e. the **red** curve.

Table 18. Summary of strength improvement by surcharging.

Summary of strength improvement by surcharging	
Advantages	Improves the bearing capacity of the underlying peat so that it can support the weight of the in-service embankment. The times for primary consolidation and secondary compression of the underlying peat can be accelerated. Stage construction normally needed in the case of higher embankments
Disadvantages	The time needed for surcharging can extend construction time. Surcharge materials may need to be brought on to site earlier than required and require double handling as a consequence. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	F3, Sw1, Sw3

Stage construction or ‘stage loading’

It is of course extremely difficult to construct a 4, 3, or even a 2 metre high surcharged embankment on weak peat without causing a shear failure in the underlying peat. As a consequence of this all of the authorities that use the preloading technique generally apply their fill materials in steps employing a “stage loading” procedure to construct their embankments. This means that each layer of the embankment is only placed when a suitable gain in strength has been achieved in the underlying peat from consolidation such that it will be able to withstand the new layer without failure.

Preloading with a surcharge is generally considered to be the most economical method of road construction in the Northern Periphery despite ending up with the apparent disadvantage of a “floating” road. The method is usually restricted to thin embankments close to the natural ground and normally means a limit of embankment heights to around 2-3m above the adjacent peatland level. It is normal practice to form the surcharge loads from temporary stockpiles of construction materials planned for use elsewhere in the permanent works such as sub-base or roadbase materials. This effectively means that the surcharges are cost neutral in the overall costing of the project.

As already discussed fibrous peats have excellent initial properties of high compressibility and permeability that lend themselves ideally suitable to stage loading. Amorphous peats can still benefit from the technique but stage timescales can be expected to be that much longer. The rate of loading peat by stages is normally determined by the rate of dissipation of porewater from the underlying peat matrix. This can be estimated from the basic peat properties but is best done by direct reading of piezometers in the field.

The first layer on the ground in stage loading is normally made thick enough to withstand the immediate construction traffic yet thin enough to prevent local shear failure of the peat below. For a fibrous peat this means that it is normally safe to have a first stage load of around 20kPa as a working platform (approx 1m of gravel) and subsequent layers of fill only placed on this when 70-80% of the primary consolidation of this layer has been reached. A similar loading philosophy is used for the subsequent stage layers and any final temporary surcharge used. Within the Northern Periphery, Iceland regularly uses preloading for the construction of thin embankments over peat. Their “Rules” for staged construction in a preloading operation for a new road are as follows:

Icelandic rules for preloading operations:

- Retain the existing fibrous surface mat of the peatland if possible as this offers a good reinforcement effect. (If the top layers of peat are more hummified a separating grade geotextile may be required before loading operations commence.)
- Excavate deep side ditches on both sides of the new road line 15m to 17m off the proposed centreline in advance of the roadworks in order to establish a stable groundwater regime for the construction and maintenance of the new road
- Use staged construction for the embankment layers with the first layer of fill restricted to a load of 20kPa on the bog surface. Subsequent construction layers to be limited to loads of 30kPa.
- Allow each layer to consolidate by 50% of its predicted settlement before placing further layers. Peat normally consolidates quickly and this usually means that further layers can be placed in about 4 weeks.

Figure 37. Icelandic road over peat showing advance ditches.

Considerable settlements can however be expected during the stage loading operations and these should be known and their effects understood with reasonable accuracy at the design stage so that they do not come as a shock to the Resident Engineering staff on site. Preloading is generally considered to be a cost effective solution for peat depths of up to 4m. It can of course be used for greater depths than this but the surcharge required will be that much larger and take longer to achieve the desired effect.

Table 19. Summary of strength improvement by stage construction.

Summary of strength improvement by stage construction	
Advantages	Produces sequential gains of strength in the peat. Minimises future secondary compression settlement of the new embankment. Higher embankments can be constructed without shear failure in the underlying peat. Does not require peat excavation, disposal or the need for additional land for storage of spoil.
Disadvantages	The time needed for the various stages to take effect can extend the embankment construction time. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved before the next layer is placed.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	Sw1

8.6.2 Load modification

The second group of methods that leaves the peat in place below the new embankment is 'load modification'. This group is concerned with altering the load distribution of the proposed embankment to better suit the existing strength of the peat.

Profile Lowering

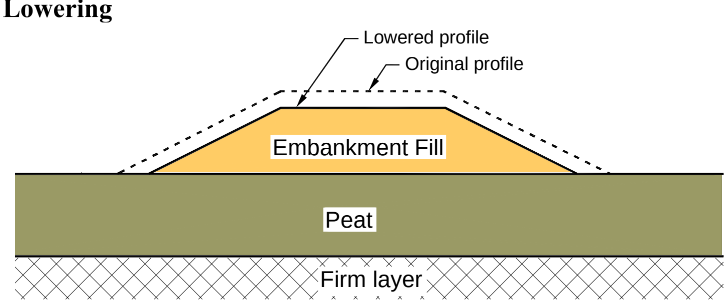


Figure 38. Cross-section through a profiling lowering exercise.
(P. Carlsten modified by G. Smith)

‘Profile lowering’ essentially means that the designer of the road amends the route vertical alignment to suit the weak soil conditions and lowers the intended embankment height across the peatland to an acceptable level for the strength of the underlying peat (normally no more than 3m above the peatland level).

The method, like avoidance, constitutes a change to the designer’s preferred road alignment and as a consequence any decision is best made early in the design process to minimize abortive work. On large works it may be possible to ‘optimise’ the applied embankment height in consultation with the design engineer to produce an optimum geotechnical solution for crossing a particular peatland but this is unlikely to be the case in most low volume road design scenarios.

Profile lowering can be extremely cost effective both in time and materials and is certainly worth considering for schemes crossing peatlands.

Table 20. Summary of load modification by profile lowering.

Summary of load modification by profile lowering	
Advantages	Reduces the quantities of fill material required. Reduces embankment loadings on the underlying peat. Reduces the amount of land required.
Disadvantages	Requires a modification of the designer’s preferred alignment. May not be possible if bridge clearances or waterway areas are critical. May give problems with bearing capacity of road embankment
Risks	Loading of peatland. Bearing capacity.
Case Histories	None

Stabilising Berms

Stabilising berms, also known as ‘counterweight berms’ or ‘pressure berms’, are used to widen the base of an embankment, distribute the imposed embankment load over a greater surface area and increase the factor of safety of the embankment against slip circle failure. As with all structures over peat stabilizing berms must firstly satisfy their own stability requirements and be loaded in a staged manner to remain in a stable condition at all times.

By widening the base of the embankment and providing a counterweight to the main embankment load the failure slip circle of the combined arrangement is forced deeper and longer into the peat foundation so improving the overall stability.

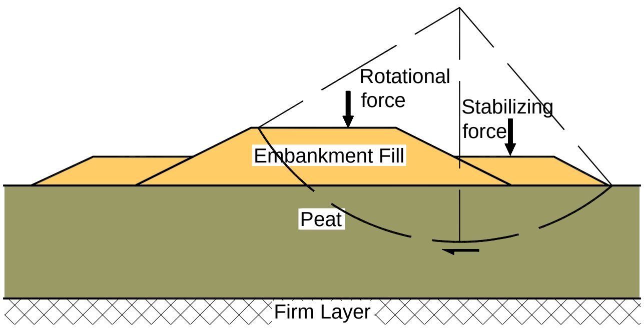


Figure 39. Cross-section showing the use of stabilising berms (G. Smith).

A typical embankment failure in peat usually takes the form of a rotational slip where the failure surface approximates to a circular arc in cross section. The construction of stabilising berms alongside the embankment has the effect of adding a counterbalance weight to the failure slip circle and this in turn has the effect of modifying the critical shear surface into a deeper and longer failure arc. To be effective a berm should be sufficiently wide to ensure that its centre of gravity acts through the counterbalance side of the slip circle. Any material can be used for the construction of a stabilizing berm (the writer has seen excavated peat being used as an emergency measure) but where they are deemed necessary berms are best constructed at the same time as the main embankment using the same construction principles.

Table 21. Summary of load modification by pressure berms.

Summary of load modification by pressure berms	
Advantages	Improves stability. Increases the depth and length of the critical slip circle. Low grade fill material (even peat) can be used as fill mass in berms.
Disadvantages	Requires additional fill material and additional land for the wider construction. Increases the overall weight of the embankment. Consolidation settlements may be increased as a result of the spread of load from the pressure berm.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	F1

Slope Reduction

'Slope reduction' is similar to the addition of pressure berms and is again intended to produce a wider embankment, a greater distribution of load over the foundation area and a longer more deep seated potential failure slip circle in the underlying peat.

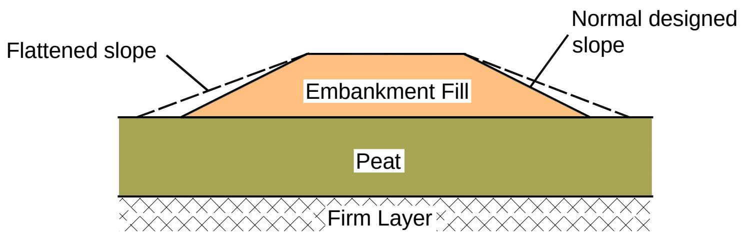


Figure 40. Cross-section through a slope reduction exercise.
(P. Carlsten modified by G. Smith)

In this method the side slopes of the intended embankment are flattened to a shallower gradient to widen the overall width of the embankment across the peatland.

Table 22. Summary of load modification by slope reduction.

Summary of load modification by slope reduction	
Advantages	Improves stability. Increases the depth and length of the critical slip circle.
Disadvantages	Requires additional fill material and additional land for the wider construction. Increases the overall weight of embankment. Consolidation settlements may be increased as a result of the spread of load from the wider slopes.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	None

Lightweight Fill

Lightweight fill is primarily used to reduce the overall weight of an in-service embankment and thereby reduce the permanent stresses on the foundation. Embankments constructed with a lightweight fill core are usually installed in conjunction with a surcharge load to accelerate consolidation and settlement and once the designed settlement has been reached the surcharge is removed leaving the finished in-service embankment on a strengthened subgrade.

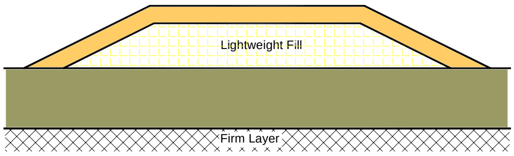


Figure 41. Cross-section through a lightweight fill installation. (G. Smith)

Lightweight fills are normally only used as part replacements of embankments due to their high cost and are generally restricted to those sections that cannot be economically addressed by other means. A good lightweight fill material, in addition to being light, should also be durable, resistant to decay, be easy to place and compact, have a good compressive strength with low compressibility and be environmentally friendly. A table of some of the lightweight materials currently being used in the Northern Periphery is shown in Table 23. The low densities of some lightweight products is not always a virtue in installations over peatlands as their light weight can pose buoyancy problems particularly with high water tables.

Lightweight forestry by-products such as bark, woodchip and sawdust wastes from the timber industry have regularly been used a lightweight fills in the Northern Periphery. These materials are normally installed with a covering layer of a low permeability material, such as clay or topsoil, in an effort to keep them moist and isolate them from the effects of the atmosphere as they can be prone to decay and spontaneous combustion if incorrectly handled.

The most popular lightweight materials today are the specifically manufactured lightweight products LECA and EPS. The major advantage of EPS is its low density of 20kg/m³ although it is generally given a higher design value of 100kg/m³ for stability and settlement calculations to allow for some water absorption over time. EPS blocks are easy to transport and handle (up to 100m³ can be transported on a single vehicle) and their only disadvantage, other than their production costs, appears to be that they can be susceptible to petrol and chemical attack. This is usually catered for in careful detailed design. EPS for roadworks is usually specified at a compressive strength of 100kPa to limit local pavement deflections under wheels. The completed installation is normally capped with a 100-150mm reinforced concrete slab topped by a 300mm gravel road base to try to tie the construction together and provide a heat storage mass to counter any variations in icing conditions along the finished carriageway between lengths of normal construction and EPS blocks.

Table 23. Table of typical lightweight fill materials.

Material	Dry Density kg/m³	Bulk Density kg/m³	Comments
LECA	300-900	650-1200	Manufactured product. Lightweight aggregate produced by heat expansion of clay pellets. Range of densities due to water absorption. Normally requires 0.6m of road construction above. May be difficult to compact if unconfined.
PFA	700-1400	1300-1700	By-product of coal fired power stations, Naturally cementitious, especially useful in backfills to bridge abutments.
Slag	1000-1400	1400-1800	By-products of heavy industry, steel furnaces, etc. Generally at the 'heavy' end of lightweight materials. Leachates can pose environmental problems.
Aerated slag	500-1000	1100-1700	Foamed by-product formed by quickly quenching molten slag in water.
Volcanic ash	650-1000	1400-1700	Natural material (particularly useful in Iceland).
Bark/woodchip	100-300	800-1000	Fresh wood is not recommended as it is difficult to compact. Aged bark can have good properties and be beneficially used but can give leachate problems in sensitive environments.
EPS	20	100 for design	Manufactured product. Extremely light, generally produced in blocks, relatively expensive, 100kPa minimum compressive strength. Installations are usually capped with a concrete slab. Requires protection from petrol, fire and UV light.
Concrete waste	500-600	750-100	Waste concrete products from precast concrete production, e.g. broken blocks, no fines concrete, etc.
Foamed concrete	600-1800	1000-1800	Manufactured product. Pre-foam added on site to ready-mixed mortar, 4MPa minimum compressive strength
Compressed peat bales	200	600-800	Past installations still exhibiting 20% buoyancy after 10 years submergence, not generally available.
Horticultural peat	200	500-800	"Garden peat bags", laid flat as bulk fill, assume 800kg/m3 for long in situ density.
'Hasopor'	100-500	100-500	Foamed glass product manufactured from waste cathode ray tubes, stable, inert material, compressive strength 6-12 MPa.
Waste tyres bales	500-650	500-650	Waste tyres compressed into bales and bound with galvanized wires.

Table 24. Summary of load modification by lightweight fill.

Summary of load modification by lightweight fill	
Advantages	Does not require as high a bearing capacity from the peat foundation. Usually does not need the underlying peat to be strengthened. Lighter embankment construction generally means less future settlement.
Disadvantages	Purchase price and transport of the specialised lightweight materials. Design and placing of lightweight materials may require special arrangements. Environmental considerations particularly with groundwater. Bearing capacity of the lightweight embankment may be limited
Risks	Placing lightweight material below ground water table. Bearing capacity.
Case Histories	F1, F8, F12, N3

Offloading

'Offloading' basically involves the removal of heavyweight material from an existing road construction and its replacement with something lighter. The aim of offloading is to produce a reduction in load on the underlying peat preferably to a level within its existing bearing capacity. Normally designers aim to effect a reduction of load of 1/2 to 1/3 of the original embankment loading with an intention of producing a reduction in load on the underlying peat of between 1/2 to 1/3 of the original.

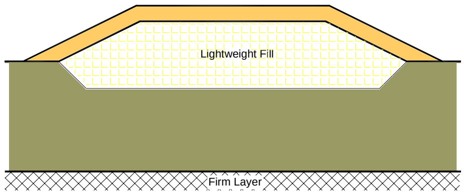


Figure 42. Illustration showing the 'Offloading' principle.
(P. Carlsten modified by G. Smith)

If these unloading ratios can be achieved the new carriageway can be expected to be relatively settlement free for the rest of its service life.

Table 25. Summary of load modification by offloading.

Summary of load modification by offloading	
Advantages	Does not require as high a bearing capacity from the peat foundation. Usually relies on the underlying peat having generated a sufficient bearing capacity to support the planned in-service embankment. The reduced embankment weight generally means minimal future settlement. No additional time required for surcharge effects.
Disadvantages	Purchase price and transport of the specialised lightweight materials. Design and placing of lightweight materials may require special arrangements. Environmental considerations particularly with groundwater. Bearing capacity of the lightweight embankment may be limited
Risks	Placing lightweight material below ground water table. Bearing capacity.
Case Histories	F7, F9, F10, F14, N1, N2, Sc5, Sc7, Sc9, Sw2, Sw5

8.6.3 Reinforcement

Embankments can be reinforced by a number of materials each governed by their own particular technologies. The area of embankment reinforcement is probably one of the more dynamic areas of research in road construction and new manufacturers and new materials regularly appear in the technical press.

Five areas will be considered in this section

- Geosynthetics & geogrids
- Timber raft construction
- Concrete raft construction
- Galvanised steel sheeting
- Steel mesh reinforcement of pavement layers

Geotextiles & geogrids

A great deal of discussion has centred around geotextiles and their application to the two types of road construction over soft ground, i.e. the 'thin' construction of roads and pavements and the 'thicker' construction of embankments.

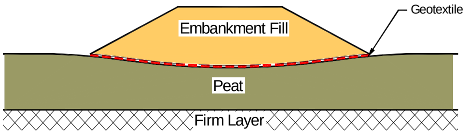


Figure 43. Thick embankment (reinforcement) (G. Smith)

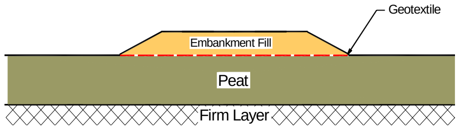


Figure 44. Thin embankment (separator)

What is generally accepted now is that for thin fills the geotextile will act as a separator and filter, and the particular material should be chosen with these properties in mind. In the case of thicker fills the geotextile or geogrid will perform more of its true reinforcement role and a suitable grade of material will require to be selected. In this case it will be necessary for the designer to establish that there will be sufficient friction generated between the reinforcement and fill and underlying soil to resist the forces created.

The installation of a geotextile or geogrid does not affect the long term consolidation settlement of an embankment or its overall factor of safety but it does have some appreciable short and medium term benefits. In particular it has the advantage of assisting the local stability of the embankment during the construction phase by decreasing the rate of spread of the fill material on the surface until the foundation soil is strong enough to support the load itself. The geotextile/geogrid should however only be considered as a temporary supplement to the strength of the foundation soil to allow time for the soil to gain sufficient strength to support the embankment in the long term.

Table 26. Summary of embankment strengthening using geotextiles and geogrids.

Summary of embankment strengthening using geotextiles and geogrids	
Advantages	Limited site disturbance. Easy to install. Provides reinforcement effect to the base of embankment for the short and medium term. Aids stability. Can reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. No excavation, disposal or need for additional land for storage of spoil.
Disadvantages	The overall settlement of the embankment is not reduced. The geotextile/greogrid can be damaged by construction equipment. Creep may affect the long term performance of the geotextile. Use of geogrid may need higher quality fill material (interlocking).
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	F9, F11, F12, N7, N8, N9, Sc6, Sc10, Sw3

Timber raft construction

Raft construction using local vegetation is the oldest method of strengthening embankments over peat. The technology has been around for many years and involves laying an interlocking platform of reinforcing materials on the peatland surface to support and distribute the loads of the new embankment until such time as the underlying peat can gain sufficient strength to support the embankment on its own.

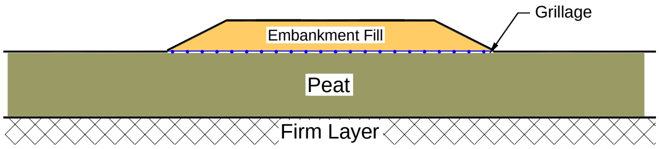


Figure 45. Cross-section showing a timber grillage. (G. Smith).

Historically there have been many types of timber reinforcements, from simple brushwood mattresses and fascines through to the substantial designed steel pinned grillages fabricated with selected timber logs. All have the same aim of preventing local punching shear of the fill material into the peatland and distributing the embankment load across its surface.

The most basic form of platform to date has been a 150mm-250mm thick mattress of brushwood laid directly on to the peatland. The design of this structure varied according to local tradition and available resources but essentially comprised a mat of criss-crossing branches (Spruce or similar) to build up a carpet of vegetation capable of supporting the gravel fill without failure. Protective mattresses have similarly been made using 'fascines' as structural members. In this method bundles of woody material (hazel sticks were popular in Scotland) were tied together to form approximately 3m long bundles and 150mm-250mm diameter. These were either laid simply alongside each other on the peatland surface or tied in a grillage at 1.2m centres and backfilled with sand.

Timber grillages can be considered to be the heavy end of brushwood mattresses and are designed to provide a resistance to bending in the base of the embankment. In their simplest form they can comprise a single platform of logs (corduroy) laid side by side at right angles to the road line as shown in the photograph of a road widening over peat below.

Figure 46. Photograph of a simple timber corduroy platform in a road widening.

And at their most complex fully designed grillages can comprise pinned structures of logs laid orthogonally to each other (generally 60º) and dowelled together with steel pins.

Figure 47. Photograph of an installation of a 2 layer pinned timber grillage.

Practice has shown that all of the above mattresses and grillages must be pushed down into the water table by the embankment within 6 months of installation to prevent decay. If they are not completely submerged in this fashion it is likely that the timber elements will rot and the platform decompose.

Timber grillages are currently not so popular as geotextiles or geogrids due to their high labour input and cost of timber but their inherent stiffness can provide better load distribution properties than high strength geotextiles. At the date of writing the competitiveness of geotextiles & geogrids and their ease of installation make them very attractive but grillages should not be totally forgotten as many roads in the Northern Periphery still rest on grillages and corduroys and these will require maintenance or widening in the future.

Table 27. Summary of embankment strengthening using timber raft construction.

Summary of embankment strengthening using timber raft construction	
Advantages	Limited site disturbance. Relatively easy to install. Provides reinforcement effect to the base of embankment for the short and medium term. Aids stability. Can reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil.
Disadvantages	The overall settlement of the embankment is not reduced. Can be damaged by construction equipment during placing of embankment fill. High element of manual labour required for fabrication of the raft. Timber raft must be submerged.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	F1, F2, F5, N4, Sc2

Concrete raft construction

Reinforced concrete rafts or slabs were used very successfully in Scotland and Ireland from the 1920's through to the 1950's. They were generally built in a series of slabs 200mm thick, doubly reinforced with edge strengthening and were either constructed directly on to the peatland surface or on top of a regulating layer of sub-base material. They were stiff structures, much more so than other strengthening systems such as mattresses, geosynthetics or grillages and needed minimum road construction layers to distribute the traffic loads. Many of these concrete rafts still remain in service over deep blanket bogs deposits in northern Scotland providing a stable load bearing platform for modern traffic.

A modern derivative of the reinforced concrete raft is the lightweight foamed concrete raft and a photograph of a typical installation (from The Netherlands) is shown in Figure 48.

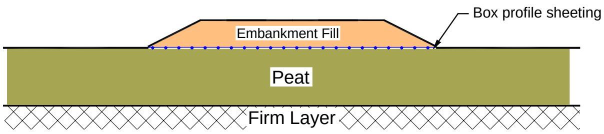
Figure 48. Photograph of lightweight concrete road.

Table 28. Summary of embankment strengthening using concrete rafts.

Summary of embankment strengthening using concrete rafts	
Advantages	Limited site disturbance. Provides long term stiff foundation for the embankment. Aids stability. Reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil.
Disadvantages	Overall settlement of the embankment is not reduced. Curing time for concrete. High element of manual labour required for fabrication of the raft.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures.
Case Histories	Sc1, Sc3, Sc7

Galvanised steel sheeting

A recent development in rafted embankment construction over peatland is the use of box profile galvanised steel sheeting as the reinforcing element. Installations of this method in the Partner areas to date have been confined to forest haul roads in Finland and Russia but their results appear promising enough to warrant trials on low volume public roads. These installations (since 1986) have used "Geoprofile" sheeting manufactured by Rautaruukki Oy of Oulu.



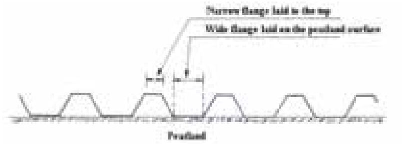


Figure 49. Cross section through embankment *Figure 50. Geoprofile sheeting*
Source: Rautaruukki Oy, "Geoprofiilien käyttö tierakenteen pohjanvakvistuksessa"

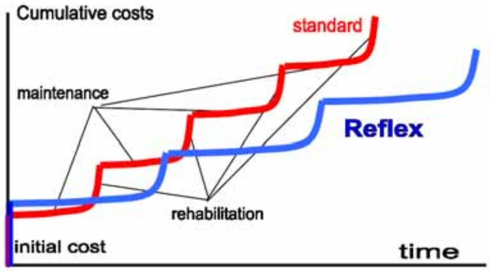
Normally 7mm corrugated steel plate is used with a zinc coating for corrosion protection. Sheets can with be installed transverse or parallel to the road line. It is considered that sheets installed crosswise give a better bearing capacity and rutting resistance whereas sheets installed along the roadline appear to be better at dealing with longitudinal depressions and frost heave.

Figures 51. & 52. Photographs of road construction over peatland with "Geoprofile"
Source: Eranti Engineering Oy

Steel mesh reinforcement of pavement layers

The reinforcement of pavement layers using steel fabric mesh is now a well established science following research carried out under the EU REFLEX project ("Reinforcement of Flexible Road Structures with Steel Fabrics to Prolong Service Life"). REFLEX started in March 1999 and lasted until August 2002 with the objective of developing technologies for road reconstruction and rehabilitation using steel reinforcement to improve 'whole life' costs of roads and extend the working life of road pavements.

Prior to the project it was expected that the use of steel meshes could increase the bearing capacity of the pavement but this was not borne out by the research. REFLEX did show that the use of meshes could improve the service life of a road and reduce maintenance and rehabilitation costs over whole life.



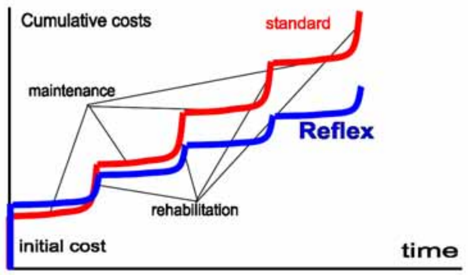


Figure 53. Extension of service life *Figure 54. Reduction in maintenance*
Source: REFLEX Final Report T9:02

Figures 53 and 54 show how steel meshes prolong service life and reduce maintenance costs by reducing the frequency and cost of individual rehabilitation measures across the life of the pavement.

Figure 55.
Source: Kalervo Niva, FinRa

Figure 56.
Source:REFLEX Final Report T9:02

8.6.4 Vertical drainage

The primary function of vertical drainage is to shorten drainage paths in a soil to produce an acceleration of the primary consolidation process and thereby a gain in strength. The process usually consists of a grid of drainage elements (usually geotextile bands) driven vertically into the soil by a mandrel which is then retracted leaving the drain in place. The area to be treated is generally prepared with a surface free draining layer 1m thick that acts both as a working platform and horizontal drain. Vertical drains are installed through this layer in one of two patterns, triangular or square, of which the square grid is generally the easiest to control but has the largest drainage path for equal centres. The principle of the process can be demonstrated in the figures 57 & 58 below.

Band drains invariably buckle within the soil as the soil mass settles and as a result it is established practice to calculate any drainage or consolidation rates below the embankment using the 'buckled drain discharge capacity', usually taken to be around 75% of the normal discharge capacity.

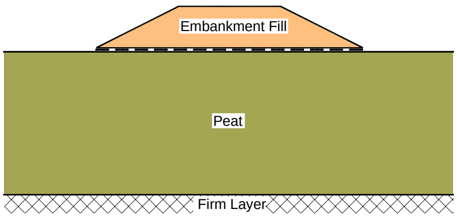


Figure 57. Normal cross-section (G. Smith)

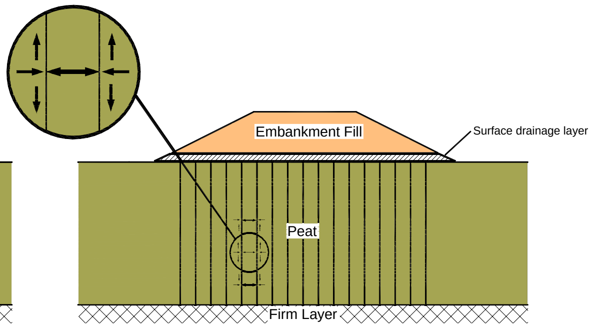


Figure 58. Cross-section with vertical drainage.

In a loaded peat the excess pore water pressures generated by the loading process have to migrate substantial distances before they can dissipate or find a suitable drainage layer. In the case of peat below an embankment this can mean a drainage route of tens of metres whereas in a vertically drained soil the maximum distance to a drainage path is half the horizontal distance between the drains (normally around 1.0m to 1.5m). This shorter drainage distance means that any excess pore water pressures can be released more rapidly from the peat thereby quickening the transfer of the embankment load to the soil skeleton.

Vertical drainage in peatlands is generally only necessary for the more amorphous types of peat and particularly when underlain by thick clay layers. Unless project timescales are exceptionally tight fibrous peats can usually be expected to dissipate any excess porewater pressures quickly enough without having the need to resort to additional vertical drainage acceleration measures.

Table 29. Summary of vertical drainage assistance.

Summary of vertical drainage assistance	
Advantages	Reduction of time for primary consolidation and secondary compression to happen.
Disadvantages	Acceleration of primary consolidation and secondary compression results in significant settlements during construction period. Performance of drains affected by buckling, heave, smear.
Risks	Loading of peatland. Bearing capacity. Effects on adjacent structures. Altering existing drainage paths.
Case Histories	None

8.6.5 Piling

Piling is not normally used for road construction over peat unless settlement control is particularly critical. The method has high mobilizing costs, setting up and driving costs and generally only comes into its own in bridge approaches and the like where settlement criteria are normally more onerous.

To date piling within peatlands has usually been carried out with precast concrete piles, 400 to 600 mm square with working loads of up to 150 to 250 tons respectively. These piles can be jointed for the deeper depths of soft soils (greater than 15m) and can be spliced by various means such as bayonet joints, wedge joints, etc. This joint must be as strong as the pile and have the same resistance to bending to ensure that no unnecessary weakness is created in the overall pile length.

CFA (continuous flight auger) piles are increasing in popularity in the Northern Periphery road districts and can be very competitive with good production rates. The piles are formed by boring a 'continuous flight auger' into the ground that supports the sides of the hole with the soil within the auger. When the auger reaches the required depth a sand-cement grout or concrete is pumped down through the hollow stem of the auger as it is withdrawn up the shaft. Reinforcement is placed immediately the auger has been withdrawn from the hole. CFA piles are available from 300mm diameter to 900mm diameter and can be driven to 30m deep.

Irrespective of the pile type chosen pile groups through peat are usually topped with 1 of 3 types of cap: either a continuous concrete slab or individual concrete pile caps or a geotextile/concrete cap combination.

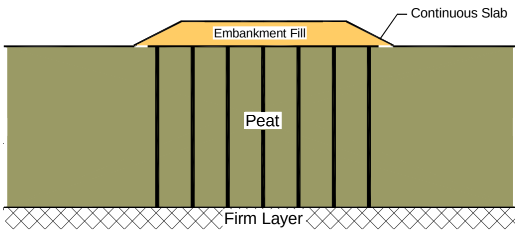


Figure 59. Continuous slab pile cap.
(P. Carlsten modified by G. Smith)

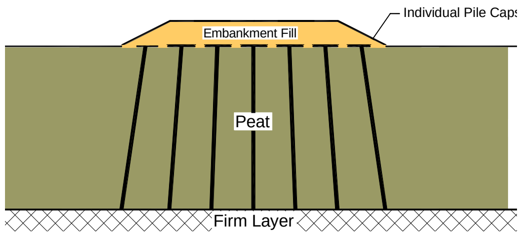


Figure 60. Individual pile caps.

Good practice normally requires a pile group to be self supporting, i.e. as if the peat was not there at all, ignoring any side resistance which may come from the peat. Raking piles are used to give added horizontal resistance where the completed pile installation is expected to be affected by horizontal forces. Finland uses 2 or 3 rows of raking piles in all piled embankment installations as it is considered that future loadings on the adjacent peatland could result in horizontal forces on the piles.

Geosynthetics can also be used as pile caps and new design philosophies are now available which matches size and centres of caps to suitable strength geosynthetics fabrics to produce a 'load transfer platform' rather than a rigid concrete slab.

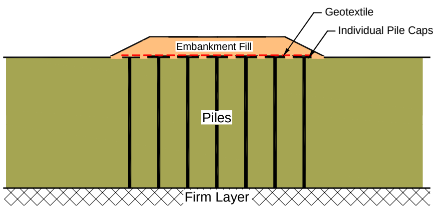


Figure 61. Geosynthetic/Cap Pile Combination.
(P. Carlsten modified by G. Smith)

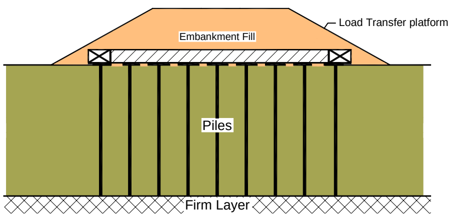


Figure 62. Load transfer platform.

In this process the 'load transfer platform' usually comprises one or more layers of geosynthetic reinforcement that is laid across the tops of the pile caps under the base of the proposed embankment. As the embankment is constructed in layers on the geosynthetic a form of soil arching occurs between the pile caps to transmit the embankment load into the piles and down to the firm layer.

Table 30. Summary of piling.

Summary of piling	
Advantages	Does not require peat excavation, disposal or the need for additional land for storage of spoil. Limited site disturbance. Minimal settlement. No additional time required for surcharge effects.
Disadvantages	Does not rely on strength of insitu peat. No support assumed from surrounding soil. Usually needs a continuous concrete slab or geotextile load transfer platform. Depth to load bearing stratum.
Risks	Piling operations. Vibration. Effects on adjacent peatland and structures. Design sophistication
Case Histories	None

8.6.6 Mass stabilisation

Mass stabilisation is a relatively new technique in road construction over peat and to date only the partner Districts of Finland and Sweden have trialled the method within the Northern Periphery. So far the method has been used as a means to improve the strength of the underlying soil in order to improve its bearing capacity and increase embankment stability but the method can also have the secondary benefit of reducing settlement time and horizontal displacement. These have not yet been fully explored.

The philosophy behind mass stabilisation is relatively simple. The weak peat is mixed together with a binding agent, usually cementious, by a mechanical mixing tool to produce a stronger and stiffer stabilised block. The essentials of the method can be shown in the line diagrams below.

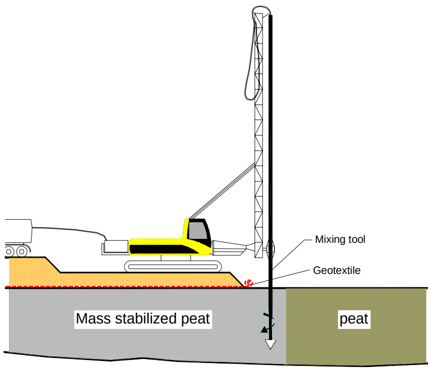


Figure 63. Illustration of the mass stabilisation process. (G. Smith).

During the process a dry binder is fed to the mixing head with compressed air and the mixing head rotated vertically and horizontally through the peat mass. Here the binder reacts chemically with the pore water in the peat and cures to a cementious mass.

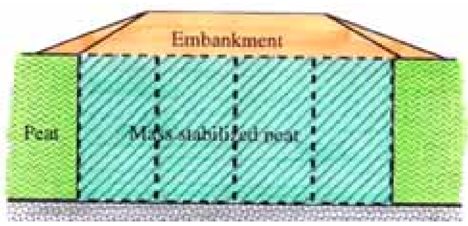


Figure 64. Cross section through stabilised peat,
Source: N Jelisc, Mass stabilization of peat in road and railway structures.

Projects involving stabilisation have traditionally involved the stabilisation of clay soils and have used slaked lime and cement lime mixtures. Proprietary binder mixes are now widely used with the most important components being limes, cements, blast furnace slag, fly ash and gypsum. As always the choice of the final binder is dependant on the characteristics of the soil to be improved and in the case of peat this means its geomorphology, its geotechnical and chemical properties.

So far mass stabilization projects in the Northern Periphery have been carried out using a mixing tool developed by YIT-Yhtymä Oy of Finland mounted on an excavator boom. A typical stabilized 'block' in road improvements normally comprises 8 to 10 square metres in plan and 3 to 5 metres in depth and is usually surcharged with 0.5m to 1m of fill material immediately after the completion of mixing to compress the stabilised material and increase its strength. This surcharged area in turn acts as the working platform for the machine for the next section.

The strength of the stabilised soil depends on the type and quantity of binder as well as the properties of the natural soil. A typical undrained shear strength for stabilised peat normally lies within the range of 50 – 150 kPa. Figure 8.42 shows some typical peat constituent components and volumes in the various stages of a stabilisation project. The figures shown were measured in Swedish case history Sw7, Road No 44 between Uddevalla and Trollhätta and are courtesy of P Carlsten.

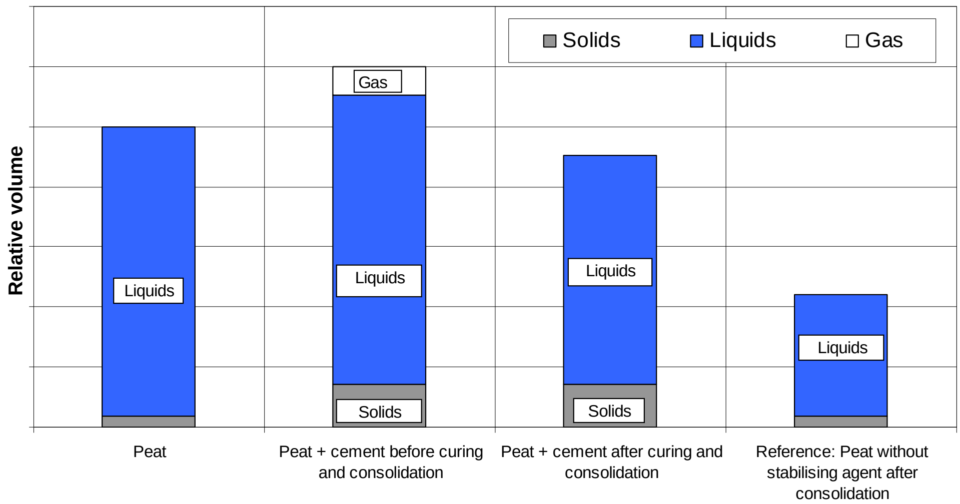


Figure 65. Cross section through stabilised peat,
Source: Carlsten, P & Olsson, M, 2004, "Masstabilisering av torv på riksväg 44"

Reading from the left, the 4 columns I the figure show:

Column 1: The constituents of the natural unloaded peat with a water content of 2000 % and a void ratio of 26.
Column 2: The constituents of the stabilised peat immediately after the stabilisation process has been carried out with 200kg/m³ of cement. Air and cement have been mixed into the peat in the field and the stabilised peat volume has increased by approx 20%. In a 5m stabilised deep peat deposit this would mean that the ground level would rise by 1m. The void ratio at this stage before curing and consolidation was 6.8.
Column 3: The stabilised constituents after 6 months curing and consolidation with a preloading fill of 3m. The void ratio at this time was 5.4.
Column 4: The typical constituents of a comparable reference preloading operation without stabilisation. A longer preloading time is needed for this operation to be effective and larger settlements can be expected. A typical void ratio after consolidation in this reference work would be 10.9.

Settlement calculations to date have been based on results from tests on specimens made in laboratory. This can lead to over estimations of settlements and shear strength in field. The process of preloading and unloading the stabilised peat ensures that the stabilisation has been tested for a load larger than the design load but the ultimate limit state should also be verified by calculations. The width of the stabilisation should be wider than the planned embankment.

Table 31. Summary of the mass stabilization method.

Summary of the mass stabilization method	
Advantages	Does not require peat excavation, disposal or the need for additional land for storage of spoil. Reduces settlements and adds to bearing capacity of the peat. Smaller demand of fill material compared to other preloading techniques. Could be suitable for high standard roads with high demands on differential settlements and bearing capacity. Could be suitable when there is soft clay beneath the peat.
Disadvantages	The time needed for preloading can extend construction time. Surcharge materials may need to be brought on to site earlier than required and require double handling as a consequence. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved.
Risks	Loading of peatland. Stabilisation operations. Bearing capacity. Effects on adjacent structures.
Case Histories	Sw5, Sw6, Sw7

9 Monitoring of Embankments over Peat

9.1 Introduction

The monitoring of an embankment over peat, or more correctly the peat below the embankment, can produce very useful information for the construction engineer on the stability of the embankment in the short and medium term. This information is particularly useful when employing a stage construction or surcharge exercise where each layer in the sequence has to rely on the strength of the underlying material.

Site monitoring is good practice and an essential aid to geotechnical risk management. Even if it does nothing else a good system can confirm that the geotechnical design is going to plan. Site observations during monitoring operations are central to this confirmation process:

- **In the short term:** as an important aid in checking the 'primary consolidation' rate of the embankment construction and the foundation stability ensuring that any excess pore water pressures generated by the new loads are given time to dissipate and that the underlying peat gains sufficient strength to support any additional layers before the layers are placed. A good system of monitoring here will be able to identify any departures from the design plan and allow appropriate actions to be taken timeously.

- **In the medium term:** as a useful tool for predicting the rate of post construction 'secondary compression' settlement over the design life of the road. If this rate is considered unacceptable, for example for the class of road or for adjacent structures, alternative methods of construction can be considered sufficiently early within the construction phase to reduce the rate to more acceptable levels. eg longer period of construction, the use of temporary surcharge layers, etc.

Once installed monitoring equipment of course produces data and this must be collected, plotted, reviewed and evaluated by competent personnel within the required timeframe if it is to be of any use in monitoring the stability and safety of the structure.

In the Northern Periphery monitoring of road projects over peatlands is usually controlled through a combination of instrumentation, soil mechanics theory and practical experience.

9.2 Monitoring instrumentation

A typical arrangement of monitoring instrumentation for monitoring an embankment over a peatland is as follows:

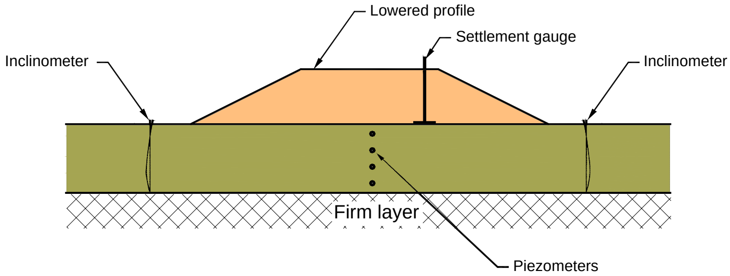


Figure 66. Typical instrumentation installation for an embankment on peatland (G. Smith)

This section will concentrate on the 3 main types of monitoring equipment normally used in the Northern Periphery roads districts; settlement, lateral movement inclinometers, porewater pressure piezometers.

9.2.1 Monitoring of settlement

The monitoring of embankment settlement is normally done using one or more of the following well established methods:

- Surface settlement plates
- Depth settlement plates
- Hydrostatic profile gauges

Surface settlement plates

The surface settlement plate is a simple visual measuring device and normally consists of a flat plate (usually 500mm x 500mm) on to which is welded a rod of sufficient length to ensure that the end extends above the surface once the settlement has taken place. The plate is positioned on the surface to be monitored, such as a construction layer, a geotextile layer or an original ground surface and as an added sophistication its rod can be sheathed in a duct to protect it during settlement of the overlying fill. These plates are then referenced back to fixed ground control points for consistency of monitoring.

Figure 67. Settlement plate

Depth settlement plates

A depth settlement plate is normally used where it is necessary to know the behaviour of a point in the soil below the settling embankment. To achieve this the settlement plate is fabricated as a short length of screw and screwed down into the soil to the depth required. Once in place the screw is effectively locked in place within the soil mass and moves as the mass settles giving an indication of the settlement at the initial installed point. As with surface settlement plates depth plates are referenced back to fixed ground control points for consistency of monitoring.

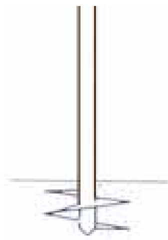


Figure 68. Depth plate.

Hydrostatic profile gauge

The hydrostatic profile gauge (or ‘hose settlement gauge’) developed by the Swedish Geotechnical Institute is a popular device for monitoring the cross-sectional profile of settlement under embankments over peat.

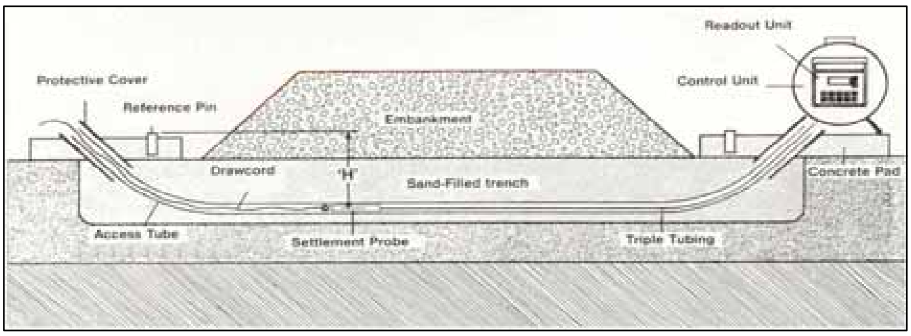


Figure 69. Installation detail for the ‘Borros hose Settlement Gauge’.

For a normal installation, a 50mm diameter plastic tube is placed on the peatland surface transverse to the road line prior to commencement of filling operations. As the layers of fill are placed on the peat, and the embankment settles, a pressure transducer is pulled through the tube to measure its deflected shape under the embankment. The measurements obtained are then reduced to the contract level datum and presented as a cross section through the embankment for use in measurement and earthworks control purposes.

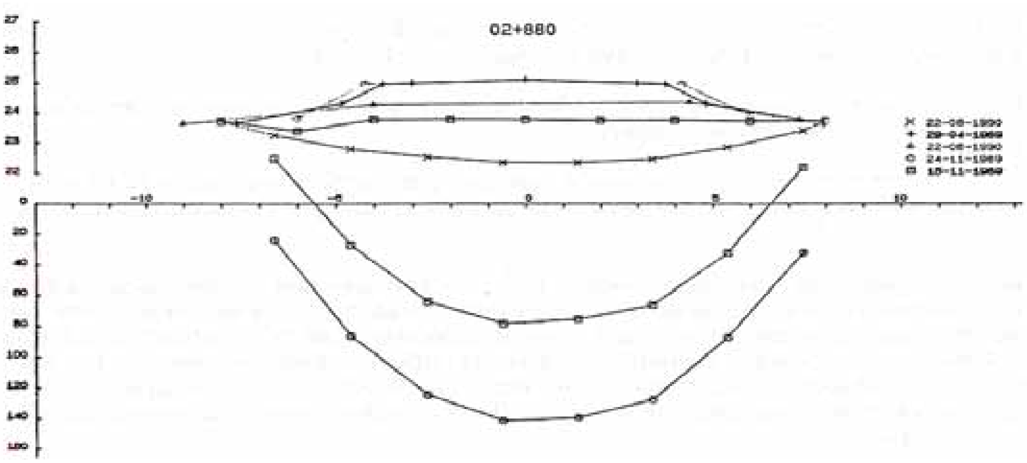


Figure 70. Typical cross-section from a Hydrostatic profile gauge.

Once the actual settlements for each cross section are known they can be compared with the geotechnical engineer's predictions of settlement. If necessary, these predictions can be recalculated to accord with the actual measured settlements by "back calculating" the theoretical peat parameters that would give rise to the measured settlements. These parameters can then be used to predict the future behaviour of the loaded peat.

9.2.2 Monitoring of Lateral displacement

Vertical settlement of embankments constructed over peat is almost invariably accompanied by some lateral movement within the peat mass away from the loaded area. This lateral displacement can result in increases in the vertical settlement of the embankment by up to 15% depending on the type and depth of the peat deposit so it is only sensible to try to measure this as it happens. Generally this is done by monitoring the lateral displacements on site and 'back calculating' the observed results to better estimate the aggregated effect.

There are a number of methods available that can help measure the amount of lateral displacement and a summary of the more popular methods used in the Northern Periphery partner areas are given as examples below.

Surface measurement of lateral displacement

Surface measurement of lateral displacement is normally done with monitoring pegs that are referenced back to a fixed datum before the commencement of operations and monitored regularly afterwards to the same datum. This can be at its simplest by means of a series of pegs driven into the peatland surface along a line of sight where the pegs will be seen to deviate during loading operations. The 'out of line' deviation can also be measured by physical survey to a digital base. The benefit of a rigorous engineering survey is that it can produce an accurate record of proceedings, that can be relied upon in the case of dispute, and also allow analyses of horizontal movements and any associated surface heave.

Measurement at depth of lateral displacement

The measurement of lateral displacement within the peat mass at depth is normally carried out using inclinometers and there are many types of devices on the market (see "Field Instrumentation in Geotechnical Engineering" by TH Hanna 1985 for a good range).

Inclinometers are normally installed at the toe of the embankment sideslopes but can also be usefully installed at other locations to monitor construction effects. Inclinometer readings can be particularly useful in the early identification of excessive lateral movements and give warning of developing instability.

Within the Northern Periphery the most popular instrument is the inclinometer developed by the Swedish Geotechnical Society. This method installs a 42mm diameter flexible plastic pipe down through the peat mass at the survey point and the inclination of the pipe is thereafter measured at regular depths (normally at 1 to 2m intervals depending on the overall depth of the peat deposit) by lowering down an inclination sensor. The pipe is provided with a telescopic tip to avoid the influence of the settling soil mass on the inclinometer as the embankment load is applied.

In this method the sensor measures the instantaneous inclination of the pipe at the point being measured and using this a deflected shape down through the peat mass can be obtained by graphically or electronically summing these successive instantaneous measurements.

9.2.3 Monitoring of Porewater Pressure

Porewater pressure is normally monitored using piezometers installed within the peat mass and a number are currently available on the geotechnical market, from the simple standpipe piezometer (Fig 71) through hydraulic (Fig 72), electrical and pneumatic varieties to the more complex vibrating wire unit. All can give satisfactory performance but not all give the same response time and this may be critical in giving warning of a soil failure.

(Diagrams taken from BS 5390:1999)

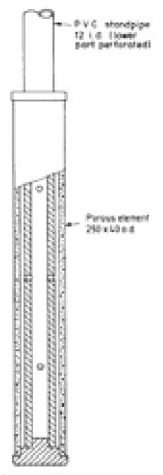


Fig 71. Standpipe piezometer

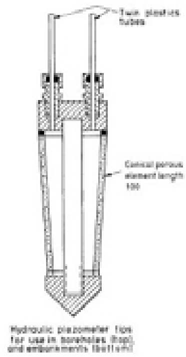


Fig 72. Hydraulic piezometer

10 Case Histories from the ROADEX Partner Areas

10.1 Case histories

This section contains a number of case histories from the Roadex Partner road districts that were brought to the attention of the writer during the researches for the Project.

These histories are not examples of specifically tailored projects to give illustrations of individual methods of construction but do give insights into past and current practices for dealing with bearing capacity problems of road construction over peat.

Road maintenance schemes were usually tackled empirically without the assistance of a detailed ground investigation. Techniques witnessed ranged from soil replacement through offloading and lightweight fills to pavement reinforcement with geotextiles. The choice of technique for a particular location was generally determined through an aggregation of the cost influencing factors appropriate to the site in question; the amount of soils investigation and testing necessary for each method, the complexity of the particular engineering works, the required time for execution of the method, the type of budgetary control in force, the amount of traffic disruption and additional traffic control required by the works, the expected future maintenance liability.

It was only after all of these construction and maintenance effects were examined that the most cost effective solutions emerged and final choice was made.

It was noticed that the period of construction envisaged by the design engineer was not always available to the successful Contractor when he got onto site. Various external influences such as changes to budgetary strategies used, political acceleration of programmes, difficulties with land entry, had restricted construction times and limited periods for consolidation with the result that planned methods of construction had had to be altered. But that is the engineering world we live in.

The case histories are gathered together by Partner area as below:

- **10.2 Finland**
- **10.3 Norway**
- **10.4 Scotland**
- **10.5 Sweden**

10.2 CASE HISTORIES: FINLAND

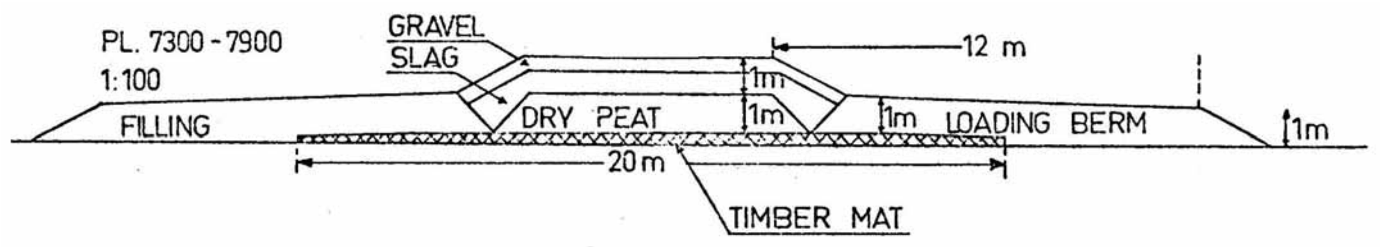
The following case histories are presented with the permission of the contributors:

F1	Road No 12 Mankala-Kausala embankment	1957
F2	Tattariharju access road, Helsinki-Lahti Motorway	1969
F3	Road No 760 Leppälahti to Köyhänperä	1976
F4	Helsinki-Lahti-Lusi motorway, Ahtiala to Härkälänkylä section	1977
F5	Road No 760, Reisjärvi to Köyhänperä	1985
F6	Road No 7621, Köyhänperä to Kalaja, Finland	1985
F7	Road No 280, Forssa-Somero Road	1987
F8	Road No 21 Kilpisjärvi	1988
F9	Mankkaanvayla Test Roads, Espoo	1990
F10	Road No 83 Sinettä – Pello Road	1995
F11	Road No Y607, Leteensuo peat bog, Hattulaa, Häme	1996
F12	Tokero to Vehkaisilta pedestrian/cycleway	1998
F13	Road No 930 Mellajärvi steel reinforcement	1999

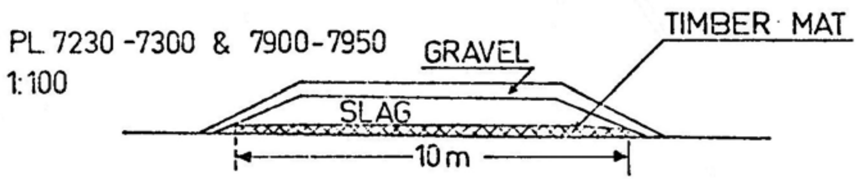
Figure 73. Widening of an existing road embankment with a timber grillage, Finland.

Case Study F1	Road No 12 Mankala-Kausala embankment, Finland				Date	1957	
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0m

This gravel road across the Vettoisten peat bog on the Mankala-Kausala road was constructed on 4m of peat over a 3m thick layer of silt and 10m of soft clay. From 73+00 to 79+00 the design employed a double wooden grillage of poles with a minimum top diameter of 15cm placed at 60° to the road line and each other. The grillage below the main road embankment was covered with 1m of air dried peat lightweight fill material to keep the imposed loads of the embankment to a minimum and the load bearing road pavement layers constructed on top of his in the normal fashion. This form of dried peat was regularly used for animal bedding on farms at that time and was readily available as a local lightweight material. The new embankment construction was additionally stabilised with 1m thick symmetrical 12m wide loading berms on both sides.

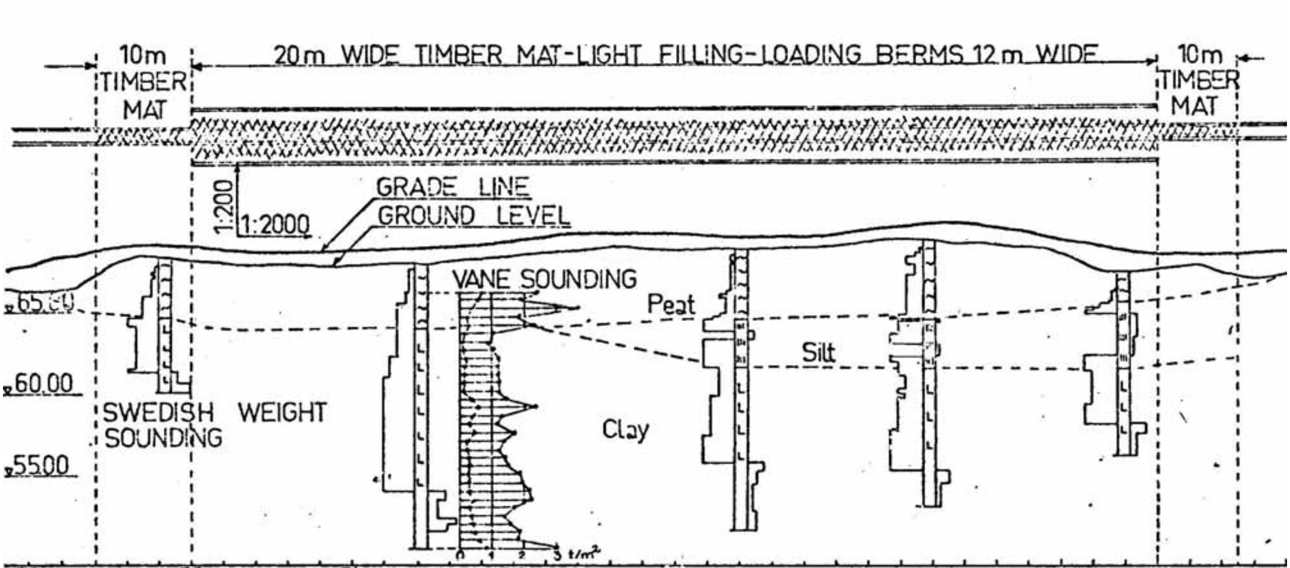


Arrangement of grillage and side berms on main embankment



Grillage reinforced embankment on edge of bog

The long section below shows the extent of the grillage employed and gives details of the contract ground investigations using 'Swedish Weight Sounding' and 'shear vane' testing carried out in preparation for the works..



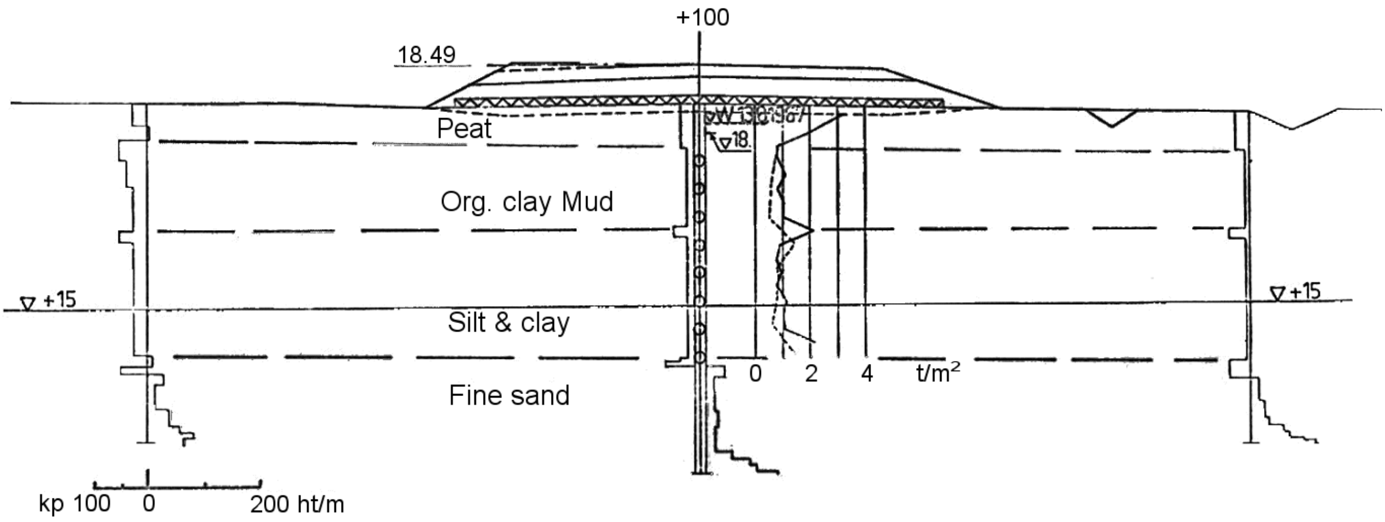
No unusual settlements or embankment behaviour were detected in the 10 years following construction of the roadeven with the use of the dried peat lightweight fill and in 1966 the completed gravel road was surfaced with a new 10cm thick layer of crushed gravel and paved with asphalt.

In 1968 the mature insitu road was investigated to determine if the embankment was stable enough to be incorporated into a new motorway being planned. The investigation found that the embankment had settled relatively evenly across its length (20cm at the centre of the bog and 10cm at its edges) and this was subsequently considered to be acceptable for incorporation as one direction of the new Mankala-Kausala motorway.

Source: M Kolhinen & R Orama, "Experiences in construction of some highways across peat bogs in southern Finland", Finish Administration of Roads and Waterways, Helsinki

Case Study F2	Tattariharju access road, Helsinki-Lahti Motorway					Date	1969
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0m

The access road leading off the Helsinki to Lahti motorway to Tattariharju was constructed on a subsoil comprising 2m of peat over 4m of organic clay on top of 8m of soft clay. The geotechnical design for the new embankment predicted significant settlement of these layers and as a result a number of design options were considered. The eventual choice for construction fell on an embankment founded on a timber mat in preference to a peat displacement exercise and a timber grillage of poles with a minimum top diameter 10cm was eventually used on site. Embankment settlement was not considered to be major design requirement as the project lay within a speed restriction area on the approach to the town and as such high surface tolerances were not deemed necessary. In the event settlement was generally even and within acceptable tolerances.



Cross-section through access road

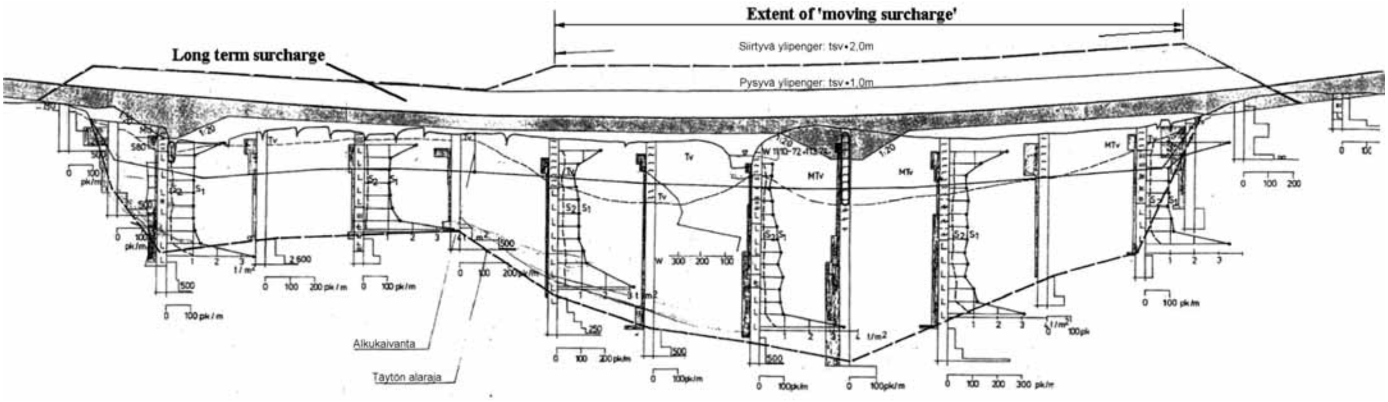
A lesson learned during construction: An old ditch running below the new embankment triggered a failure in the subsoil during the embankment construction sequence. This localised collapse was rectified by strengthening the timber grillage in the vicinity of the slip and reducing the weight of the embankment by using a lightweight fill. An internal post construction report on the project concluded that in general "*ditches should be at a greater distance from the embankment*" and "*that old ditches should be backfilled with peat or other light filling before loading*"

Source: M Kolhinen & R Orama, "Experiences in construction of some highways across peat bogs in southern Finland", Finish Administration of Roads and Waterways, Helsinki

Case Study F3	Road No 760 Leppälahti to Köyhänperä, Finland					Date	1976
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0/7.0m

This project on the Leppälahti to Köyhänperä road involved an embankment drive using partial excavation across a 520m long soft area comprising up to 3m of very humified peat (shear strength of 10-20 kPa) over 2 to 3m of very organic clay (“gyttia”, shear strength 3-10 kPa) over 2-4m of clay. It was decided to use the displacement method to avoid secondary settlements and possible failures and try to found the embankment on sound material 4-9m below.

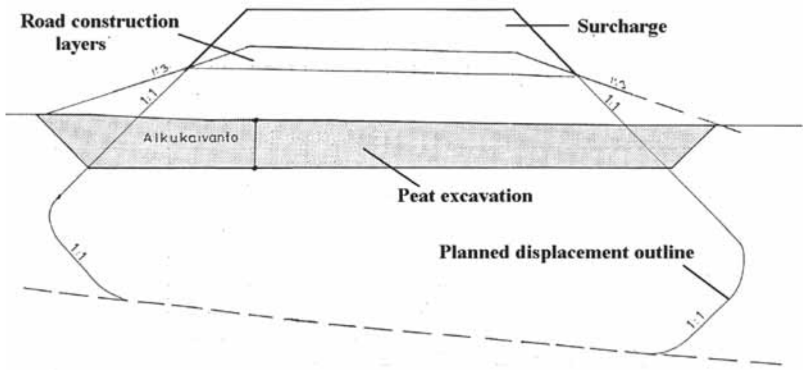
The method used is a combination of partial excavation with soil displacement (Section 8.5.2). The embankment was driven across the bog by its own weight aided by a surcharge with an additional ‘moving surcharge’ to force the displacement at the leading edge of the embankment.



Profile of displacement from 60+00 to 66+00

The finished embankment varied from 0.8m to 1.9m above bog level. This permanent material was enhanced by a 1.0m high ‘long term’ surcharge over the full length of the bog crossing and a further 1.0m high working platform or ‘moving surcharge’ was used from 62+20 to 65+80 effectively giving a 2.0m short term surcharge at the front of the displacement. 1.5m of peat was excavated out ahead of the displacement operations to remove the surface mat of vegetation and ease the displacement process. In the profile above the results of the ground investigations (penetration and vane testing) are shown as in Section 5.2.1. Peat is denoted by the symbol ‘Tv’.

.



Cross-section of displacement showing typical embankment outline

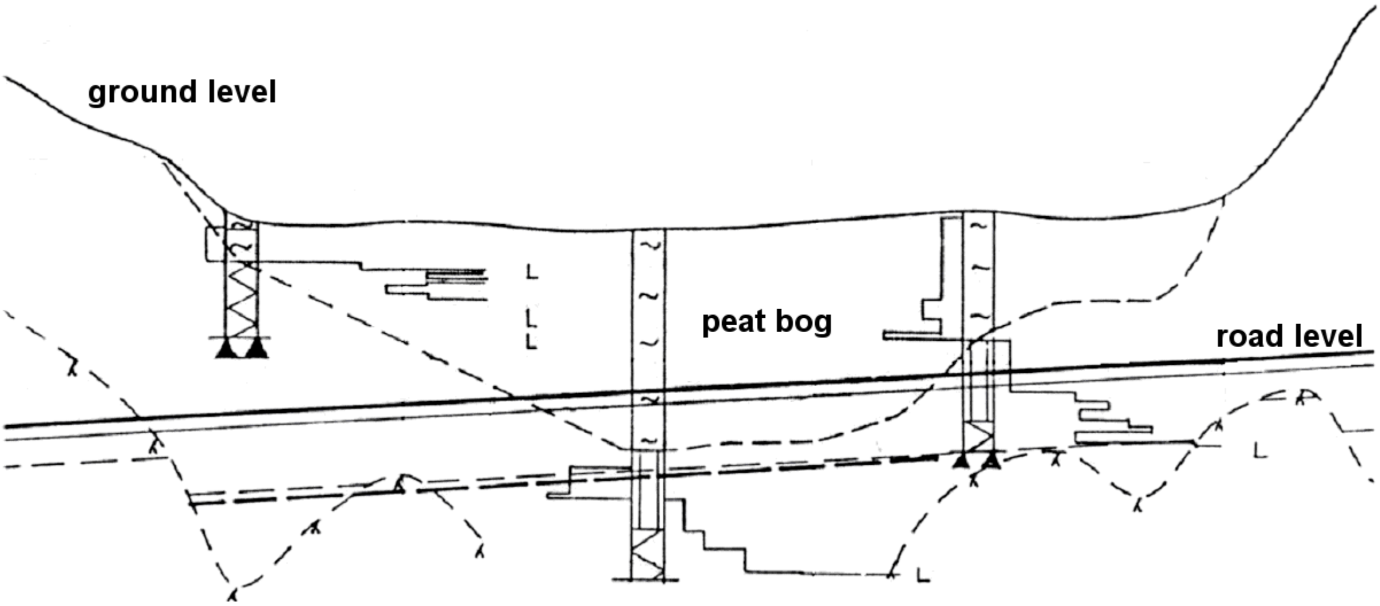
All material displaced above the bog level during the displacement operations was removed as soon as it appeared so reducing any resistance to the displacement of the embankment. An existing culvert at 63+60 was considered to be a possible hard area to the displacement and was excavated out to 4m deep and the surrounding ground disturbed to ensure displacement proceeded as planned. Settlement of the embankment was monitored by settlement plates until such time as the displacement was considered to have ceased at which time the 1.0m long term surcharge was removed. Once the embankment was considered sound the permanent culverts were installed.

This method is a common method of construction in Finland and produces acceptable results where the displacement can be effectively carried out.

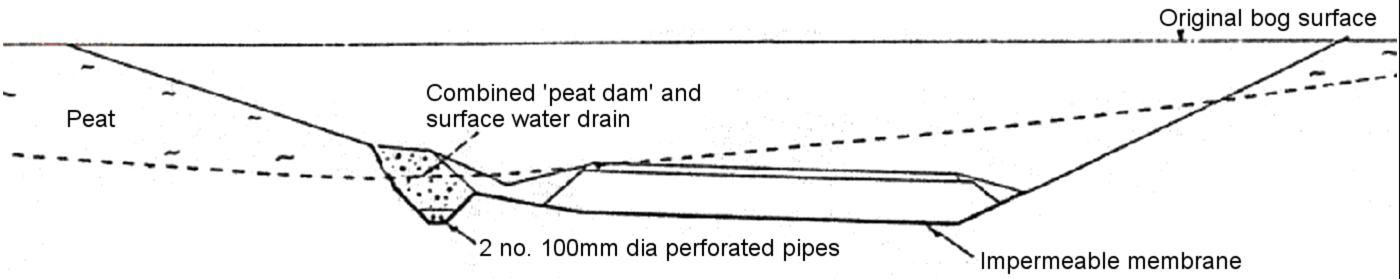
Source: R Louet, Keski-Pohjamnaan Piiri internal report, 14 September 1976

Case Study F4	Ahtiala to Härkälänkylä section, Helsinki-Lahti-Lusi motorway					Date	1977
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0/7.0m

An interesting peat engineering feature was trialled on the Ahtiala to Härkälänkylä section of the Helsinki-Lahti-Lusi motorway. A gravel 'dam', as detailed below, was constructed along a 200m long cutting through a small peat bog of maximum depth 6m. This 'dam' provided a retaining toe for the insitu peat slope and reduced the peat excavation quantities. The 'dam' structure also acted as a sideslope drain with 2 no 100mm diameter perforated drainage pipes to lower the groundwater table within the slope. The dam was wrapped with a coarse grade geotextile (Finnish geotextile class IV) during installation to permit the overall structure to act as a filter drain.



Long section through peat bog



Cross section through 'peat dam'

Without this feature it was considered that the excavated bog slope could drain directly into the roadside intercepting ditch with the resulting risk of icing in winter that, in severe conditions, could spill over onto the carriageway and create an icing hazard.

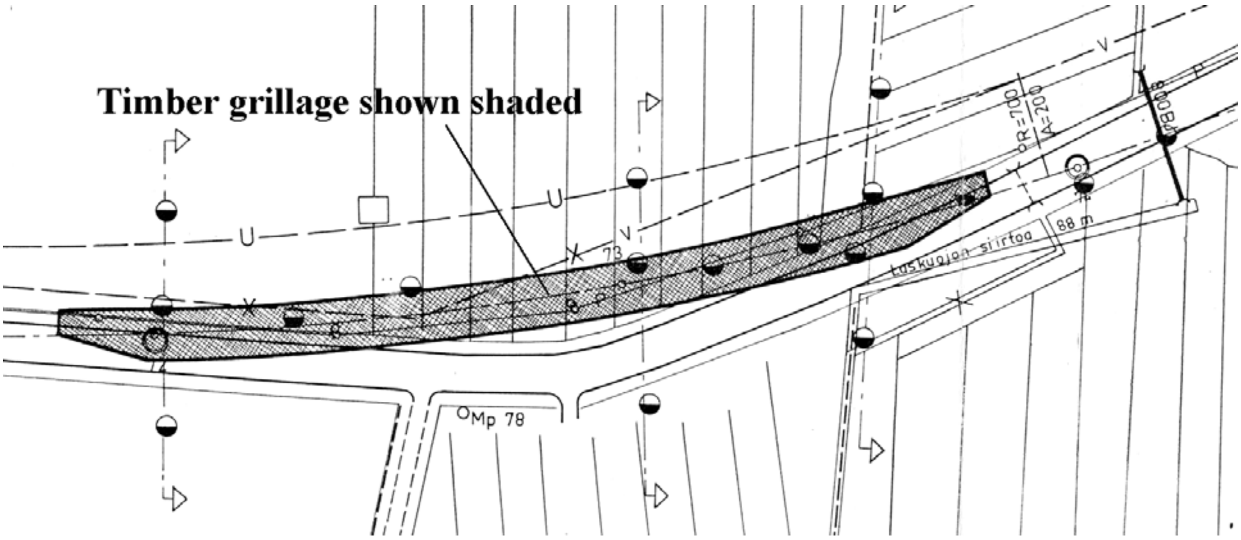
Source: Contract drawings for Helsinki – Lahti – Lusi Motorway, Ahtiala to Härkälänkylä section

Case Study F5	Road No 760, Reisjärvi to Köyhänperä, Finland					Date	1985
AADT	n/a	**Heavy vehicles**	n/a	**Speed limit**	n/a	**Carriageway width**	**6.0/7.0m**

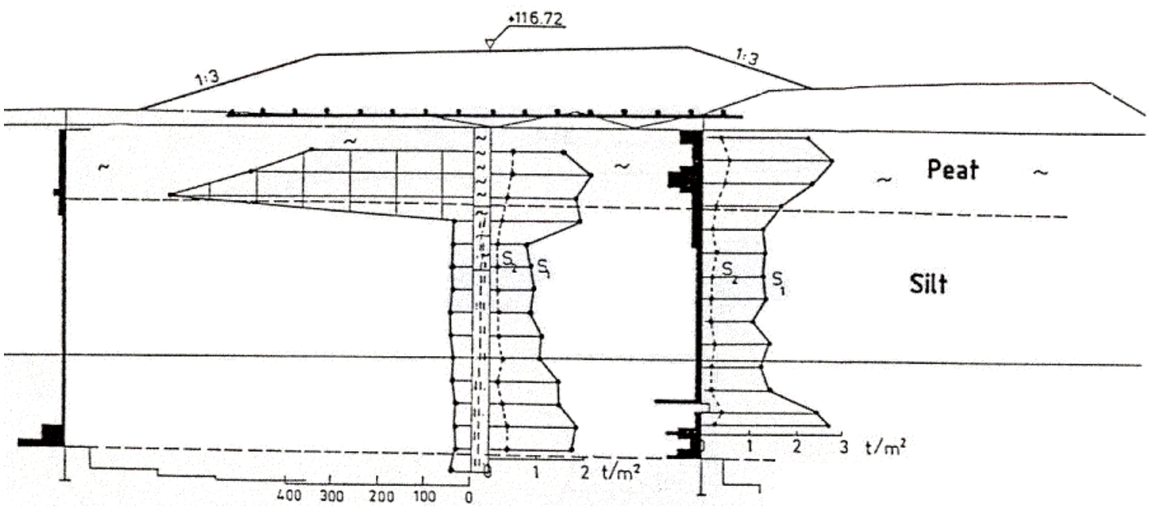
This section of the upgrading of the 760 Reisjärvi to Köyhänperä road involved the crossing of a 430m area of 4-8m deep soft ground between 70+20 and 74+50. The ground comprised 2-4m of peat with an insitu shear strength of 20kPa and moisture content of 500% on top of 2-4m of soft silt with a shear strength of 5-10kPa.

The new road alignment ran parallel to the existing road and used the existing embankment where possible. Construction was generally by standard 'excavation and replacement' techniques but from 71+80 to 73+80 a full width timber grillage was used to carry the new 1.2m high embankment across the bog. This grillage was constructed in 2 layers of 11m long poles laid at 0.6m centres and 45° to the roadline. The completed structure covered 2200m² overall

Before construction of the grillage all existing ditches below the new embankment were filled with peat from outwith the roadline and the material compacted using the bucket of the excavator. The existing surface of the bog was left untouched without clearance and the site levelled to prepare a working platform for the grillage to be laid out. The poles laid at the designed angle and spiked together at every other node with a steel reinforcement bar. On completion the grillage was backfilled with sod peat to protect the timber from decay.



Layout Plan (showing field ditches)



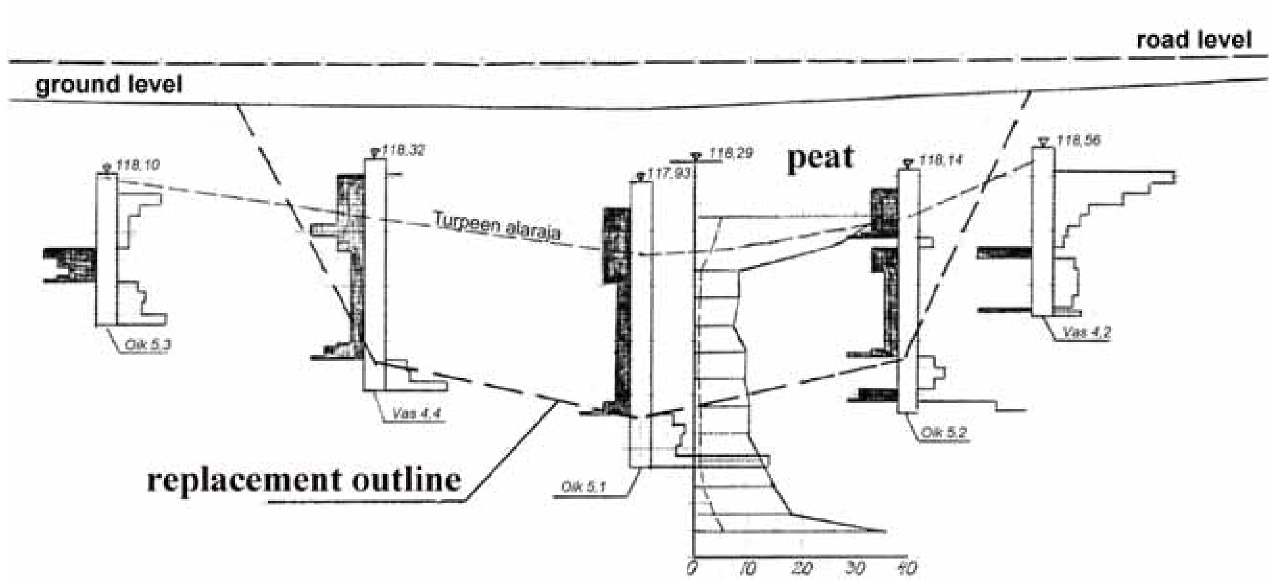
Cross-section through old and new embankments showing timber grillage

Source: R Louet, Keski-Pohjanmaan Piiri internal report, 16 January 1978

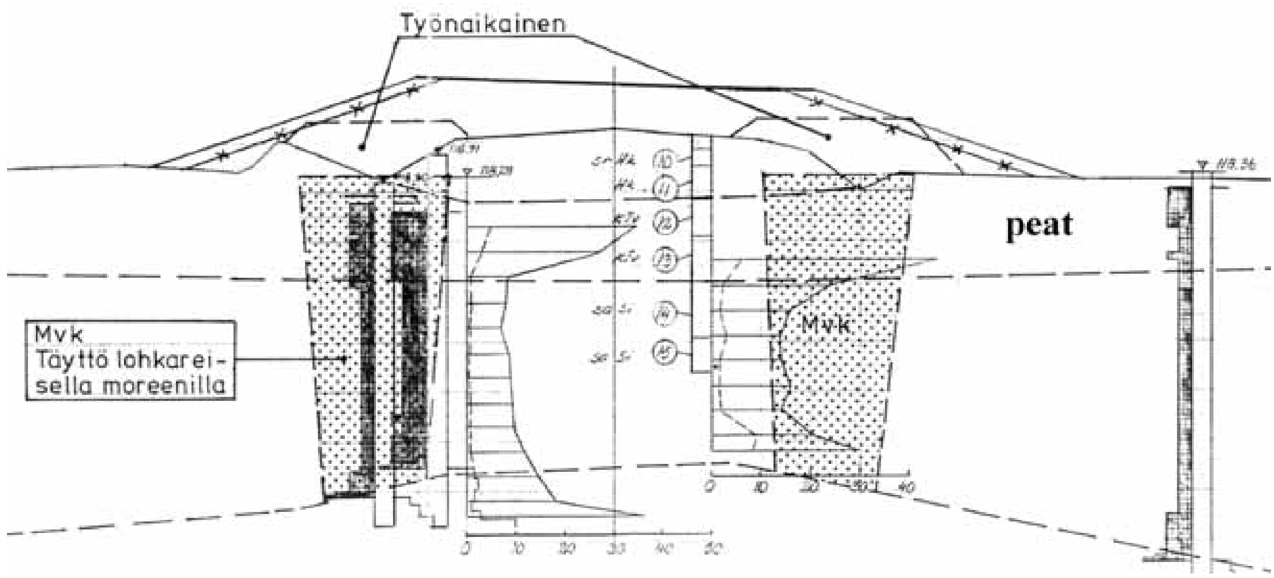
Case Study F6	Road No 7621, Köyhänperä to Kalaja, Finland					Date	1985
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0/7.0m

This project involved the widening of an existing carriageway over peat and silt by excavation and replacement using a method commonly called the "Legs" Solution

In the exercise the existing 6m wide carriageway on Road 7621 was left in place after having first confirmed that the soils beneath the existing embankment had gained sufficient strength for the new loading. Soil replacement trenches ("legs") were constructed on either side of the existing embankment to provide the necessary base width and support for the new widened road embankment.



Long section along road line



Cross-section through widened embankment

Interestingly a layer of compressible material was designed to be retained at the base of the legs to allow the new works to settle at the same rate, or greater, than the old construction.

This method is only considered appropriate where the new road is symmetrically located above the existing road, ie a balanced structure in respect of the new loading, drainage, subsoil conditions, etc. Where the new road arrangement does not conform to this requirement the normal practice in Finland is full excavation.

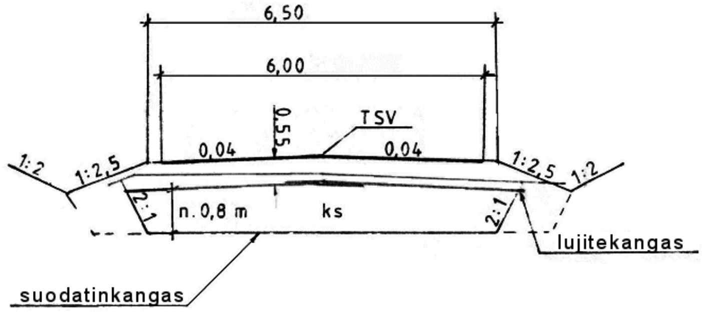
Source: Keski-Pohjanmaan Piiri contract drawings

Case Study F7	Road No 280, Forssa-Somero Road, Finland					Date	1987
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0m

Road No 280, the Forssa to Somero Road, passes through the western section of Finland's Torronsuo National Park. This 15km long peatland park is a raised "string bog" of pools and ridges. The road had a history of regular floods during times of high water levels that resulted in numerous roads closures and in 1987 it was decided to replace the road in-situ with a higher and lighter embankment to lift the road out of the areas subject to flooding. The project geotechnical survey found that the existing road lay on approximately 6m-8m of peat above a deep clay deposit. The strength of the in-situ peat varied from 4 to 8kN/m² (4 to 8kPa). The existing shallow gravel embankment was founded on an old timber grillage.

The road replacement design solution involved removing the heavy gravel construction of the existing road and unloading the underlying peat by means of a LECA lightweight fill enclosed in a geotextile. Where additional bending stiffness was required a timber grillage was installed.

Above this a new carriageway was constructed with a 400mm thick crushed gravel roadbase incorporating a structural steel mesh, 150mm of crushed gravel subbase and 100mm of asphalt. The grillage comprised 100mm diameter logs at 0.5m centres laid at 45° to 60° to the road centreline. The grillage logs were required to be in contact with each other at the node points and where this was not possible due to the irregular shape of the logs wedges were employed to ensure a fit. Once laid the grillage was fixed by spiking at every node.



Typical Cross-section of lightweight embankment

Excavation of frozen construction

New embankment construction

Geotextile and lightweight filling

Geotextile, LECA and grillage

Geotextile, LECA and grillage

Completed road in 2003

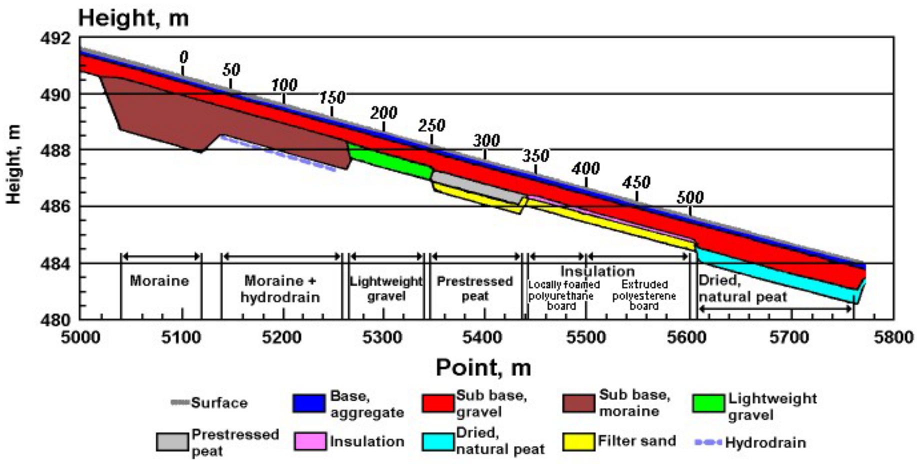
An interesting feature of the project was that the works were deliberately constructed in winter whilst the peat was frozen. The local hydrology, flooding problems and the presence of the National Park prevented the road being constructed in summer. The only other real alternative was to construct a new road on a new alignment around the bog outwith the Park but this was considered an undertaking too far.

The reconstructed road has been performing well since re-opening in 1987. Some minor uneven settlement was observed during a post construction monitoring survey in 2002 but this has not affected the traffic ability of the carriageway.

Source: A Valkeisenmäki, Finnra, Helsinki 2003

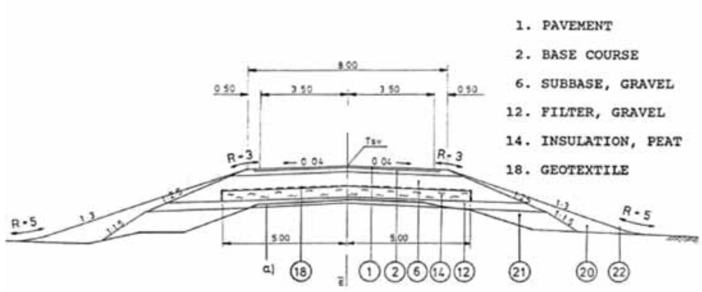
Case Study F8	Road No 21 Kilpisjärvi, Finland					Date	1988
AADT	405	**Heavy vehicles**	**16%**	**Speed limit**	**80 km/h**	**Carriageway width**	**7.0/8.0m**

Road 21 at Tulli, Kilpisjärvi was a research trial location of the 'Artic Road Project' that had the aim of developing design methods and road structures to deal with extreme frost heave, icing and snow drifting. The project did not deal with 'roads on peat' in the normal geotechnical sense but 2 of the 7 test sections at Tulli trialled peat as an insulation material within the embankment that are worth recording here.



Long section of trials of insulation materials at Tulli

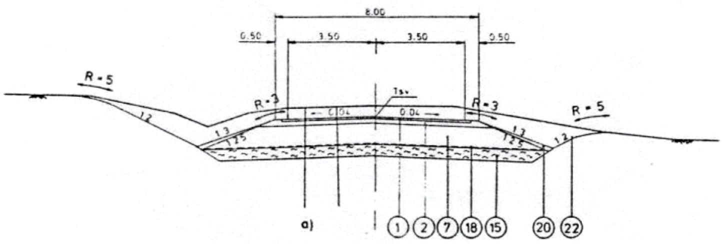
Test section 5340-5440 (shown coloured grey in the above profile) involved the use of prebagged compressed horticultural peat bales in a layer below the main carriageway. These bales were laid on edge on 300mm of filter sand to produce a 450mm thick insulating layer of compressed peat. In section 250m to 300m the bags were installed as sealed units and from 300m to 350m the bags compressed bales were installed unwrapped (see photograph below). This lightweight layer was then capped with 650mm of gravel sub-base, 150mm of base course and 40mm of wearing course.



Insulation layer of compessed peat bales

Photograph of installation of peat bales

Test section 5610 - 5760 (shown coloured blue in the profile) involved the use of locally excavated dried peat blocks in the base of the excavation below the new embankment. These blocks were laid in a 500mm layer and covered with a Class II geotextile. A standard embankment construction of 800mm of sub base gravel was then placed on the geotextile and the completed embankment topped with 150mm base course and 40mm wearing course.



Insulation layer of local as-dug sod peat fill

The results of The Artic Road project record that the bearing capacity of the insulated embankments is generally lower than a similar standard embankment constructed with mineral soils and some potholing has occurred in the pavement. These road sections continue to be monitored under the Roadex project. Fuller details of the performance of these sections can be found in the 2001 Roadex CD ROM

Source: S Saarelainen, Artic Road Construction Project at Kilpisjärvi 1993 and Saarenketo, Roadex Project 2001

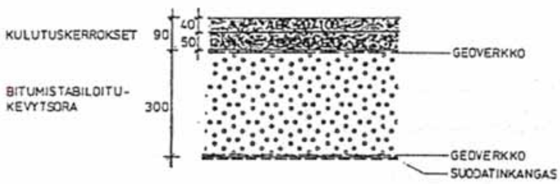
Case Study F9	Mankkaanvayla Test Roads, Espoo, Finland					Date	1990
AADT	n/a	**Heavy vehicles**	n/a	**Speed limit**	n/a	**Carriageway width**	**6.0m**

Espoo is a fast growing new town to the west of Helsinki. In 1988 Viatek Oy were commissioned to design a new distributor road network for a new phase of the city development that was to be constructed over soft ground.

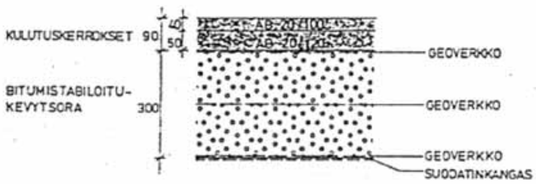
The company's soils investigation revealed that the planned road would cross soils with in-situ shear strengths of as low as 2kPa with natural water contents of 100% - 150% in deposits varying between 4m and 12m deep along its alignment. Soil replacement techniques were considered to be too expensive and unacceptable due to the proximity of adjacent developments and the design team concentrated their energies on producing an acceptable "floating road" with a sufficiently stiff pavement to be capable of withstanding the traffic flows envisaged.

Their final report recommended that 3 new types of lightweight bituminous pavements should be constructed directly on top of the regulated bog surface to fulfil the immediate needs of the cities expansion. These roads were to be continuously monitored for settlement, rutting, general condition, etc to identify the most suitable option(s) for the future phases of development.

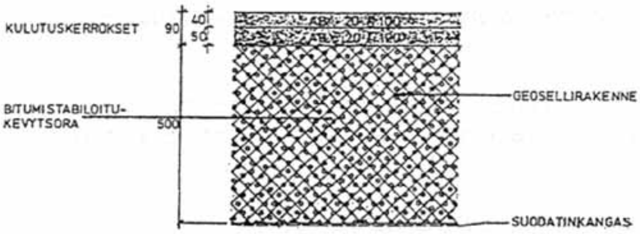
The three types of pavements installed under the contract were:



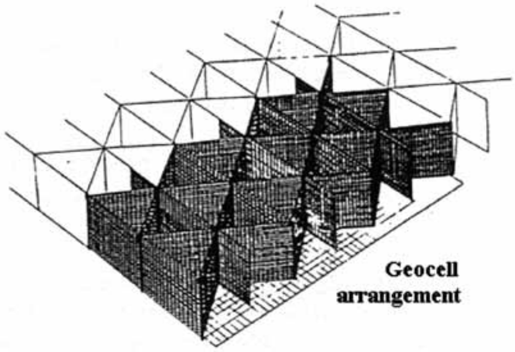
Bitumen stabilised LECA fill



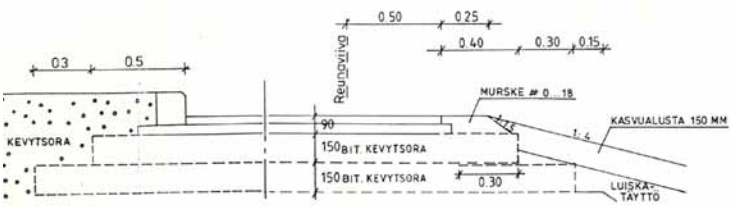
Bitumen stabilised LECA fill with geogrid



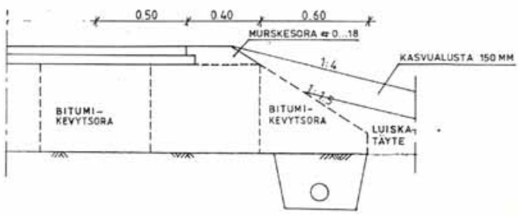
Geocell filled with Bitumen stabilised LECA



Geocell arrangement



Cross-section through bitumen stabilised LECA road construction



Cross-section through Geocell construction

At the date of writing all 3 test pavements have been in place for almost 15 years with no major problems reported apart from an expected 'hard' area in the carriageway where an existing piled pipeline crosses the network approximately 0.5m below the former ground level.

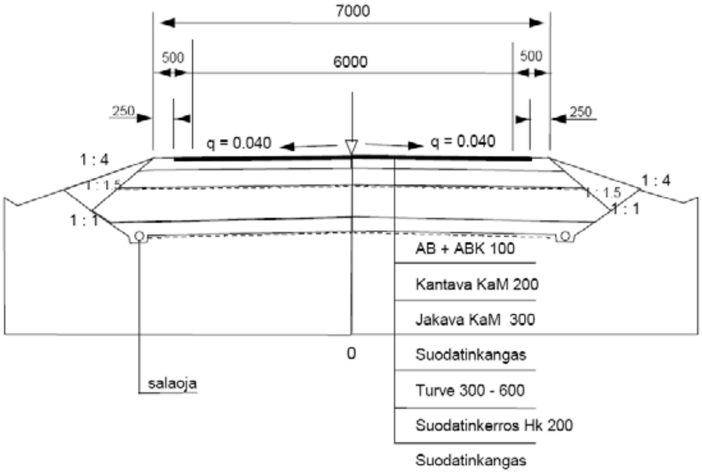
In recent years however the use of lightweight materials close to finished road level in Finland has been found to give rise to carriageway icing conditions on occasions and the practice is no longer popular. The reason lies in the secondary insulation properties of the LECA fill material. On very cold nights the lightweight granules below the pavement can insulate the underside of the carriageway and prevent the heat from the warmer embankment below from migrating up. This can cause the carriageway above the lightweight fill areas to freeze faster than the adjacent roads and cause localised icing.

As a consequence of this perceived problem Finnra now recommends that pavements built over lightweight materials should have a minimum construction thickness of 65cm above the fill material to act as a ‘heat sump’ against localised icing conditions.

Source: Viatek Oy, Espoo, Finland

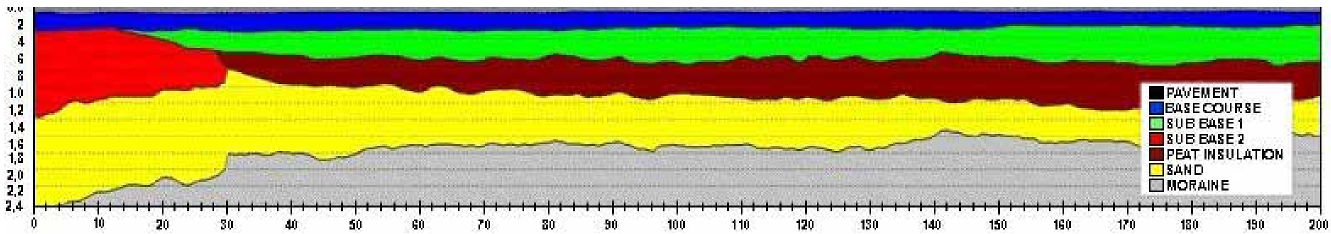
Case Study F10	Road No 83 Sinettä – Pello Road, Finland					Date	1995
AADT	1130	Heavy vehicles	4%	Speed limit	100km/h	Carriageway width	6.0/7.0m

Road No 83 between Sinettä and Pello was constructed in 1955. In 1974 the road was improved and in 1995 a section exhibiting differential frost heave was selected for reconstruction under the Finnish Roads Administration's TPPT (Road Foundation and Pavement Structure) programme. Under the TPPT programme the old road structure was completely excavated out and a new structure comprising a geotextile, 200mm of sand, 300-600mm of compacted sod peat, a geotextile, 300mm of unbound sub base 0-80mm, 200mm of unbound basecourse 0-50mm was constructed. This was completed with a 100mm bituminous pavement of 55mm of bitumen bound basecourse and 45mm of asphalt concrete wearing course.

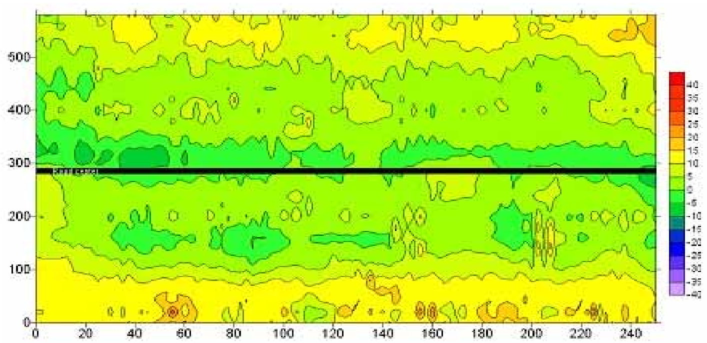


The use of sod peat blocks as a road construction material is long established in Finland but has fallen out of use in recent years due to changing road construction practices. This project allows the use of sod peat to be monitored and evaluated with modern techniques to establish if it has a place in modern road engineering.

The reconstructed TPPT road was inspected in 2000 as part of the Roadex project and a ground penetrating radar survey was carried out. A radar plot of this survey is shown below for interest. Fuller details of the GPR survey and evaluation are contained in the 2001 Roadex CD ROM



Ground penetrating radar plot and interpretation of TPPT sod peat insulation structure



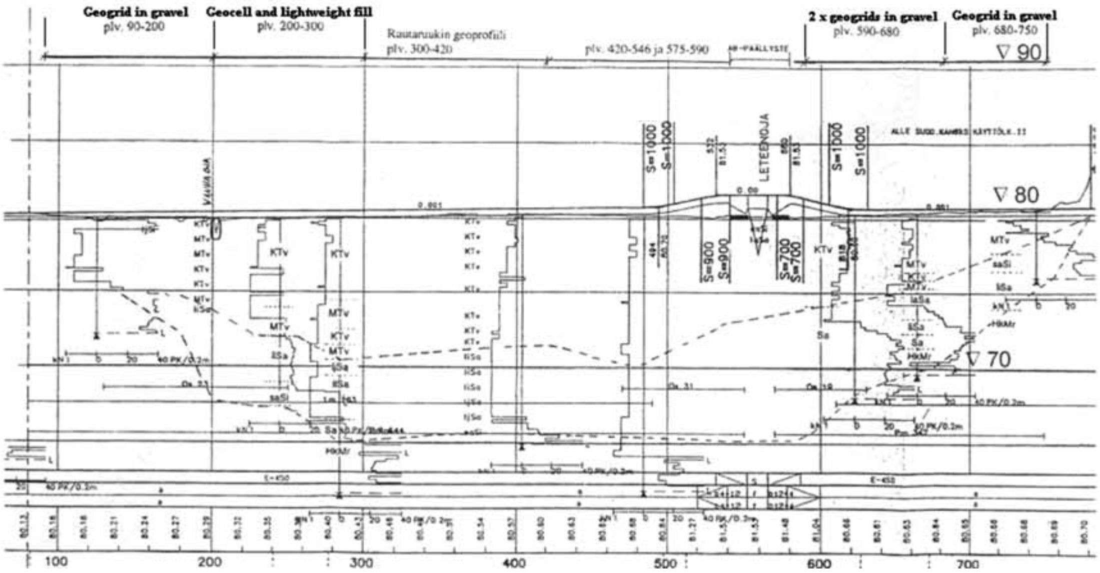
Map of road surface of TPPT sod peat insulation structure

Photograph of TPPT road section in 2000

The Roadex report concluded that the sod peat insulation structure appeared to be working well in 2000. The measured mean rut value for the right lane was 4.5mm, 5 years after installation, and 4.7mm in the left. This equates to a rut increase of 0.57mm/10 000 heavy vehicles, an indication of a strong structure. The road was stated to be 'very smooth' (see map of road surface above) with mean IRI values of 1.4mm/m in the right lane and 1.6mm/m in the left. Frost heave was not considered to be a problem. Some limited longitudinal cracking was apparent (photo) suggesting that the road had a slight differential across the carriageway. The GPR survey showed that the pavement and unbound layers were slightly thicker in the left lane possibly indicating that the peat in the left lane had compacted slightly more.
Source: TPPT Report 40 and T. Saarenketo, Roadex Project 2001

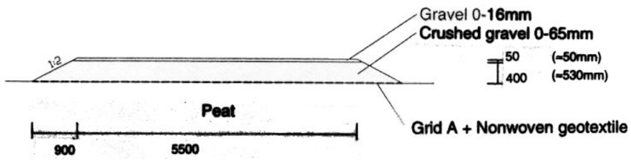
Case Study F11	Road No Y607, Leteensuo peat bog, Hattulaa, Häme, Finland					Date	1996
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	5.5m

This road is a private road that crosses over 10m of peat and is a part of the Finnish Georeinforcement Research and Development Project. The scheme was specifically used as a test bed for 3 different types of geogrid embankment reinforcement methods with the aim of improving the bearing capacity of the peat and reducing the potential for uneven settlement along the road. Two sections of geogrid trials and one section of geocell trial were constructed as shown on the test beds long section below.



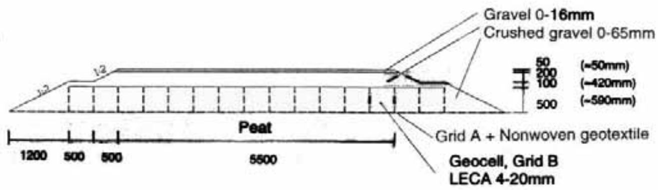
Trial section A: Single geogrid

A nonwoven separator grade geotextile was laid on the cleared bog surface. A Tensar SR30 grade polypropylene geogrid was laid directly on top of this geotextile followed by the crushed gravel bearing layer. Settlement of this trial section was as expected and after 18 months the embankment had almost fully settled into the peat. The settlement was uniform but road drainage had become a problem. It was considered that a lightweight fill (LECA) would have produced better results.



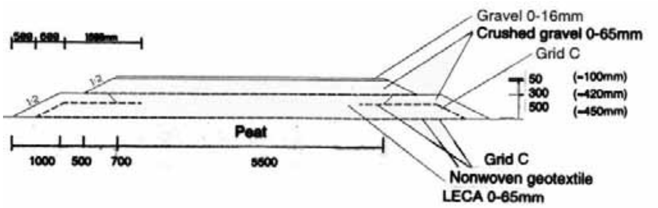
Trial section B: Geocell with lightweight filling

A nonwoven separator grade geotextile was laid on the cleared bog surface. A Tensar SR30 grade polyethylene (HDPE) geogrid was laid directly on top of this geotextile and used as a foundation for the construction of a 500mm high Tensar SR55 geocell array. This geocell was backfilled with 4-20mm LECA light expanded clay aggregate. Settlement of this trial section performed as expected and was relatively uniform along the section. The embankment settled almost to the depth of the lightweight geocell but the road construction layers remained dry.



Trial section C: Double layer of geogrid

A nonwoven separator grade geotextile was laid on the cleared bog surface. A Fortrac 35/20-20 grade PVC coated polyester geogrid was laid directly on top of this geotextile followed by a 500mm layer of 0-65mm LECA light expanded clay aggregate. This layer was topped by a further layer of Fortrac 35/20-20 geogrid and finished with a road construction of 350mm of crushed gravel. Settlement of this trial section performed as expected and was relatively uniform along the section. The embankment settled almost to the depth of the lightweight material but the road construction layers remained dry.

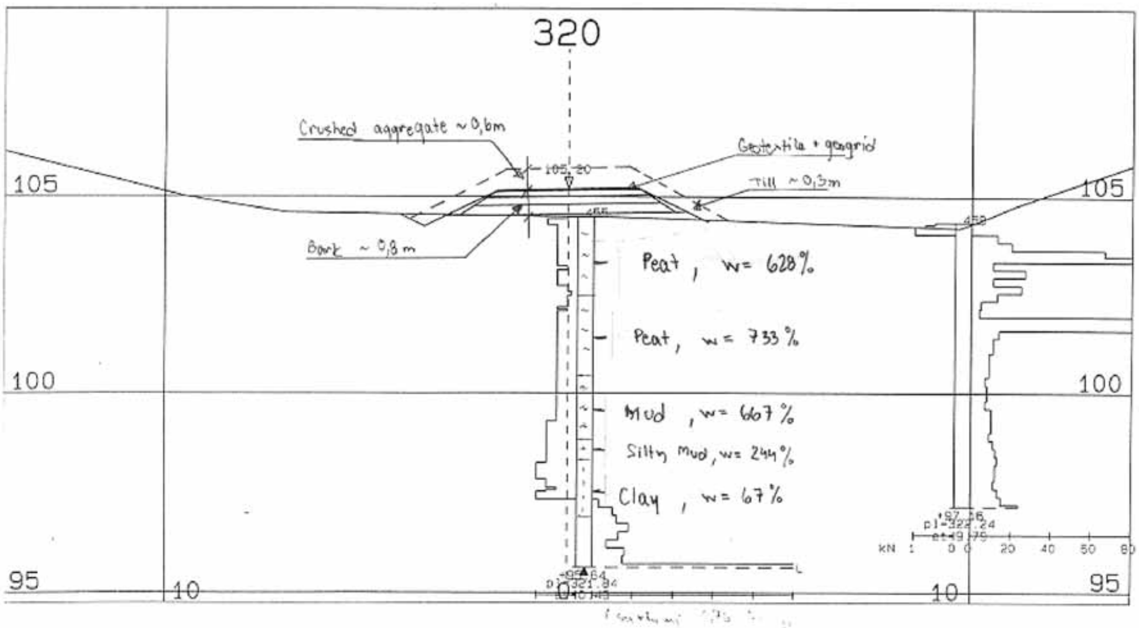


From the above trials it can be concluded that reinforced sections can produce more regular settlement across the cross-section when installed correctly. The geocell option was expected to be stiffer than the 2 geogrid system but this was not necessarily proved. The measured stresses in the geogrids were 2.0 to 5.0 kPa for the embankment and +0.4 to 2.5 kPa for vehicle loading.

Source: J Forsman, Geovahvistetutkimus, test structures 1996-2001, Finnish Road Administration, Helsinki 2001

Case Study F12	Tokero to Vehkasilta pedestrian/cycleway, Finland	Date	1998

This scheme involved the construction of a new 3m wide pedestrian/cycleway alongside Tokero to Vehkasilta road across an area of soft ground comprising 5m of peat over 4-5m of mud and soft clay. In common with many other amenity tracks the new track was surfaced with asphalt to permit its use by skiers year round. Three sections of the track were constructed over peat using chipped dry wood bark as a lightweight fill material. This material was primarily chosen due to the proximity of a suitable sawmill with a source of woodchip and the cost of the installation of the material.



The lightweight embankment construction sequence was as follows:

- Drainage ditches crossing the line of the new embankment were backfilled with wet excavated peat ahead of the main construction works to produce a uniform subgrade for the foundation of the new embankment.
- Chipped forest bark was delivered to site and installed as a 800mm layer brought to line and level
- A Class III non woven geotextile was unrolled across the lightweight core to enclose the material and act as a separator for the higher layers
- A Tensar SS20 polypropolene geogrid was then laid across the prepared formation
- The completed embankment was then topped with 600mm of crushed aggregate and the running surface finished with 40mm of asphalt.

All lengths of track were completed successfully using the above procedure although one length settled significantly due to overloading during the construction sequence.

Chipped lightweight wood bark embankment core

Installation of geotextile and geogrid on the wood chip embankment

Source: J. Immonen, Finnish Roads Administration, Helsinki 2003

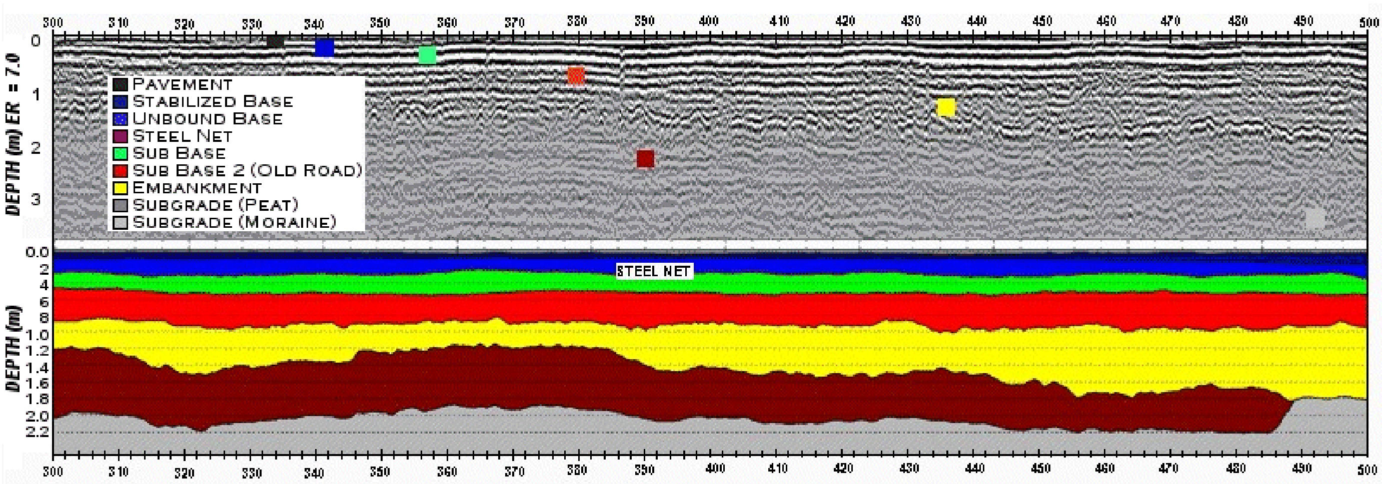
Case Study F13	Road No 930 Mellajärvi steel reinforcement, Finland	Date	1999
AADT 220	Heavy vehicles 9%	Speed limit 100km/h	Carriageway width 6.6/7.0m

Road 930 between Ylitornio and Muurola was constructed in 1962 and in the early 90's the section at Mellajärvi was selected as a site for improvement under the 1997-1999 MISU project for testing new road survey methods and developing integrated road analysis techniques based on the survey results.

The steel reinforced rehabilitation structure at Mellajärvi as outlined below was developed as part of the project.

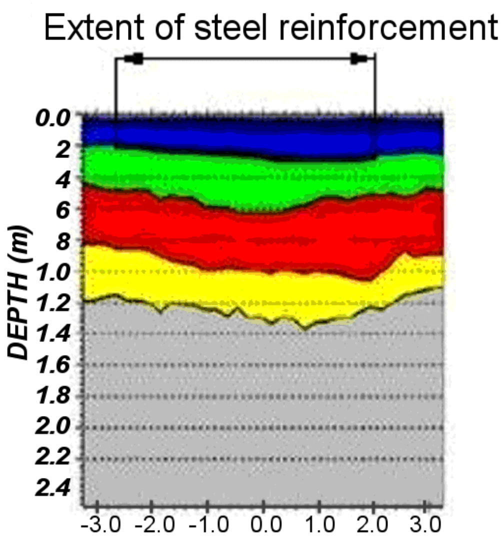
Photograph of Road 930 before rehabilitation

Steel grid reinforcement is not new in Finland and the design for the Mellajärvi test section called for the grid to be installed at a depth of 250mm for optimum reinforcement against longitudinal cracking and permanent deformation. The existing bituminous pavement layers were milled out and the underlying unbound basecourse exposed. Welded steel mesh reinforcement sheets were then installed at 100mm into this unbound layer and once installed an additional 100mm of unbound basecourse was added and compacted. This top 100mm layer was then stabilised by remixing with emulsified bitumen and compacted before a final 40mm asphalt wearing course was laid to complete the structure. By this means the steel grid was installed at its designed depth of 250mm



Ground penetrating radar plot and interpretation

The Mellajärvi scheme is a good example of the beneficial use of steel grid reinforcement within road structures for strengthening roads over peat. The MISU improved section is performing well since the grids were installed. One lesson learned was that the steel grids must extend over the full width of the carriageway and should extend under the road shoulders. If not cracking will concentrate at the end of the grid as can be seen in the photograph on the right taken after the works. A fuller account of this project can be found on the 2001 Roadex CD ROM.



Source: T.Saarenketo, Oulu 1999 and Roadex Project 2001

10.3 CASE HISTORIES: NORWAY

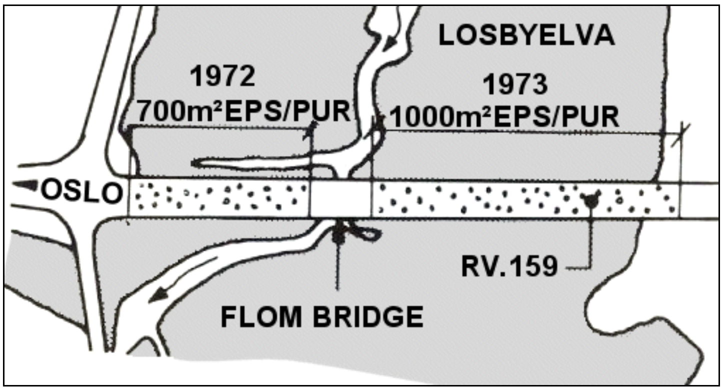
The following case histories are presented with the permission of the contributors:

N1	Road No Rv-159 Flom	1972
N2	Road No Rv-154 Solbotmoan	1975
N3	Road No Rv-610 Sande - Osen, Sogn og Fjordane District	1983
N4	Forest Road, Slatjernmosen, Rømskog	1986
N5	Road No Ev-6 Sandmoen, Sør-Trøndelag	1986
N6	Road No Fv-102, Nordre Mangen, Akerhus	1987
N7	Access road to Harøy windfarm, Sandøy, Møre & Romsdal	1999
N8	Road No Fv-228, Fræna Kommune, Møre & Romsdal	2001
N9	Road No Ev-10 Austerstraumen – Gullesfjordbotn	2003

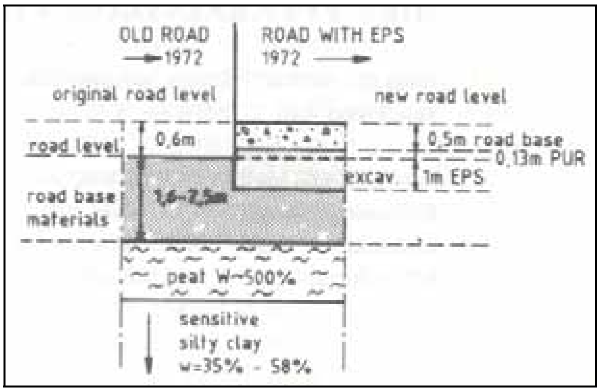
Figure 74. Road No Ev-10 Austerstraumen – Gullesfjordbotn.

Case History N1	Road No Rv-159 Flom, Norway					Date	1972
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0m

Road No Rv-159 at Flom in the District of Akerhus was the first road to use EPS as a lightweight fill material. In 1972 road Rv-159 was a main route into Oslo from the east carrying 15,000 vehicles per day. At Flom the road crossed a deep boggy area of 3m peat over 10m of soft clay by means of a low embankment and a small bridge on concrete piles. The approach embankments to the bridge were settling fast at approximately 20-30cm per year and at the time were almost 60cm lower than the bridge approaches and a potential hazard for traffic. Various alternative solutions were considered to resolve the situation and eventually the Roads District Office opted for an 'offloading' exercise using EPS blocks as the lightweight fill for the replacement embankment. The aim of the project would be to reduce the load on the peat bog by 10 kPa.



Site plan at Flom



Cross-section showing old v new construction

In the event the chosen solution in 1975 was an offloading exercise that removed the top 80cm of the existing embankment and replaced it with a 1.1m thick layer of EPS blocks with a 13mm sprayed coating of polyurethane and a 50cm road pavement. This exercise resulted in an offloading of 5 kPa on the underlying peat. No particular problems were reported on site during construction of the embankment apart from some difficulties spraying the specified polyurethane coating to the finished EPS installation. This requirement for a sprayed polyurethane finish is no longer used on EPS in road embankments in Norway.

Photograph of Flom embankment during construction

Photograph of Flom during flooding event in 1987

There is an interesting postscript to this project. On 16 October 1987 northern Europe was hit by exceptionally severe weather with high winds and rainfall. River levels rose and Norway experienced major flooding events countrywide. At Flom the river and ground water levels rose and the road embankment was inundated with flood water. Despite having been designed for normal buoyancy forces the EPS core of the embankment lifted under the action of water and the individual blocks floated apart.

When the floods abated and the damaged embankment was inspected it was found that the individual EPS components were still intact and a decision was taken to rebuild the road again using the same method and blocks. The reconstruction using the 'recycled' blocks was successful and the road is still in service.

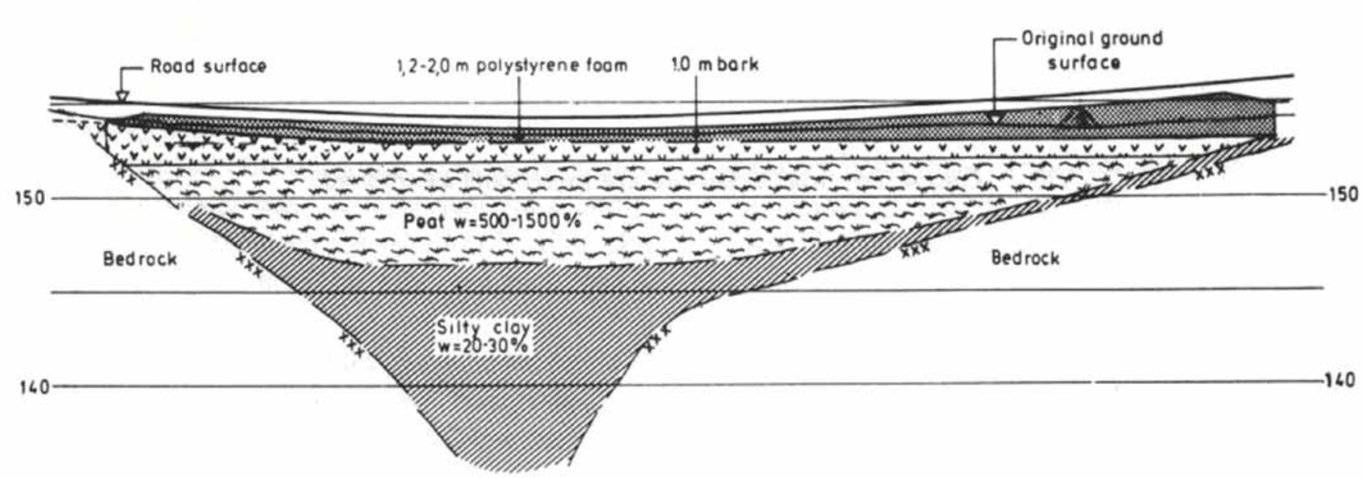
Source: Veglaboratoriet Report 61 'Plastic Foam in Road Embankments', 1987 and Vegdirektoratet Internal Report 1885 'Expanded Polystyrene – The Light Solution', 1996

Case History N2	Road No Rv-154 Solbotmoan, Norway					Date	1975
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.0m

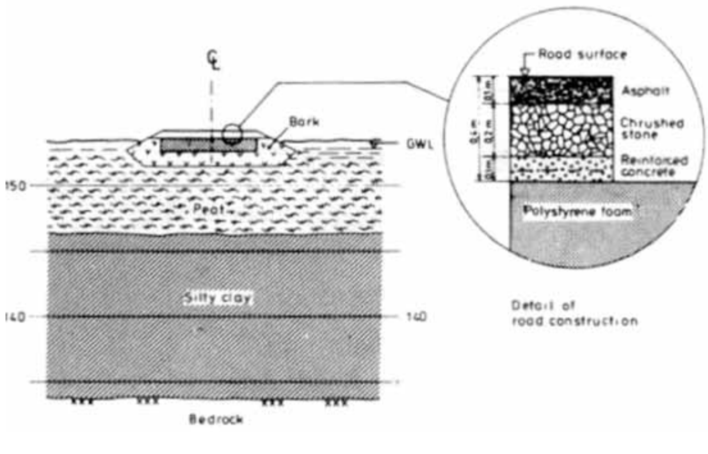
Road No Rv-154 crossed a very boggy area at Solbotmoan, 30km south east of Oslo, and the road regularly flooded twice a year. Each application of new materials that was placed to raise the road out of the bog caused a further settlement of the road until finally in 1975 the original subgrade level had settled 1.7m below its original level. At this point it was noticed that settlement was increasing and cracks were appearing indicating a danger of imminent structural collapse.

The road needed to be raised above the flood level of the bog over a distance of some 140m with a reduction in the weight of the embankment. The use of conventional materials was ruled out as it was considered that they would lead to further accelerated settlement and collapse and a decision was taken to use EPS to reform the existing embankment at a lower weight and higher level.

To achieve this the existing road embankment and pavement layers were excavated out to a depth of 1.0m and chipped bark used as a filling material up to the normal groundwater table. After compacting and levelling this bark layer an EPS block embankment varying in thickness from 0.5m to 2.7m was placed on the prepared surface as below.



Longitudinal section showing EPS embankment



Cross-section of old and new construction **Photograph of the EPS embankment at Solbotmoan**

On completion the finished EPS embankment was capped with a 10cm thick reinforced concrete slab (see case history N3), topped by a road construction of 20cm of crushed rock roadbase and 10cm of asphalt.

Source: Veglaboratoriet Report 61 'Plastic Foam in Road Embankments', 1987

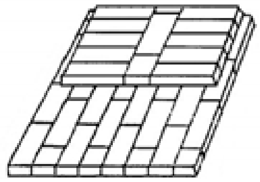
Case History N3	Road No Rv-610 Sande - Osen, Sogn og Fjordane District, Norway					Date	1983
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	7.0m

State Road No Rv-610 was upgraded in the early 1980's and in the section between Sande and Osen in Sogn og Fjordane District the new alignment was scheduled to cross a 200m wide boggy area of peat and clay 10-12m deep. The local Roads District Office decided to construct the works using an expanded polystyrene (EPS) solution with the Norwegian Road Research Laboratory as consultants. The organic soils in the bog were not able to sustain any increase in effective stress without undergoing unacceptable settlements and as a consequence the load of the new embankment had to be carried by the buoyancy of the EPS blocks. In practice this meant that the ground water had to be maintained within a specified limit of +/-30cm.

The sequence of construction was as follows:

- A 9m wide by 1.2m deep trench was excavated in the peat surface and the bottom of the excavation brought to level with sawdust

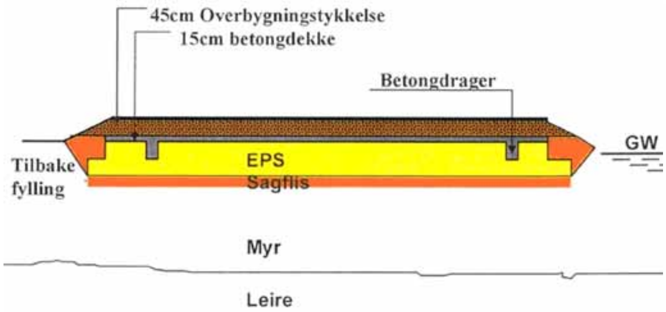
- The EPS embankment was constructed by placing the first layer of EPS blocks on the prepared sawdust formation with the block longest dimension along the road. The second layer of blocks was then placed at right angles to the bottom layer leaving a 25cm gap on each side of the layer for a reinforced concrete rib to be formed.



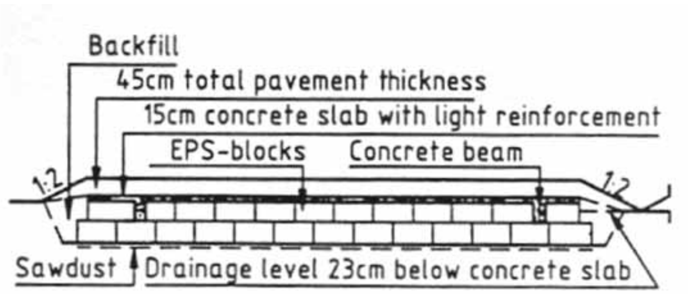
Laying arrangement

- Edge formwork was erected and a 15cm thick reinforced concrete slab cast on top of the EPS blocks including integral ribs

- A 45cm thick standard road construction of crushed aggregates finished with an asphalt wearing course was then placed on the finished slab



Cross-section through EPS embankment



Detail of EPS embankment construction

Placing of EPS blocks on sawdust layer

Casting of concrete slab and integral ribs

Very heavy rain was experienced almost continuously during the embankment construction work and this posed numerous problems with water in the excavations. These were overcome through local drainage measures by site staff and the road completed as scheduled. The road is still performing satisfactorily 20 years later.

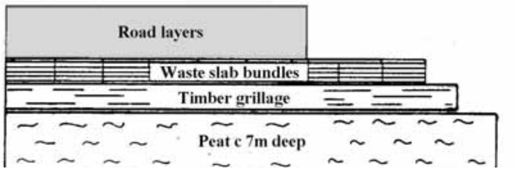
Source: Veglaboratoriet Report 61 'Plastic Foam in Road Embankments', 1987 and Vegdirektoratet Internal Report 2209 'EPS – den lette løsningen', 2001

Case History N4		Forest Road, Slatjernmosen, Rømskog, Norway				Date	1986
AADT	50	Heavy vehicles	30 %	Speed limit	50 km/h	Carriageway width	4.0 m

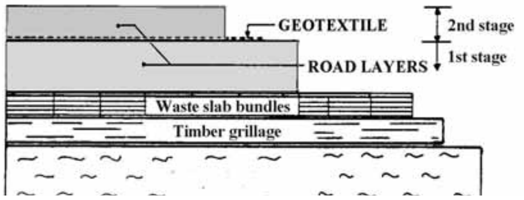
This project dealt with the reinforcement of a forest road resting on a 7m deep peat bog at Slatjernmosen, Rømskog. In addition to providing access to the forest timber operations the road was also the main transport route to a working chalk quarry within the forest area.

The history of the construction of the road involved 3 stages:

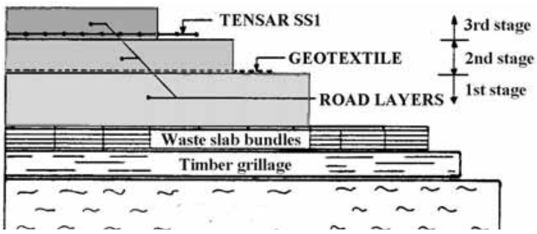
Stage 1: The road was constructed using a typical local forest road construction method for crossing peat bogs that comprised a timber grillage covered with a layer of bundles of waste sawmill cuttings that were available locally as a by-product from nearby sawmilling industries. This floating platform of timber products was a quick solution used by the forest industry at the time and generally produced a good robust floating platform on which to build an internal forest haul road.



Stage 2: The thin unreinforced gravel construction above the 'floating' forest road started to develop significant settlement and rutting problems shortly after trafficking and it was decided to improve the construction by reinforcing the existing layers with a single layer of a class 3 geotexile to try to keep the road passable for timber haulage. The top surface of the existing road was graded level in preparation for the geotextile to be unrolled and a new 15-20 cm of crushed gravel base and wearing course was added and compacted. This 'reinforced' road soon started to exhibit the same degree of deformation and rutting in the wheel tracks as the original unreinforced road and required further attention.



Stage 3: A geogrid reinforcement solution to the wheel tracking was proposed using a Tensar SS1 geogrid. The surface of the existing 'Stage 2' construction was again regraded to produce a good sound formation to work from and a SS1 geogrid rolled out in a 4m width along the prepared section. Crushed gravel with a grading of 0-30mm was laid in a 20cm layer on top of the grid and well compacted into the grid to ensure that the new road construction aggregates effectively interlocked with the grid.



The geogrid reinforced road showed an immediate improvement in performance and a monitoring exercise was established for the next 2 years to check if the wheel tracking would return. In the event this did not happen despite heavy trafficking and a summary of the level survey is shown below.

	Left wheel track					Right wheel track				
Pkt.nr.	E	D	C	B	A	A	B	C	D	E
1	1.40	1.40	1.40	1.40	1.60	1.65	1.40	1.40	1.39	1.40
2	1.50	1.49	1.50	1.50	1.69	1.69	1.48	1.48	1.49	1.50
3	1.50	1.49	1.46	1.46	1.66	1.64	1.46	1.46	1.49	1.50
4	1.48	1.47	1.44	1.41	1.61	1.59	1.45	1.45	1.48	1.48
5	1.52	1.50	1.44	1.42	1.61	1.61	1.45	1.45	1.50	1.51
6	1.57	1.55	1.50	1.47	1.69	1.63	1.46	1.48	1.54	1.56
7	1.60	1.57	1.51	1.50	1.69	1.70	1.52	1.52	1.57	1.59
8	1.60	1.57	1.52	1.51	1.66	1.68	1.55	1.55	1.57	1.61
9	1.55	1.52	1.46	1.42	1.68	1.64	1.46	1.46	1.52	1.54
10	1.53	1.49	1.43	1.39	1.67	1.63	1.38	1.40	1.46	1.49
11	1.46	1.44	1.37	1.36	1.62	1.60	1.35	1.36	1.43	1.44
12	1.40	1.38	1.35	1.34	1.53	1.56	1.36	1.36	1.37	1.38
13	1.38	1.37	1.35	1.31	1.51	1.49	1.29	1.30	1.33	1.35
14	1.38	1.37	1.37	1.33	1.51	1.51	1.31	1.31	1.33	1.33
15	1.26	1.33	1.35	1.29	1.50	1.45	1.26	1.28	1.27	1.28

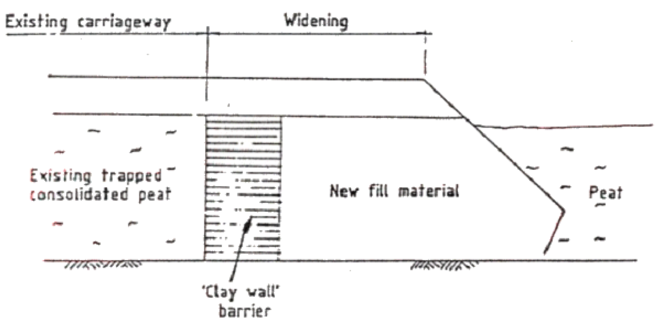
(History references A: before installation of Tensar geogrid and new road layer, B: after installation of Tensar SS1 and compaction 2 Oct 1986, C: after haulage of 1500m^3 of gravels 24 Oct 1986, D: after transport of 1000m^3 of timber 15 July 1987, E: after transport of timber and haulage of 180 tonnes chalk 18 Oct 1988)

Source: S Stokkebø, Nor-Vest AS / Stokkebø Competanse AS

Case History N5	Road No Ev-6 Sandmoen, Sør-Trøndelag, Norway	Date	1986

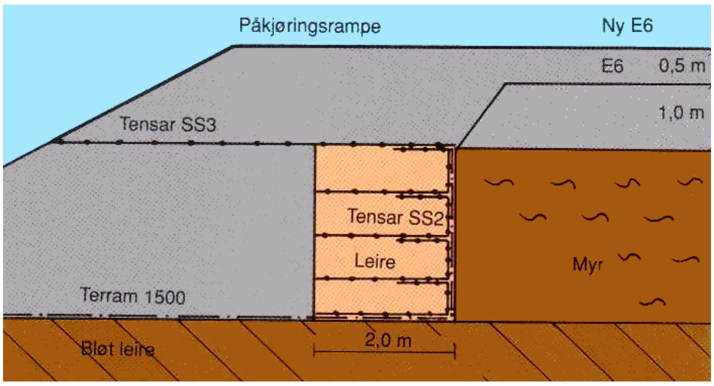
AADT	n/a	Heavy vehicles	15%	Speed limit	80km/h	Carriageway width	8.5 m

This project involved the construction of 2 approach ramps to and from a new grade separated junction and overbridge to Trondheim on the E6 national road at Sandmoen, Sør Trøndelag. The design of the works was carried out by the Trondheim Kommune office in conjunction with the Sør Trøndelag District office of the National Roads Administration with Nor-Vest AS as geosynthetic consultants.



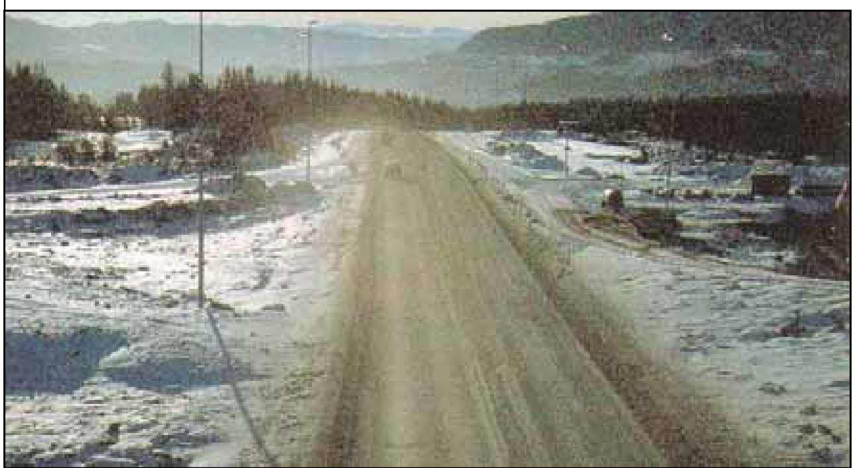
The Sør Trøndelag area and Trondheim Kommune had extensive experience of building and widening roads over peat. The planning of the work, carried out by a local consulting company, required that the approach roads should be built using a standard 'soil replacement' technique to widen of the existing E6 road shoulders. Under this design the existing consolidated peat beneath the old road was to be left in place as it was considered it to have developed sufficient bearing capacity to support the new loadings. No extra works were stated to prevent drainage of the existing E6. The new ramps were to be founded on the clay beneath the peat, a geotextile separator backfilled wth rockfill, a baselayer of crushed rock and finished wth an asphalt pavement.

The local Trondheim Kommune engineer recognised that a drainage problem could be created when the new volumes of gravel or rockfill were placed against existing peat below the E6. In particular he considered that the new free draining constructions in the road widening could act as a linear drain for the peat below the existing road and so cause increased settlements under the road when the old consolidated peat, previously in hydrological balance below the road, dewatered. A revised design was therefore proposed that attempted to isolate the new widened constructions from the existing road structure to retain the existing consolidated properties of the trapped peat. As further safeguard the widening at Sandmoen was carried out in winter whilst the peat was frozen and excavations were immediately sealed with the clay wall to further prevent dewatering of the existing peat.



The final design of the clay wall between the new and old construction took the form of a reinforced earth structure following consultations with Nor-Vest AS, Tensar-distributor in Norway at that time. The clay wall structure developed is shown to the right and involves a 2.0m wide clay plug reinforced with Tensar SS2 geogrids laid on the compacted clay surface at 0.5m centres and returned vertically up the exposed face of the excavated peat. The clay material used in this process was a 'workable' clay excavated on site from selected suitable areas. A layer of geogrid, Tensar SS3, was laid over the total widening at the top of the peat / clay barrier level to prevent damage from possible different settlements in the different soils over the width of the widening.

This novel form of construction has worked out very well since installation with no unexpected problems in the ramps or the existing E6.



Sources: Trondheim Kommune and S Stokkebø, Nor-Vest AS / Stokkebø Competanse AS

Case History N6	Road No Fv-102, Nordre Mangen, Akerhus, Norway					Date	1987
AADT	300	Heavy vehicles	20 %	Speed limit	80 km/h	Carriageway width	8.0 m

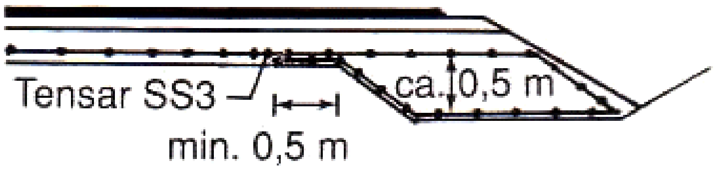
Road No Fv-102 was an unsurfaced public gravel road in 1986 that led to sawmills operated by Stangeskovene AS. The road was heavily trafficked and had a history of problems of poor bearing capacity and as a result a decision was taken to upgrade, strengthen and widen the road with a view to paving the widened carriageway with asphalt.

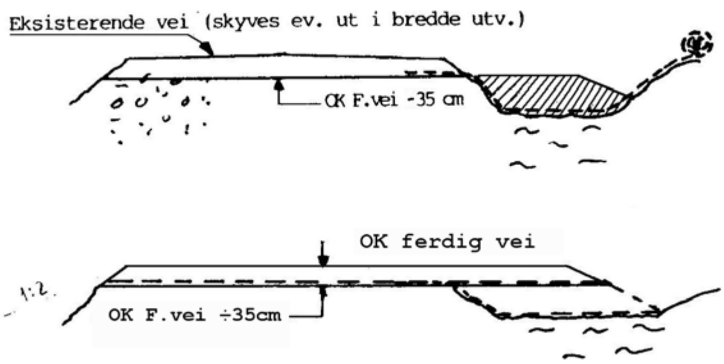
Part of this section of Road Fv 102 passed over a 230m long peat bog of up to 4m deep. In addition to the problems of the low bearing capacity of the existing road it was known that the road edge support was poor, such that the road had a temporarily weight restriction of 6 tonnes axle load during the spring thawing season to protect it until the strengthening works could be effected. This restriction posed great problems for the sawmilling operations of Stangeskovene AS and a quick resolution was required to reopen the public road to normal traffic flows.

A number of alternative solutions were considered for the works including excavation and replacement of the 4m of peat. In the end a geogrid based reinforcement solution was chosen as the most cost effective technique based on the Tensar SS3 polypropylene geogrid.

The chosen solution involved a reinforcement of the existing gravel road and a reinforced widening as follows:

- The widening area was prepared by excavating a shallow trench into the adjacent peat of up to 0.5m below the existing road level to provide a constant shape on which to form the new construction.
- A geotextile class II was laid as a separating layer to keep underlying peat from mixing with the new construction.
- A Tensar SS3 geogrid was laid on across the existing gravel road from 0.5 m past the wheeltrack and out under the widening.
- The new widening was backfilled with a 15 cm thick layer of crushed stone 0-70mm (for interlocking) + rockfill with compaction up to level with the adjacent road.
- The geogrid was then pulled over the top of new construction and onto the full width of the widening and the existing road.
- The full width of the widened road and shoulders was completed with 30cm of crushed aggregate 0-70mm road construction to provide interlock with the geogrid.





The road was left in the above state for the winter of 1987/88 without problem and has since been surfaced with asphalt.

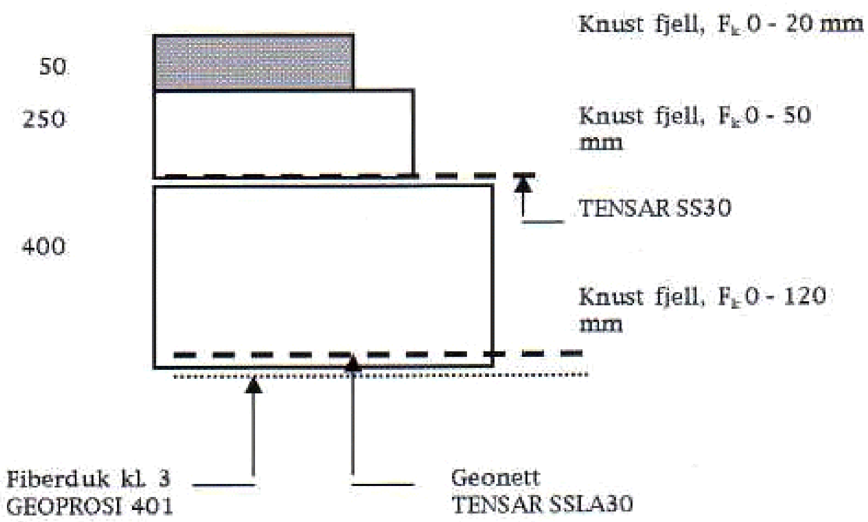
A lesson learned in this project was that it was very difficult to construct the widening to a sufficient tolerance to be able to easily wrap the geogrid back over on itself. At the outset of the design it was thought that it was necessary to have the geogrid continuous around the new construction to give a degree of bending resistance. This is no longer considered necessary and in current installations geogrids are installed in 2 separate layers, one in the base of the widening and one in the top. This arrangement gives the same results as the 'wrap around' method and is easier to install.

Source: S Stokkebø, Stokkebø Competanse AS

Case History N7	Access road to Harøy windfarm, Sandøy, Møre & Romsdal, Norway	Date	1999

This project involved a gravel access road for heavy construction traffic for a new wind farm development for Kraftmontasje AS at Harøy in the Sandøy Kommune. The access road was designed primarily for the very heavy construction phase traffic that included the heavy haulage loads of the windmill turbines, blades and tower sections being delivered to site and the heavy lift cranes (up to 330 tonnes lifting capacity) for the assembly of the delivered units.

On site it was found that turbines at sites 1 & 2 could be installed on sound ground but the route to wind turbines at sites 3, 4 & 5 crossed a 5m-6m deep peat bog that was expected to comprise a 'stiff' peat with a CBR of around 1.0. Initially this section was considered as a fairly deep 'excavation and replacement' exercise but in the event the contractor Braute Maskin AS opted for a 'floating' road solution over the bog. The new road was required to be designed for the short duration construction traffic and a long term in-service axle load of 12 tonnes. The existing surface vegetation mat was to be retained intact to act as a natural reinforcement. The decision to proceed with this amended design was later proved to be a sound decision when the peat in the lower bog was found to be wetter and more liquid than was expected.



Cross-section of 'stiff plate' geogrid sandwich

The design of the haul road was revised by geosynthetic consultant GEOPRO to meet the new ground conditions and the new road constructed as a 'stiff plate' geogrid sandwich to provide a 14m wide platform over the peat. In essence a 7m wide carriageway and two 3.5m wide shoulders were provided. The road layers were constructed with a basal separator geotextile, a basal 'Tensar SSLA30' polypropylene geogrid, 400mm of crushed rock aggregates 0-120mm size, a mid level 'Tensar SS30 polypropylene geogrid, 250mm of subbase 0-50mm size and a 50mm thick wearing course of 0-20mm crushed aggregate. Finally excavated peat was placed on the wide shoulders, on top of the lower geogrid and crushed rock layer in order to widen and stabilise the new road platform against failure, and provide some organic cover to establish vegetation.

View of finished road and wide verges

View of finished access road to a wind turbine

Since construction in 1999 the road has functioned well under the heavy site traffic loading without rutting. The decision to 'float' the road rather than drain and excavate the peat bog has been proven to be a sound and cost effective decision. The project was completed on time to the satisfaction of the developer. The access road is now only used for maintenance traffic.

Sources: SR-BR avd. GEOPRO, S. Stokkebø, Stokkebø Competance AS

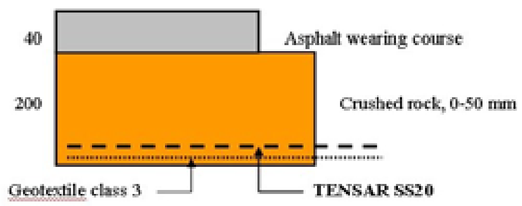
Case History N8	Road No Fv-228, Fræna Kommune, Møre & Romsdal, Norway					Date	2001
AADT	300	Heavy vehicles	10 %	Speed limit	60 km/h	Carriageway width	4.7m

This project concerned the upgrading and strengthening of Road No 228 in Fræna, Møre & Romsdal. The work involved was issued for tender as a 'function-contract', where the contractors were able to price alternative solutions. As part of this tender process prospective main contractors, Per Olaf Vassgård Maskin AS, invited geosynthetic consultant Stokkebø Competanse AS to produce a range of geogrid based reinforcement options to meet the likely geotechnical, alignment and pavement circumstances along the route. These options would act as a site 'toolkit' for the contractor's local staff to deal with the various ground conditions expected (from sound moraine to deep peat areas) as well as the different forms of improvements to be carried out (on line strengthening, widening and in diversion).

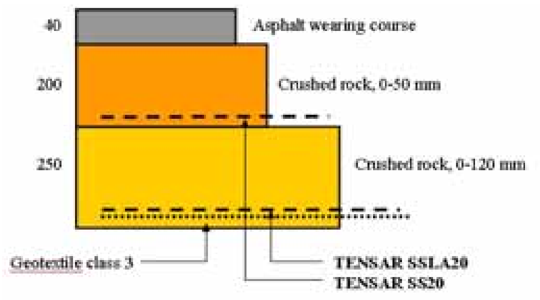
Geogrids have been used in the reinforcement of roads in Norway since the late 1970's and their application is now relatively commonplace especially in those cases where mass excavation and disposal of the peat is considered to be uneconomic. The use of geogrids is widely considered to give a satisfactory solution for roads resting on weak soils or for binding newly widened areas to the main construction works.

In the event the 'toolkit' produced under the project at Fræna allowed the contractor to assess the particular cluster of problems at a locality and select a 'fit for purpose' solution from a range of workable options. 4 examples from the range of options dealing with the upgrading of the road over sections over peat are shown below for information. The existing road embankment is assumed to be a minimum of at least 30cm thick in all of the cases illustrated.

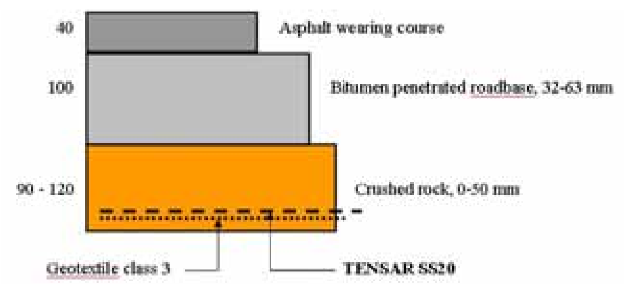
Detail A – Strengthening of existing road over peat



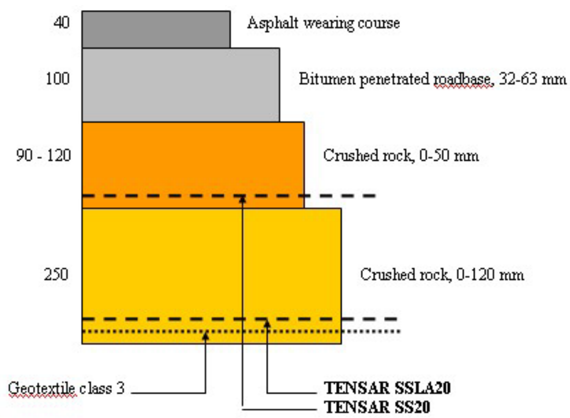
Detail B – Widening of roads over peat



Detail C – Strengthening of existing road over peat



Detail D – Widening of roads over peat



Photographs of the installation of geogrids and crushed rock on Road Fv-228, Fræna

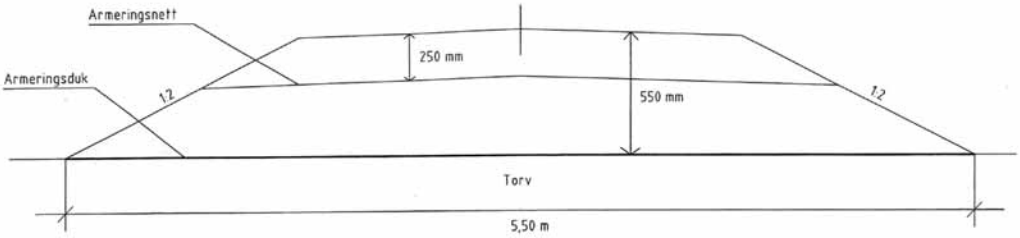
Source: S Stokkebø, Stokkebø Competanse AS

Case History N9	Road No Ev-10 Gullesfjordbotn - Austerstraumen, Norland, Norway					Date	2003
AADT	600	**Heavy vehicles**	10%	**AADT**	600	**Heavy vehicles**	10%

The 14.6km Ev-10 Gullesfjordbotn – Austerstraumen project is part of the 29.6km Lofoten Islands Gullesfjordbotn to Raftsundet link, otherwise known as 'Lofast I'I. This ambitious project aims to give the 23,000 residents and industries of Lofoten a fixed link to the mainland of Norway in place of the series of existing ferries that are overwhelmed in summer with tourist traffic.

The road section of the case study concerned the westerly approach to the new 6.4km Sørdals tunnel through the Brynjulfslåttheia mountain range. This section was constructed as a 3m wide temporary haul road over virgin peatland to access the tunnel face. When this temporary road reached the tunnel portal and tunnelling commenced the permanent road was constructed back from the tunnel using crushed rock spoil generated from the tunnel workings. This innovative scheme gave a cost effective solution through an area of recognised natural beauty and ensured that the maximum volume of fill material was generated within the site thereby minimising adverse effects on the local landscape.

The design of the haul road had to cater for the heavy axle loadings of the site dump trucks hauling fill materials to the front of the embankment as it advanced across the peatland towards the tunnel. The design solution chosen comprised a high strength TeleVev 150/150 geotextile laid directly on the undisturbed peatland surface, 300mm of 'as dug' gravel fill, a TeleGrid 30/30 polypropylene geogrid and a final 'surfacing' layer of 250mm of gravel pavement.



Cross-section across the temporary haul road

Filling over geotextile placed on bog surface

Rolling out geogrid on embankment and laying pavement

This 7 km long temporary road was driven to the tunnel portal in 3 months to ensure that construction of the tunnel could commence ahead of winter conditions. Barring unforeseen problems the permanent Ev-10 road will open in autumn 2007.

Source: Vidar Engmo and Einar Karlsen, Vegvesen.

10.4 CASE HISTORIES: SCOTLAND

The following case histories are presented with the permission of the contributors:

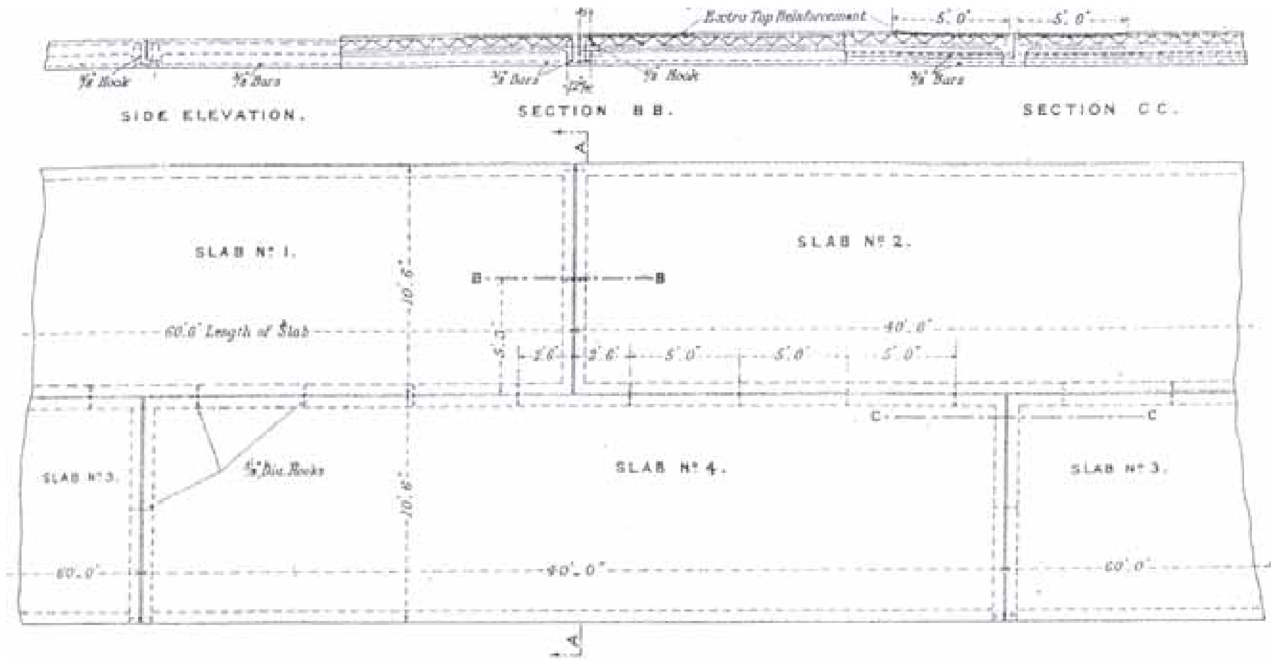
Sc1	A9 Slochd Summit, Inverness	1927
Sc2	A837 Loch Assynt, Sutherland	1960
Sc3	A835 Elphin, Sutherland	1961
Sc4	A838 Laxford – Rhiconich phase II, Sutherland	1990
Sc5	A837 Ledbeg Lightweight Embankment, Sutherland	1991
Sc6	Subsea 7 Fabrication trackway, Wester, Caithness	1991
Sc7	B876 Killimster Moss rehabilitation, Caithness	2000
Sc8	B8043 Kingairloch Road Diversion, Lochaber	2003
Sc9	B871 Loch Rosail tyre bale embankment, Sutherland	2003
Sc10	Causewaymire Windfarm access roads, Caithness	2004

Figure 75. A838 Laxford – Rhiconich phase II, Sutherland.

Case Study Sc1	A9 Slochd Summit, Inverness, Scotland					Date	1927
AADT	n/a	Heavy vehicles	n/a	Speed limit	100 km/h	Carriageway width	5.5m

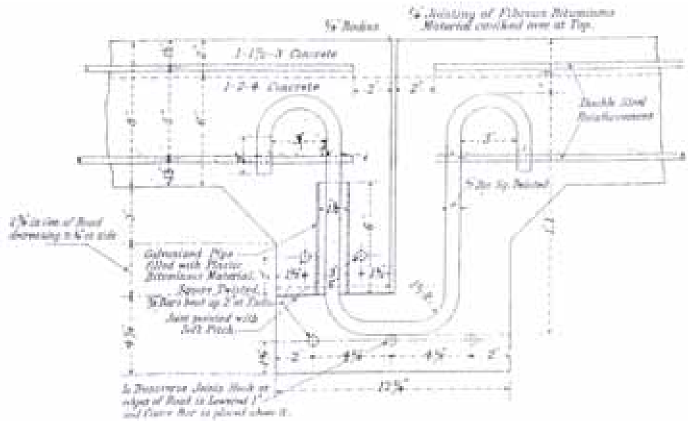
125km of the main A9 national route from Perth to Inverness, "The Great North Road", was upgraded in the 1930's as a 'work creation' programme of road improvements through the Ministry of Transport. At five places in the project the road was carried across peat bog, up to 5m deep, and it was decide to construct the new road on reinforced concrete rafts at these locations. The total length of concrete raft involved was 1700m.

This form of construction was considered novel at the time particularly the use of side beams to stiffen the structure, the staggered overlapping slab arrangement to prevent independent vertical movement, and the use of tie hooks as hinges to hold the individual raft elements together. All of these features can be seen in the construction drawings below:



Plan & cross-sections of overlapping slab arrangements

The design of the raft called for a 200mm thick slab with edge stiffening on all 4 sides. Top and bottom reinforcement was provided by 8mm bars at 200mm crs square grip welded mesh held apart by 6mm diagonal shear reinforcement. A further layer of reinforcement was provided along each joint for additional strength. Both the longitudinal and transverse joints of each slab were designed to form an overlap, one edge-beam resting on a nib cast previously in the adjacent slab. Each slab was then tied to its neighbour by a U-shaped hinge through the edge overlap that tied the structural elements together for continuity.



The finished raft was given a wearing surface shortly after opening comprising a thin layer of cold bitumen emulsion with a blinding of granite chippings.

Cross-section through joint between slabs

In 1976 a further upgrading of the A9 was carried out and the reinforced concrete raft section at Slochd summit, south of Inverness, was exposed and removed for the new alignment. During this wok opportunity was taken to inspect the condition of the raft and underlying layers after 50 years of service. Surprisingly, the raft was still serviceable even though it had not been constructed in accordance with the recorded specification. The investigation team found that the concrete quality in the raft slab was very poor, probably as a result of the unskilled 'labour relief' workforce and supervisory staff used at the time. Honeycombing was evident throughout the concrete and particularly in the bottom of the slab where the bottom steel was exposed in many places. It was also seen that little attempt had been made at forming the edge beams and nibs although the edges of slabs had been thickened.

200mm of bituminous material had built up over the years with successive maintenance resurfacings and the raft was still performing well. Below the raft an old 'Telford' macadam road 760mm deep was uncovered and 4.25m of peat.

Sources: R Bruce, "The Great North Road over the Grampians", ICE paper 4812, 1931 and New Civil Engineer, 8 April 1976

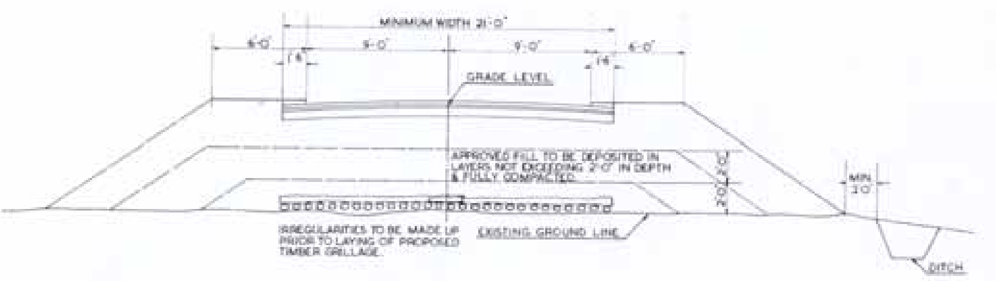
Case Study Sc2	A837 Loch Assynt, Sutherland, Scotland					Date	1960
AADT	405	**Heavy vehicles**	**10%**	**Speed limit**	**100 km/h**	**Carriageway width**	**5.5m**

The Ledmore to Stronecrubie section of the A837 Inveran to Lochinver Road was upgraded from 3m to 5.5m wide in 1960 under the 'Crofter Counties Programme' funded by the Scottish Office. The existing A837 road prior to this improvement was an un-engineered single track road taking the 'line of least resistance' through the landscape and as a result had a very poor horizontal and vertical alignment. The upgraded road was designed to a minimum standard of 80kph stopping sight distance throughout.

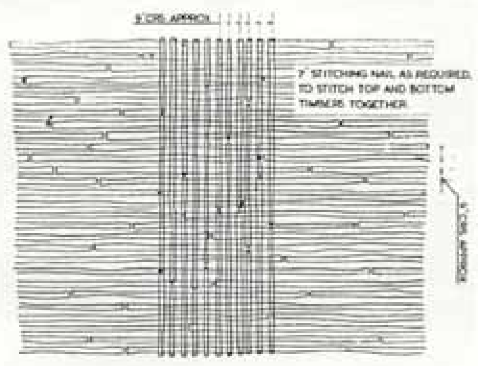
As part of the works the realigned A837 had to cross a 100m long virgin peat bog up to 8m deep at Loch Assynt. This was accomplished by founding the new embankment directly on the peat bog on a timber grillage comprising 2 layers of logs at 225mm centres. The grillage was constructed by first roughly levelling the existing peat surface with a regulating layer of dry peat and brushwood before laying the first layer of logs longitudinally along the line of the road. Once this layer was in place the upper layer of logs was laid transversely to the road line and spiked regularly to the lower layer with 175mm nails to create a sound platform. Thereafter the road embankment was constructed in the standard manner using locally obtained blasted rockfill placed and compacted in 600mm layers.

A837 grillage installation in 1960

A837 grillage embankment in 2003



Cross-section through timber grillage and embankment



Plan of timber grillage layout

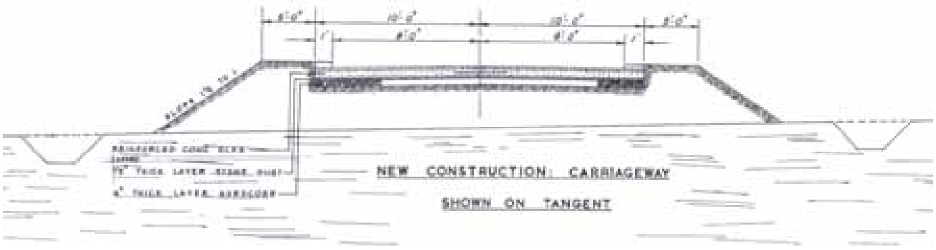
Since construction the carriageway has been surface dressed twice (in 1965 and 1990) with hot bitumen emulsion and 10mm crushed gravel chippings and was still performing well when inspected in January 2004.

Source: A837 contract drawings, 1960

Case Study Sc3	A835 Elphin, Sutherland, Scotland	Date	1961

AADT	585	Heavy vehicles	9%	Speed limit	100 km/h	Carriageway width	5.5m

The A835 public road between Elphin and Ledmore was upgraded and widened in 1961 as part of the Scottish Office's Crofter Counties' programme. At Elphin the design called for a floating concrete raft construction to cross a 200m long predominantly flat peat bog with a depth of greater than 12m.



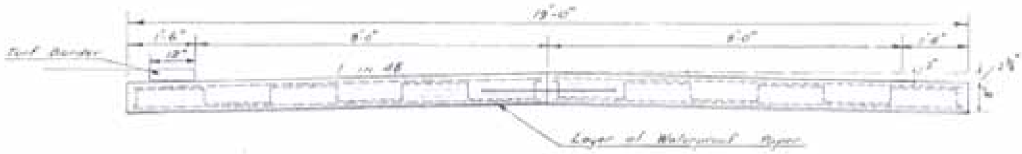
Typical cross-section through embankment and reinforced concrete slab

The slab was constructed in a series of 20cm thick, 20m long reinforced concrete 'half carriageway' bays as below

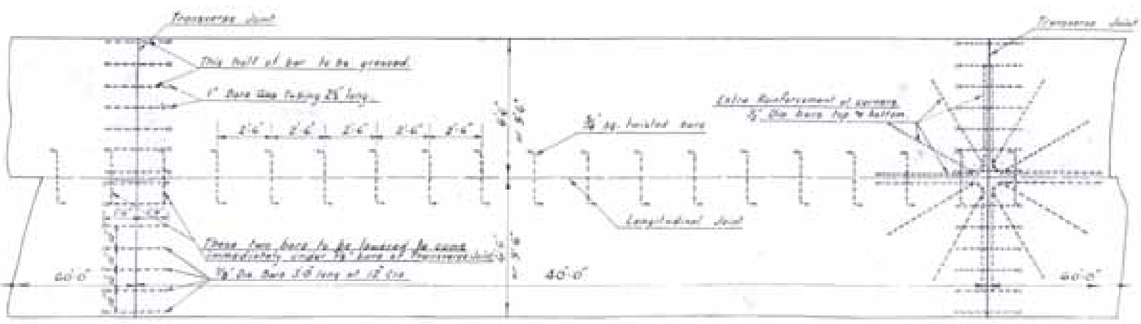
Bay No1	Bay No2	Bay No3	Bay No4	Bay No5

Diagram showing arrangement of slabs

One lane of the new carriageway was constructed over its entire length before the second lane was started. Bays were constructed alternately on a 'hit and miss' basis with two days elapsing before concreting adjacent bays, ie bays Nos 2 and 4 were not concreted until 2 days after bays Nos 1, 3 and 5.



Reinforced concrete slab cross-section



Reinforced concrete slab plan

Each slab was reinforced top and bottom with mild structural steel sheets with 6mm diameter bars at 200mm centres in both directions in the top (2.2kg/sqm) and 8mm diameter bars at 200mm centres in the bottom (3.8kg/sqm).

It was intended that the reinforced concrete road would be constructed above the bog surface and existing road but it is the recollection of a site engineer that the formation was prepared by first excavating out approximately 1.5m of peat, laying 2 layers of chestnut paling fence at 45° to each other and backfilling with selected small rockfill to the required level.

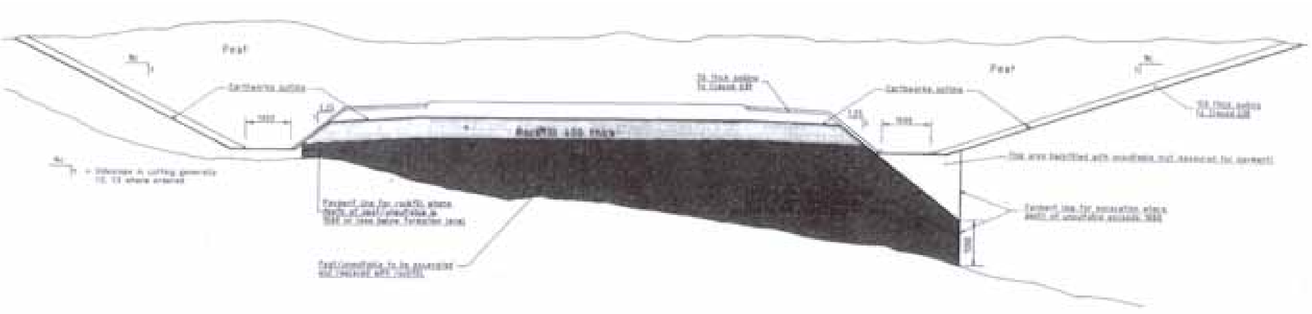
The performance of the slab was monitored by the construction staff for the following six months and it was noticed that the slab could move up to 15cm in response to ground water levels in the bog. The finished slab was left exposed as a running surface for traffic for a period of 2 years after which time all distortion was regulated out and the road surfaced with bituminous macadam.

The road is still functioning as part of the public road network and continues to be as serviceable as adjacent sections of road founded on sound material.

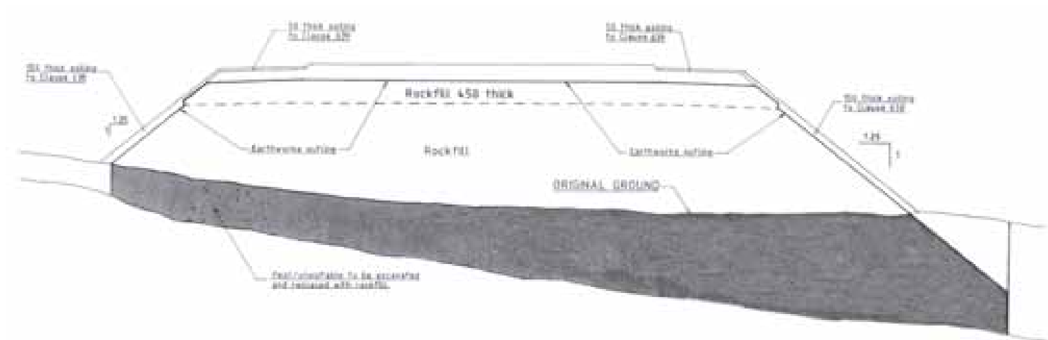
Sources: Contract 18 working drawings, 1959 and Mr H Mackay, Moray Council, Scotland, 2004

Case Study Sc4	**A838 Laxford – Rhiconich phase II, Sutherland, Scotland**					**Date**	**1990**
AADT	405	**Heavy vehicles**	**10%**	**Speed limit**	**100 km/h**	**Carriageway width**	**5.5m**

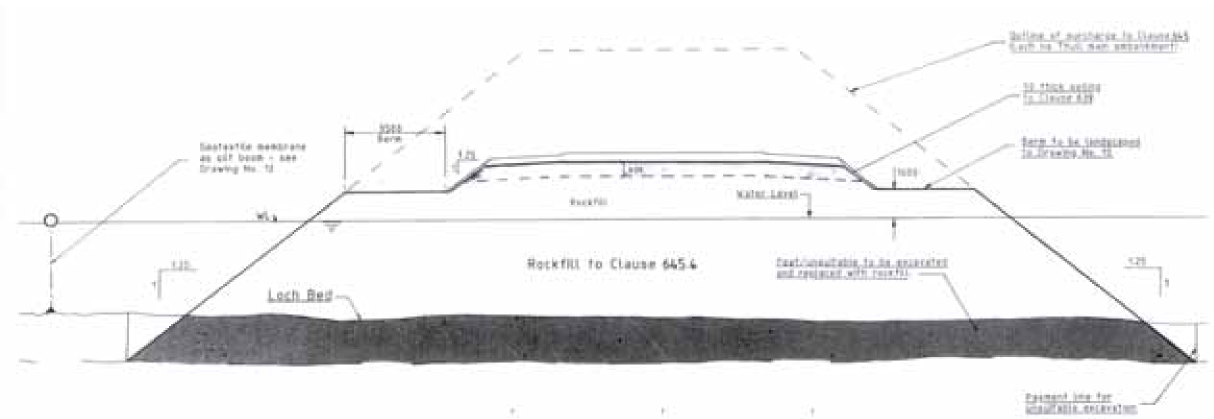
The upgrading of the A894 in west Sutherland is typical of the strategic road improvements carried out in the Scottish Highlands in the 1990's. Roadwork across peatland was governed by the client requirement that "*the design of the earthworks and road pavement shall ensure that all peat below any new construction shall be removed*". As a consequence the normal standard cross-sections for these roads are as below.



Typical cross-section in cutting in peat



Typical cross-section in embankment in peat



Surcharged embankment to deal with submerged peat at "Loch na Thull"

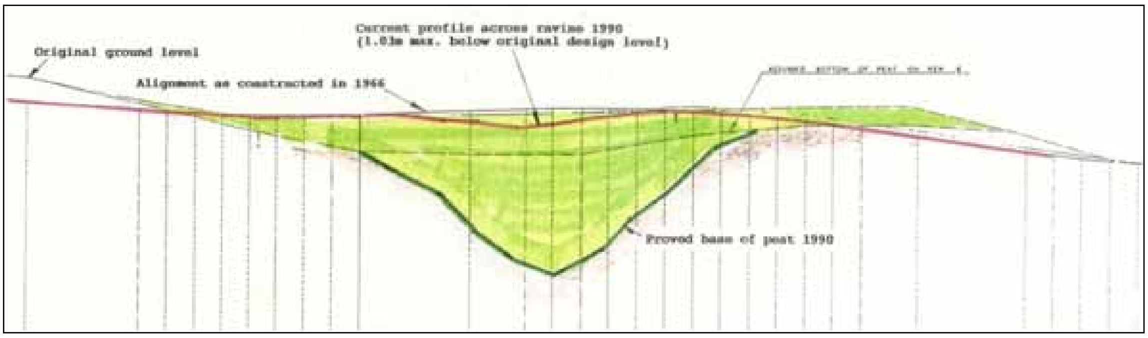
Typical site photographs of peat excavation on strategic roads within The Highland Council area

Source: Project Design Team 3, The Highland Council, TEC Services, Brora, Sutherland

Case Study Sc5	A837 Ledbeg Lightweight Embankment, Sutherland, Scotland					Date	1991
AADT	625	Heavy vehicles	10%	Speed limit	100 km/h	Carriageway width	5.5m

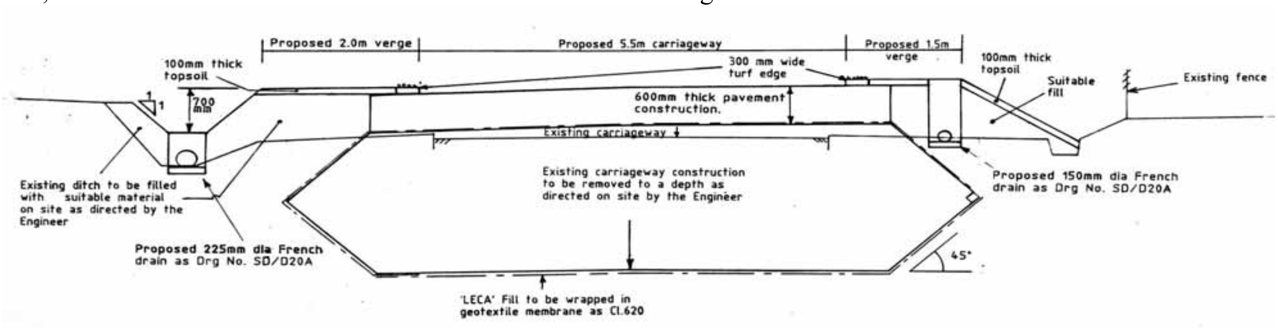
The A837 public road between Ledmore and Skiag was reconstructed in the early 1960's and many sections of the new route crossed deep peat. At Ledbeg the new road crossed a 100m wide ravine of saturated peat (m/c 800%) of maximum depth 12m and soon after construction the road embankment began to settle into the bog. The local Roads Authority attempted to retain the designed profile and alleviate flooding by successively regulating the carriageway with bituminous macadam over the years but this was only effective for short durations. By the summer of 1989 the carriageway at Ledbeg was so severely misshapen with a maximum settlement of approximately 1.0m that substantial corrective measures were required.

Flooding of the carriageway before the project



Long section along peat depression

Initial costings showed that a lightweight fill replacement would be the most cost effective method and a project was designed using the standard practice of the Finnish National Road Administration. Over the years a 1.45m of bituminous material had been laid giving a loading of 3.75 tonnes/m². In 1992 the settling road embankment was 'unloaded' by removing the existing construction layers and replacing them with a lighter LECA substitute topped with 300mm of sub-base, 200mm of unbound roadbase, 60mm bituminous basecourse and 40mm bituminous wearing course.



Cross-section through lightweight embankment

Excavation of existing embankment

Placing of LECA lightweight fill

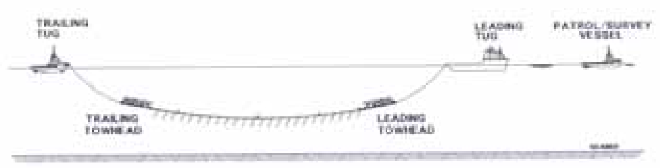
Placing of road construction

Since 'unloading' the embankment and carriageway have performed well and regular monitoring continues.

Source: The Highland Council, TEC Services, Brora, Sutherland

Case Study Sc6	Subsea 7 Fabrication trackway, Wester, Caithness, Scotland	Date	1991

The ‘Controlled Depth Tow Method’ (CDTM) is a cost effective way of delivering fully ready pipelines to oil field wellheads on the seabed. Using this method, bundles of pipes, control lines and instrumentation cables can be transported pre-assembled within a single carrier pipe to their finished operational location suspended between 2 tugs.

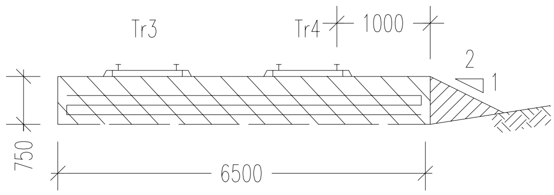


The ‘Subsea 7’ facility at Wester in Caithness fabricates the CDTM pipeline assembly ready for towing out to sea and the process is carried out on an 8km long site that comprises 2 fabrication sheds (one at each end) and a trackway between that traverses various types of soils including a 2.5km section of blanket bog up to 6m deep.

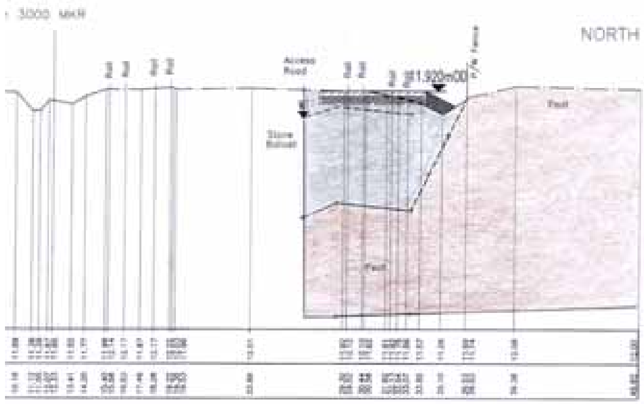
The original trackway at Wester was constructed in 1979 as a two track 1.9km long facility using a rockfill embankment on a Lotrak geotextile laid directly on the peat and was extended to 7.8km in 1996. In 2000 it was decided to commission consulting engineers Ove Arup and Partners Scotland Ltd to design an ‘embankment remediation’ scheme to strengthen the trackway below tracks 3 and 4 to accommodate larger 300 tonne towheads with resulting ground pressures of 25 tonnes/m² that could involve significant stationary periods. A soils investigation was carried out by Fugro Limited using static cone penetration tests, 65mm Mostap sampling and piezometers.

The brief was accomplished by excavating out 750mm of the existing construction and replacing it with a new reinforced pavement structure, comprising a 500mm thick Tensar polypropylene geogrid enclosing 40mm nominal size crushed rock aggregate as below.

Aerial view of site at Wester, Caithness



Cross-section through geogrid envelope



Cross-section across widened pipe trackway

Geogrid backfilled with aggregate

Geogrid envelope being stitched

A towhead on the move

The first heavy tow out using the strengthened track took place successfully in August 2001 and since then 7 further assemblies have used the facility with no adverse effects.

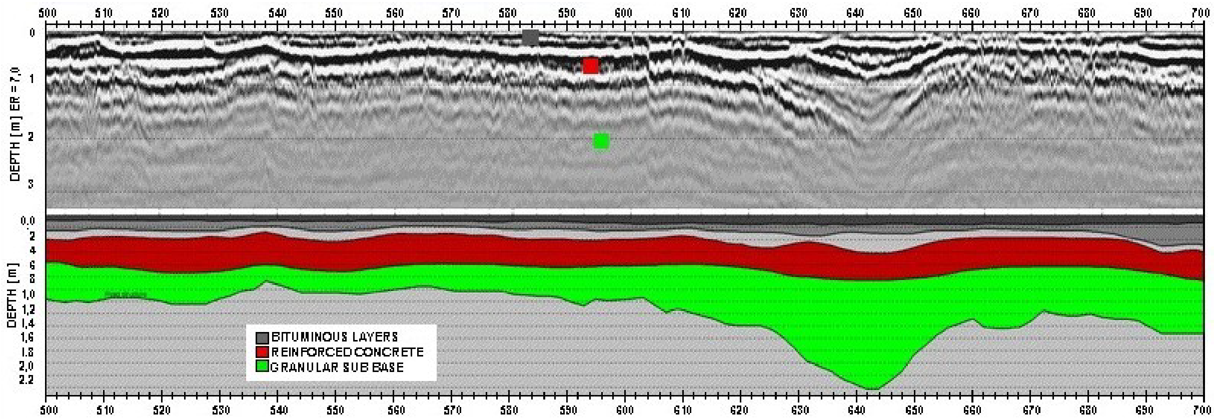
Sources: W. Watt, Subsea 7, Wester, Caithness, 2004 and Ove Arup & Partners Scotland Ltd ‘embankment remediation’ drawings,2000

Case Study Sc7	B876 Killimster Moss rehabilitation, Caithness, Scotland					Date	2000
AADT	1500	**Heavy vehicles**	7%	**Speed limit**	100 km/h	**Carriageway width**	7.0m

The B876 public road across the Killimster bog was constructed as a 150mm thick reinforced concrete slab around 1930 and had been widened and surfaced with bituminous materials at least 3 times since then. By the late 1990's the existing road was suffering under the effects of modern heavy traffic particularly those heavy transports hauling waste to a nearby Council landfill site. As a result the road was exhibiting increasing distress with longitudinal and transverse cracking. Carriageway depressions were up to 60mm deep and potholes widespread. The whole road was reported to shake when heavy vehicles passed. Funding for a replacement road was not available and a decision was taken locally to use the Nordic co-operations of the Roadex project to identify a suitable cost effective strengthening scheme.

Photograph of original construction in 1930

Ground penetrating radar investigations revealed that the underlying peat varied from 2m to 7m deep and that the severest settlement coinciding with the deepest peat. Bituminous macadam overlays varied from 100mm to 600mm deep. The peaty soil of the verge was found to be trapping water in the road construction both above and below the concrete and there was a saturated layer of sub-base about 150 mm thick beneath the concrete. Below the sub-base was a layer of blue clay which had presumably been laid as a capping layer to enable construction traffic to travel over the peat. A plot of the GPR output is shown below.



A modern 7m carriageway was required and the design opted for called for the removal of all of the existing bituminous surfacing to reduce weight and expose the present condition of the concrete slab. The water in the construction layers was drained but the saturated sub-grade was left in place as it was felt that any drainage of this would cause shrinkage problems leading to further settlement. The final solution presented by Roadscanners Oy of Rovaniemi involved minor surface repairs to the concrete slab and sealing open joints, regulating the slab with bituminous macadam, laying a steel mesh of 7mm bars on a 150mm square grid and finishing with a 100mm thick layer of bituminous macadam. The mesh panels were specifically cut to the size of 7m x 2.4m, sufficient to span the full width and extend into the verge but small enough to fit on a standard vehicle for delivery.

70 year old concrete slab surface exposed

Preparation for carriageway widening

Steel mesh installation

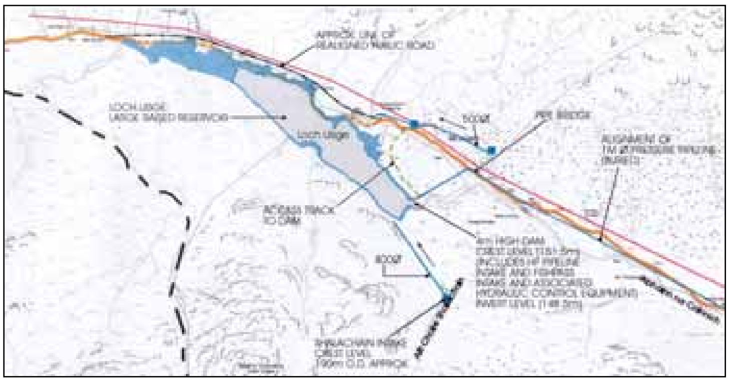
No measurable settlement has taken place since construction and monitoring continues.

Source: R Guest, Area Roads & Community Works Manager, TEC Services, The Highland Council

Case Study Sc8	B8043 Kingairloch Road Diversion, Lochaber, Scotland	Date	2003

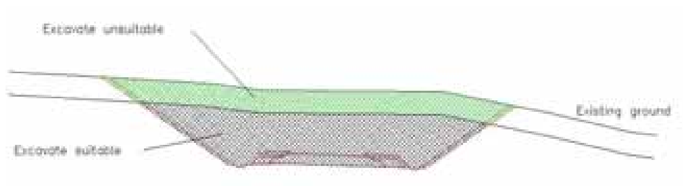
AADT	80	Heavy vehicles	11%	Speed limit	100 km/h	Carriageway width	3.5m

The Kingairloch Hydro Electric Scheme is located approximately 30km south west of Fort William in western Scotland. The project involves the construction of a new earth dam, intake works, a 3.3km buried pressurised pipeline, a 3.5MW power station together with all associated civil, mechanical and electrical infrastructure. The new dam when filled will result in the water level of the main reservoir of 'Loch Uisge' being raised by 3m, flooding an additional 10 hectares of land including a 750m section of the 3m wide single track B8043 public road leading to the village of Kingairloch. In preparation for this Scottish and Southern Energy plc, the promoters of the Scheme, commissioned The Highland Council to design a realignment of the affected 1.7km long section of the B8043 to keep it clear of the planned enlarged reservoir.

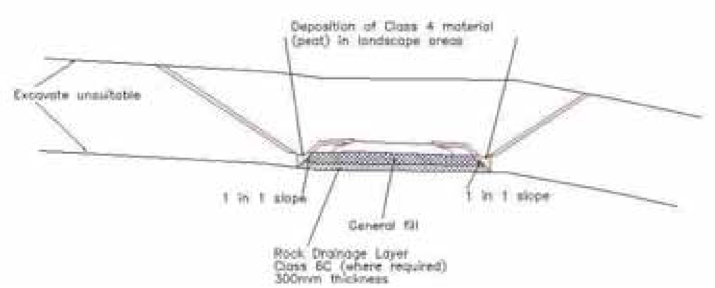


Kingairloch Diversion Layout

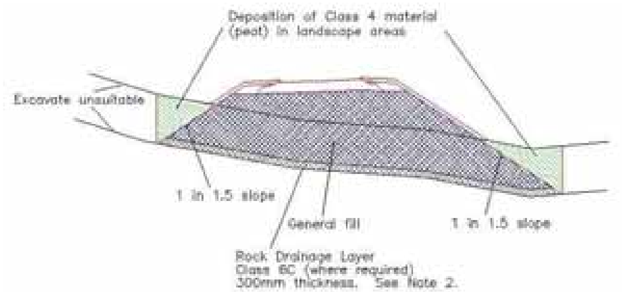
The route of the realigned road passes through an undulating peat and moraine landscape with peat depths of up to 4m in places. The design team opted to carry out the works using a standard peat excavation and replacement method and typical cross-sections for measurement are shown below:



Road in Cutting – Shallow Peat



Road in Cutting – Deep Peat



Road in Fill – Deep Peat

Photographs of road construction

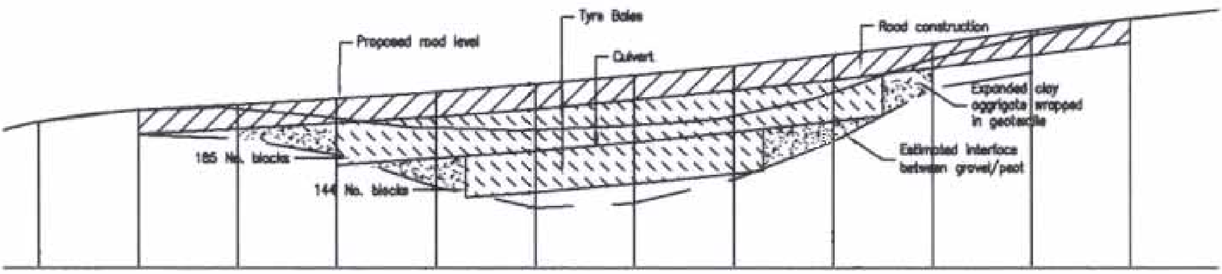
The works commenced on site in September 2003 with AMCO as main contractor and Torosay Sand Ltd as the subconctractor for the roadworks elements. The road was subsequently opened to traffic during November 2004. Reservoir impounding started immediately afterwards and at the date of writing in December 2004 the old road has been flooded.

Sources: Scottish & Southern Energy Plc, Foster Yeoman, AMCO

Case Study Sc9	B871 Loch Rosail tyre bale embankment, Sutherland, Scotland					Date	2003
AADT	55	Heavy vehicles	23%	Speed limit	100 km/h	Carriageway width	3.5m

The B871 road in central Sutherland is the main timber extraction route for the forests of Naver and Rimsdale. At Loch Rosail the existing road embankment was progressively settling into the underlying 6 metres of peat with consequent carriageway flooding. Various maintenance schemes had been tried to cure the problem by raising the embankment but this had only added weight to the structure and caused it to sink further. In 2003 the Sutherland area office of the Highland Council TEC Services proposed an innovative 'offloading' exercise whereby 55m of the existing heavy road materials spanning the peat hollow would be removed and replaced with a lightweight embankment of waste tyre bales at a higher level alignment above the bog. This was the first scheme to use waste tyre bales in the UK.

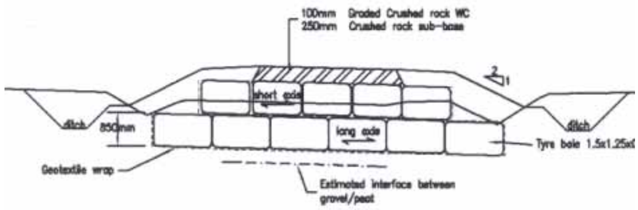
Flooding on existing road be3ore project



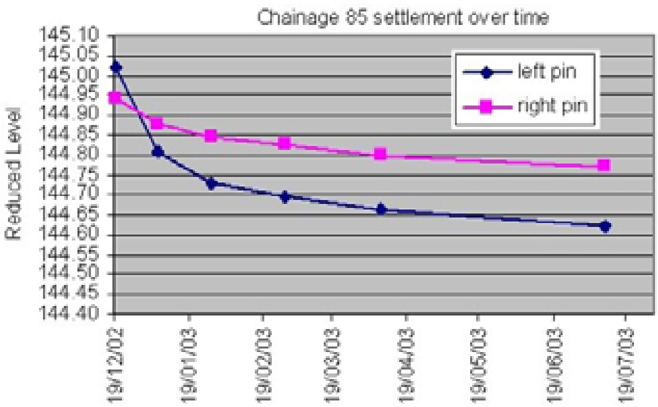
Long section through the tyre bale embankment

Bales were delivered to site as 1.2m x 1.2m x 0.85m blocks tied with 5 bands of high tensile wire and containing approximately 120 car tyres. This gave a bale density of around 0.6tons/m³ but for the purposes of design when submerged in the peat 'neutral buoyancy' was assumed.

The bales were encapsulated in a separator grade geotextile to prevent peat migrating into the voids. As an additional measure lightweight expanded clay aggregate was poured over and into the bales prior to closing the geotextile envelope to act as a further barrier to peat penetration. The embankment was finished with a 45cm rockfill layer dressed to level with a layer of BRC A252 welded reinforcing mesh at mid depth.



The finished gravel surface on the road was finally reshaped in mid July 2003 and given a double surface dressing treatment of 10mm chippings and Surfix 80 bitumen emulsion. Monitoring of settlement of the completed embankment commenced on 11 December 2002 and a graph of the cumulative settlement of the left hand and right hand pins at chainage 85 is shown below as an example.



Record of embankment settlement at 0+85

Installation of tyre bales in embankment

Source: G Smith, The Highland Council, TEC Services, Brora, Sutherland

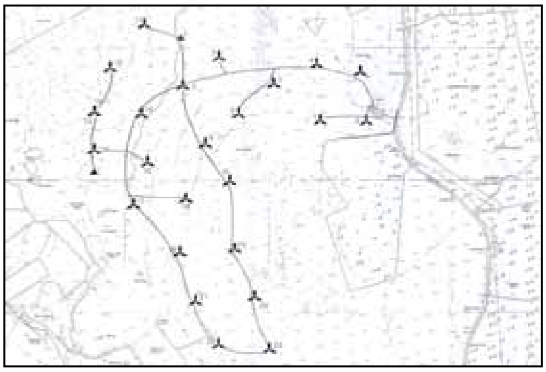
Case Study Sc10	Causewaymire Windfarm access roads, Caithness, Scotland	Date	2004

This project involved the construction of a network of access roads across the Dale Moss blanket bog for the Causewaymire Wind Farm of National Wind Power, one of the UK's most experienced wind farm developers. The main contractor for the works was Bonus Energy A/S of Denmark and their 'turn-key' contract required the provision of 21 pylons incorporating 2.3MW wind turbines accessed by a 10km network of 4m wide roads plus verges.

General photograph of the Causewaymire windfarm

Dale Moss is part of the 'Flow Country' an internationally important wildlife habitat and blanket bog conservation area. It is characterised by extensive peatlands with pools, lochans and sluggish burns and peat depths of up to 6m of varying moisture contents and strengths. Under the terms of the main site works contract the main civil engineering contractor, Edward Mackay Ltd, was required to design and provide all access roads and hardstandings to permit the wind turbines to be erected.

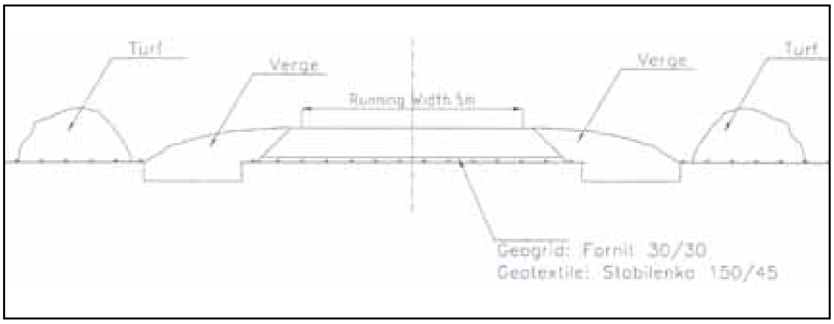
The access road design had to cater for all site construction traffic, pylon components delivery and the main erection cranes of 400t and 1200t. A full range of construction options were considered and eventually a decision was taken to proceed with a reinforcement rockfill embankment solution on top of the existing bog surface in recognition of the sensitive conservation nature of the site. Initial sections of road were constructed using a 0.6m to 2m deep embankment on a high strength 150/45 Hueskar 'Stabilenka' geotextile laid directly on top of the undisturbed peat mat but this arrangement was subsequently revised to a Hueskar 'Fornit' 30/30 polypropylene geogrid once trafficking with the 30 tonne site haulage dumper trucks (15 tonne axles) had been assessed. This final solution was empirical but based on previous experience by the roads contractor on earlier wind farm projects.



Wind farm access road network

For peat depth of 0 -1m

A single width of geogrid was laid on the bog surface and 60cm of rockfill carefully placed on the grid and onto the bog surface to give an overall width of 6m. This formation was then trafficked by the site dump trucks (15 tonne axles) to give an initial compaction following which a main compaction by D6 dozer and towable vibrating roller (Bomag BW6) was carried out. The prepared roadway was then given a wearing coat of 50mm crushed rock aggregate imported from a local quarry to supply the final shape before final compaction.



For peat depth of 1m – 2m

A double width layer of the Fornit 30/30 was laid directly on top of the bog surface to give a wider reinforced platform and aid stability. Placing of the rockfill roadway and compaction was as peat depth 1-2m.

For peat depth of 2m – 4m

4 widths of Fornit 30/30 grid were laid directly on top of the bog surface to give a full width reinforced basal platform beneath the roadway and verges. Placing of rockfill and compaction was as previous descriptions.

Photograph of rockfill being placed

Photograph of rockfill roadway

The access roads were completed in March 2004 and the overall windfarm commissioned by July 2004. During the works 6,000 to 8,000 tonnes of rockfill were transported by 15 tonnes axle vehicles per week over the site road network. In all a total of 350,000 tonnes of rockfill.

Sources: National Wind Power, Bonus Energy A/S, Edward Mackay Ltd

10.5 CASE HISTORIES: SWEDEN

The following case histories are presented with the permission of the contributors:

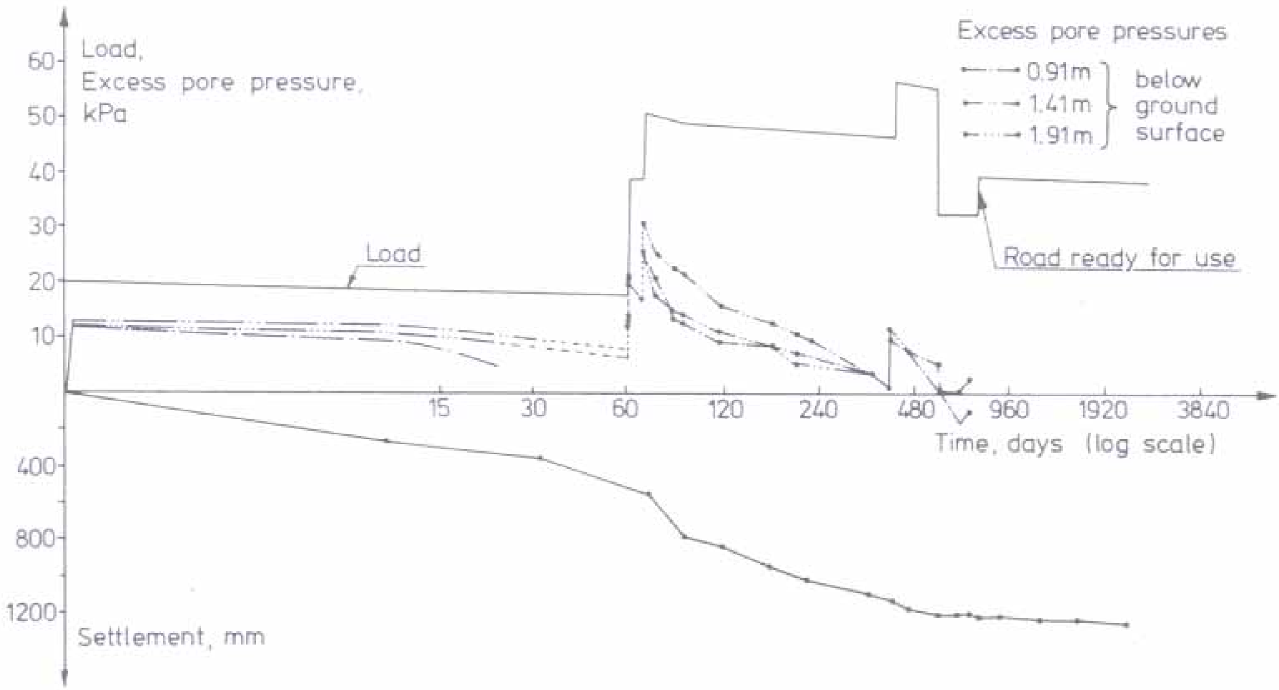
Sw1	The Dalarövägen road	1980
Sw2	Road No 820, Malmbäck	1984
Sw3	Road No 867, Bäck to Yaböke, Hallands Län	1989
Sw4	B65, Malmö to Ystad Road at Börringe Monastery	1991
Sw5	Road No 601, Sundsvägen at Råneå, Luleå	1995
Sw6	Road No 45, Akkavare – Autsjaur road	1998
Sw7	Road No 44 between Uddevalla and Trollhättan	2004

Figure 76. Road No 867, Bäck to Yaböke, Hallands Län.

Case Study Sw1	The Dalarövägen road, Sweden					Date	1980
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	24m

This project concerns an 850m long section over peat of the "Dalarövägen" road south of Stockholm. This road was constructed as a motorway in 1979-1981 with a crest width of 24m. The soil profile below the new embankment comprised a 2 to 3m thick layer of fibrous peat of Von Post H_2 to H_4 and water content 800-1300%. Under this layer was a very thin layer of gyttja and organic clay (0.1m) underlain by a sand layer 0.5 to 2.0m thick on top of compressible layers of clay and silt.

The new road embankment was constructed using a 2 stage preload operation incorporating a surcharge to accelerate settlement. The first layer of 1m of fill material was placed on the bog surface and the second stage fill of 1.5m including surcharge placed when the majority of the generated excess pore pressure of the first layer had dissipated (approx 50 days). After a year the surcharge was increased by a further 0.5m for 6 months at which time the overfill of approx 1m was removed. A porewater pressure and settlement history of the Dalarövägen works is shown below in illustration of the preloading process undertaken.



Measured settlement, excess pore pressures and load at Dalarövägen site (after Carlsten, 1988).

This history clearly shows that the use of surcharging can reduce post-construction settlement. At Dalarövägen the finished road embankment settled by 10 to 20mm during the four years after opening to traffic. The total settlement during the 19 months of construction was 1.2m of which 1.0m occurred in the peat layer and 0.2m in the clay layer. A very minor heave was observed when the surcharge was removed

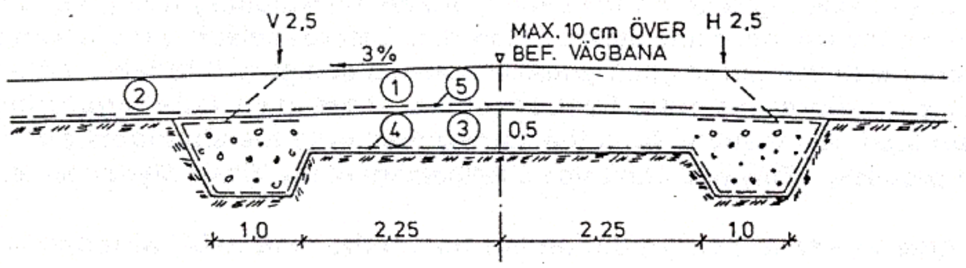
Source: P. Carlsten, Peat Geotechnical Properties and Up-to-Date Methods of Design and Construction, State –of-the-Art-Report Preprint, SGI Report 215, 1988

Case Study Sw2	Road No , Malmbäck, Sweden					Date	1984
AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.5m

This project involved a series of experimental trials of a range of lightweight materials in the repair of an existing gravel road over the "Lövhulta Mosse", Småland Province, south Sweden. It's aim was to examine the use of lightweight fill materials in 'offloading' exercises, ie the replacement of the existing heavy embankment material with lightweight material, and their resultant effects on the running surface of the road.

The method aimed to use the existing strength of the consolidated peat under the road and attempt to minimise future secondary settlements. Materials used were bark, woodchip, compressed baled peat, compressed baled straw, expanded polystyrene blocks and expanded clay aggregate.

A standard cross section was developed for the project to enable a comparative assessment to be made of the effectiveness of the different repair materials. This comprised a 6.5m wide 0.5m deep trench excavation locally deepened along the longitudinal edges by up to 1.0m to stiffen the repair. The prepared excavation was lined with a Class II geotextile ahead of the placing of the lightweight fill. Once placed and compacted the lightweight material was coved by a further layer Class II geotextile 8m wide to allow overlap onto the existing shoulders. The completed installation was capped with a suitable thickness of normal fill material and finished with 100mm maximum of new gravel road construction. (This common installation process did not always accord with the recommended practice for laying the respective materials.)



The results of the trials on the unsurfaced gravel road were as follows:

Expanded clay aggregate (LECA). This material was considered to be a good, if somewhat expensive, lightweight material for road maintenance repairs. Transport to site, placing and compaction of the material caused some problems but all were manageable. Minor rutting was noticed in the reformed gravel road surface but this was felt to be as a result of using a 40cm construction thickness rather than the recommended 60cm. Settlement along the test bed ranged from 10-80mm in the first year.

Expanded polystyrene blocks. This material was again considered to be a good, but expensive, lightweight material for road maintenance repairs. The standard Swedish Roads installation detail calls for a the blocks to be capped with a concrete slab and a minimum of 0.5m gravel but for the purposes of the trial the material was only capped with the gravel layer. The resultant road surface without the benefit of the normal concrete slab exhibited substantial deformation and damage during the first year after opening to traffic, approaching destruction in places.

Woodchip. This material was considered to be a reasonable material particularly if there was a local source at hand. It was easy to place, compact and traffic by construction plant. There were some questions regarding the effect of the woodchip on groundwater especially if the material had been reduced from timber that had been impregnated with preservatives. The settlement along and across the test length was relatively even at 320-370mm. The running surface did not exhibit cracking.

Chipped wood bark. This material was considered comparable to woodchip as a lightweight material. The age of the bark was important for leachate and environmental reasons. Fresh or old decayed bark was not considered suitable due to their potential for contamination of groundwater supplies. Like woodchip the settlement along and across the test length was relatively even at 370-400mm. The running surface did not exhibit cracking but rutting of up to 10mm was recorded.

Peat bales & straw bales. These experimental 50cm thick bales of peat and straw were trialled to determine their usability as lightweight materials in roadworks. The straw bales, like woodchip and bark fills, were considered to pose a possible leachate threat to groundwater. This was not thought to be a problem with peat bales. The section with peat bales showed some uneven settlement at the running surface along with some rutting but there was little pavement damage. The section with the straw bales, on shallower peat, showed heavy rutting and pavement damage.

The road was subsequently paved after the completion of the test exercise and is now being monitored for long-term settlement in the carriageway and any adverse effects on pavement condition. In 1990 the best material appeared to be either bark or baled peat. The Leca and EPS alternatives were considered to be too expensive for this secondary road and the straw option produced an unacceptable deformation in the carriageway probably due to its decay.

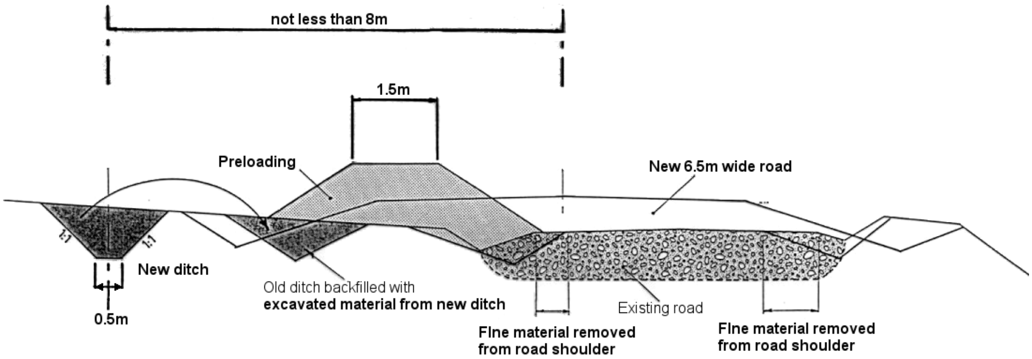
Source: P Carlsten, Vägbyggnad på torv, SGI Vägledning 2, 1989

Case Study Sw3	Road No 867, Bäck to Yaböke, Hallands Län, Sweden	Date	1988

AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	6.5m

Road No 867 from Bäck to Yaböke in Hallands County, southern Sweden was a 4.5m wide gravel road prior to widening in 1988. Part of the route crossed a 450m wide peat bog of up to 6m deep at Öxnalt and in preparation for the upgrading of the road a georadar survey was carried out to establish the condition of the existing road and identify suitable methods of construction. Longitudinal and transverse sections of the gravel road embankment and the underlying bog were made together with transverse sections across the road.

This survey established that the existing road had an overall construction depth of between 0.5 and 1.2m thick reflecting the varying depth of underlying bog and it was considered that the old road had over its lifetime become stable enough to permit its retention in the new works. Preloading (and surcharge) would be used to bring the adjacent bog up to a strength equal to the peat below the road and a new widened road would be constructed on the common embankment. It was calculated that preloading would be required for approximately 90 days.



The sequence of construction events on site was as follows:

1. A new intercepting ditch was excavated in the bog approximately 10m off the edge of the existing road on the side to be widened and the excavated material from it used to refill the existing roadside ditch.

2. The shoulder of the existing road on the side opposite the widening was graded to a depth of 200mm to remove the top poor fine surface materials before replacing them with a separating geotextile covered with 300mm of good granular material compacted to falls.

3. A similar grading exercise was then carried out on the side of the road to be widened. A 5m wide reinforcement grade geotextile was laid on the existing shoulder, side slope and across the adjacent bog surface in readiness for the preloading embankment.

4. Preloading operations were commenced from the existing road by means of a 360° excavator placing the first layer of 0.5m of fill on to the geotextile. Subsequent layers were carefully controlled through the use of marked settlement rods that enabled the direct measurement of the actual preloading embankment depth to be known as settlement developed. The preloading heights varied from 1.0m to 2.0m depending on the depth of underlying peat. The sections of higher preload were placed in 2 staged operations 14 days apart.

Placing of fill material

5. Settlement happened quickly. Up to 0.8m of settlement was recorded in the first few days of loading.

6. The preloading was left in place for its designed period of 90 days without effect on the continuing traffic flows on the adjacent road. Over this period settlements were monitored and found to be generally in accordance with the design expectations.

7. On completion of the exercise the excess preloading material was dozed from the side on to the existing road to act as an additional roadbase layer. The road has since been paved and is performing well.

Preloaded widening in place

Source: P Carlsten, Swedish Geotechnical Institute and L G Svensson, Swedish National Road Administration, 1990

Case Study Sw4	**B65 Malmö to Ystad Road at Börringe monastery, Sweden**					**Date**	**1991**
AADT	n/a	**Heavy vehicles**	n/a	**Speed limit**	n/a	**Carriageway width**	**6.5m**

This project involved the use of lightweight fill material in the repair of a 500m section of the B65 Malmö to Ystad Road at Börringe monastery. The B65 road was built in the late 1960's through an area of up to 8m of organic soils comprising peat, gyttja and gyttja-bearing clay using a standard 'excavation and replacement' method as shown in Fig 1 below. Unfortunately some organic material was left in place below the new embankment and over the years significant deformations occurred in the running surface of the road requiring regular repairs to the surface with base-course material and asphalt.

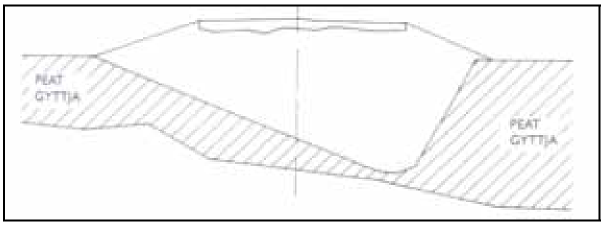


Fig 1 Börringe Monastery; Geotechnical conditions prior to treatment.

In 1991 a survey on the road found that an asphalt layer of approximately 1m had built up through these successive maintenance repairs and this had added to the instability of the embankment. In addition it was observed that a 30m longitudinal crack had developed in the road. A permanent solution was necessary to avoid the recurring maintenance works and an 'unloading' exercise was proposed to reduce the load on the road embankment (see Fig 2).

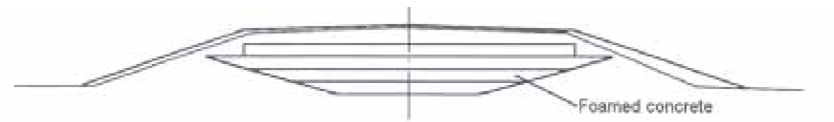


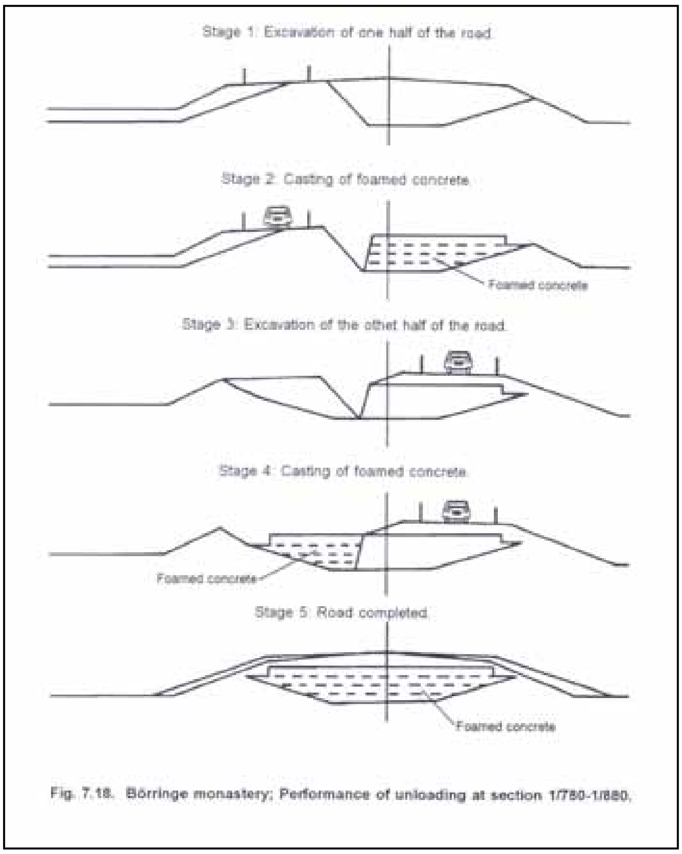
Fig. 2, Börringe Monastery; unloading cross-section.

The unloading works on site were performed using foamed concrete as the lightweight replacement material assuming an insitu density of 0.6t/m³ for the concrete after allowing for the absorption of water. The project was designed using stability analyses of the existing and proposed embankment structures with a requirement that the embankment factor of safety should be increased from the existing 1.2 up to 1.5. The unloading exercise was executed in 2 phases as described below.

Phase 1: The existing traffic flow was directed on to the left carriageway and excavation with stable sides commenced in the vacant right hand side. Lightweight foamed concrete was cast in four successive layers and after curing the completed installation was covered by base course material and asphalt.

Phase 2: Traffic flows were then diverted on to the 'unloaded' side of the road and excavation of the left hand carriageway commenced. Once prepared the foamed concreting process was repeated as in the completed carriageway and the final installation capped with a base course material and asphalt.

The whole construction from unloading, casting of foamed concrete and preparation of the road surface for the 500m section of the road was finished in 3 weeks.

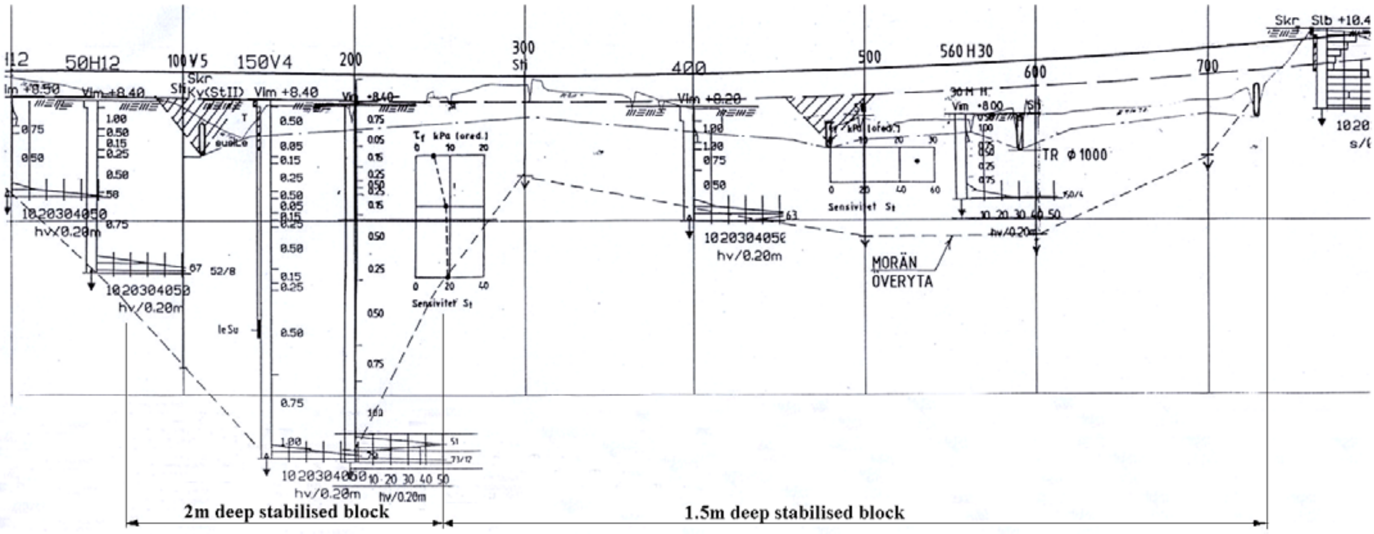


Börringe Monastery; Sequence of unloading by stages

Source: Carlsten, P., (1995) "Construction methods for roads in peatland areas", Bulletin 11, Danish Geotechnical Society, Lungby

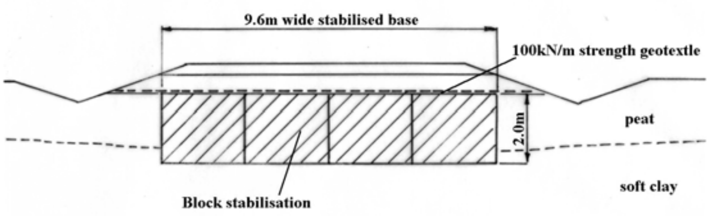
Case Study Sw5	Road No 601 Sundsvägen at Råneå, Luleå, Sweden							Date	1995
AADT	1470	Heavy vehicles	8%	Speed limit	70 km/h	Carriageway width	6.0m		

Road No 601 Sundsvägen at Råneå may be the first public road to be constructed over 'block stabilised' peat and used a Finnish design developed by Viatek Oy. The project was carried out in 1995 on a peat with an undrained shear strength of 7 kPa during a 5km general road rehabilitation project of road 601 of which 660m involved block stabilisation of peat. The stabilising agent used was 'Lohjamix', a proprietary blend of 40% Portland cement and pulverised steel industry by-products of blast furnace slag and fly ash.



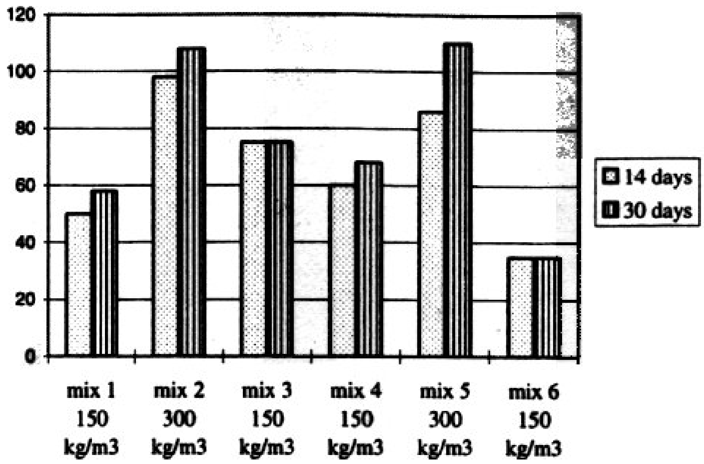
Long section along 660m treated by 'block stabilisation'

Under the stabilisation works The 'Lohjamix' was used at a rate of 150 kg per cubic metre of peat in a 2m deep 'block stabilisation' process between 0/070 and 0/250 and 1.5m deep blocks between 0/250 and 0/730. In all some 10,000 m³ of peat was treated in this fashion. During the works it was found that stabilisation depths could be reduced without adverse results and this meant depths as low as 1.0m could be adopted in some locations. The effect of this process was to produce blocks of peat that varied in strengths from 40 kPa to 200 kPa when tested by a cone penetration rig after 30 days, well above the design requirement of 50 kPa.



Follow up geotechnical and environmental investigations of the completed structures in 1998 reported that the stabilised material satisfied the requirements of the Swedish National Road Administration and that the results of the investigations indicated that block-stabilisation and the stabilising agent comprised an appropriate foundation method." (Source: J Mácsik, K Pousette & A Jacobsson, Luleå University of Technology, H Rosén, AB Jacobson & Widmark, O Seger, Swedish National Road Administartion)

Photograph of mass stabilisation works



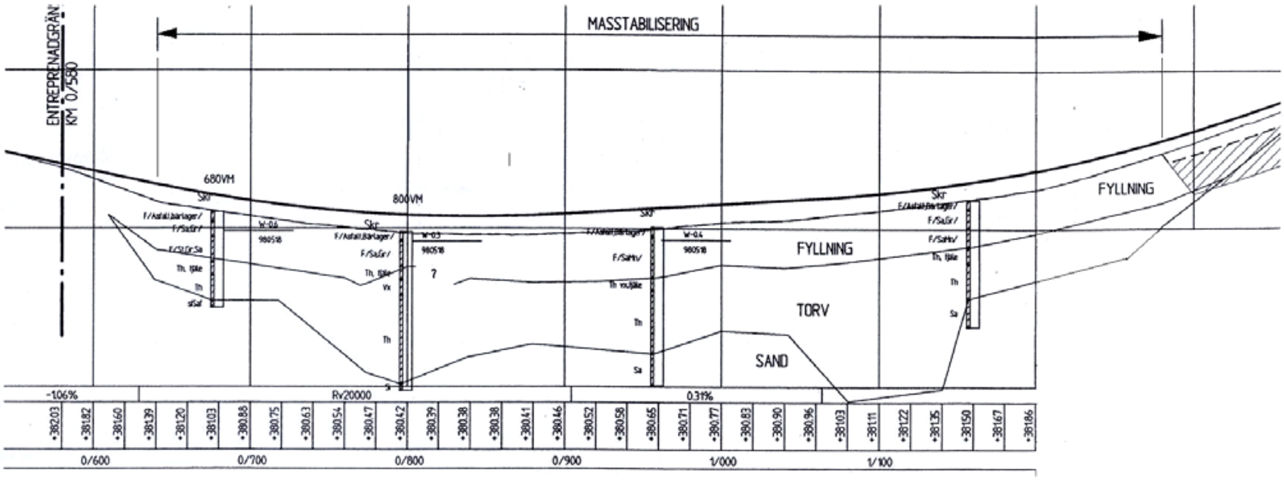
Comparison of stabilising agent mixes

Source: H Rosén, AB Jacobson & Widmark, Luleå 1998

Case Study Sw6	Road No 45, Akkavare – Autsjaur road, Sweden	Date	1998

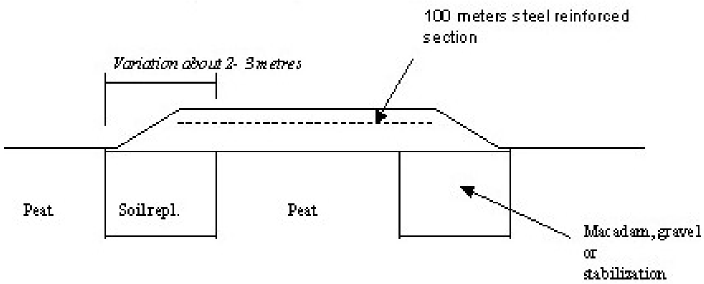
AADT	470	Heavy vehicles	8.5%	Speed limit	90 km/h	Carriageway width	8.0m

This 1988 scheme involved the widening and strengthening of a road sitting on peat. Road 45 had been originally constructed in 1975 on 2-3m of peat using a 'floating' gravel embankment with an oil gravel pavement. In 1988 the carriageway was scheduled to be widened to 8.0m and within this contract 650m (0/630 to 1/280) was to be widened by 'mass stabilisation' (or soil replacement) below the shoulders as follows.



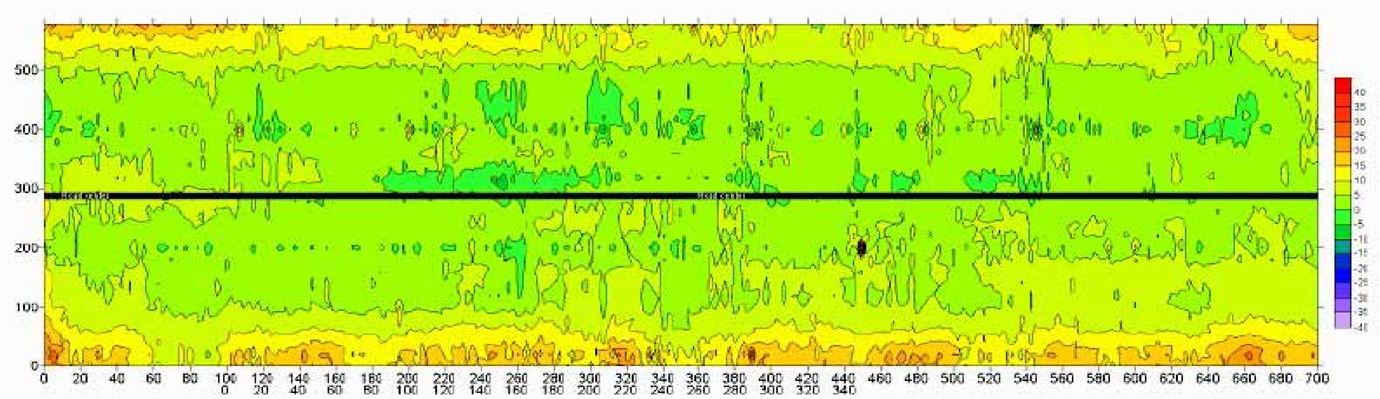
Long section along peat bog

The peat below the road shoulders at the edges of the existing embankment was excavated out over a 2-3m wide strip to a 'hard bottom' and the resulting void filled with either macadam, gravel or stabilised peat. (Replacement of the peat down to a sound layer has been carried out where the peat is less than 5m deep but if greater the replacement is normally confined to the top 3m of the bog) On conclusion of the 'mass stabilisation' process the widened embankment was placed on the widened prepared area as normal. Standard pavement layers were then laid with a steel grid reinforcement in the unbound base course at a depth of 150 to 250mm.



Schematic arrangement of widening

The road was surveyed in 2000 as part of the Roadex project with the following evaluation: "*The survey data collected from Road 45 in Akkavare proves that this special structure is working well. The rutting values were 7.5mm in the right lane and 6.1mm in the left lane which translates into approximately 2.5mm/10 000 trucks during the first two years. These higher rutting values can be expected from this type of special structure during the initial years. Slightly higher rut values were measured …. Where the road was not constructed over peat. The surface contour map (see below) shows that … in the peat area … the road has maintained its shape quite well.*"



Road surface map

Fuller details of this project together with ground penetrating radar data and evaluation are contained in the 2001 Roadex CD ROM

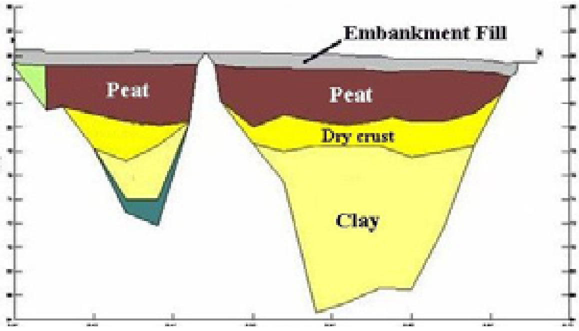
Sources: Väg 45 Akkavare- Auktsjaur Contract Drawings, Kjessler Et Mannerstråle, Luleå 1998 and T Saarenketo, Roadex project 2001

Case Study Sw7	Road No 44 between Uddevalla and Trollhättan, Sweden	Date	2004

AADT	n/a	Heavy vehicles	n/a	Speed limit	n/a	Carriageway width	21.0m

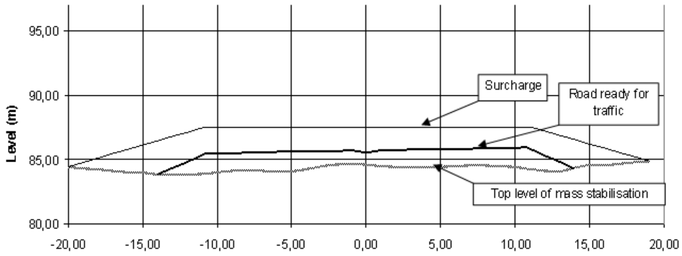
This project concerned the construction of a new 280m long section of the 21m wide, 4 lane, Road No 44 across the ”Bräcke mosse between Uddevalla and Trollhättan in southern Sweden by mass stabilisation. The geotechnical conditions across the peatland comprised up to 5 m of a ”normally” consolidated, highly compressible peat with a low degree of decomposition, water content 1200-2000%, over a 1m dry crust on top of up to 17m of clay, over consolidated by 40-60 kPa as shown on the profile adjacent.

The 21m crest width of the finished embankment required a stabilised width of 27m on the peatland and this was sub-divided transversely across the section into 5 parcels of 4m x 5.4m (27m = 5 x 5.4m) for ease of working. The stabilisation of each parcel in the section took approx 2 hours to carry out using 200kg/m³ of rapid hardening SH-Cement allowing the full cross section to be completed in a working day. A 70kN/m geotextile was then placed on top of the newly stabilised area and settlement plates and hose gauges installed to monitor the settlement of the new embankment construction. An initial 1m layer of embankment fill was placed on the geotextile in 2 stages, 0.6m and 0.4m, to provide a working platform for the stabilisation of the next tranverse section and the stabilisation works repeated as above.

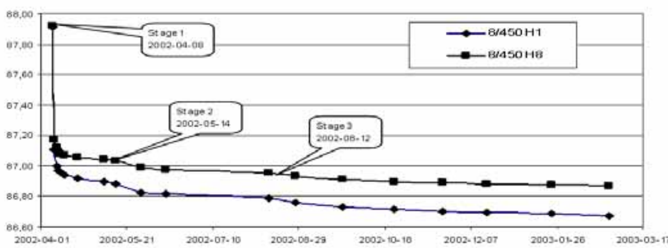


Profile along road alignment (brown colour indicates stabilised peat

The aim of the above works was to produce a characteristic shear strength of 50 kPa in the stabilised peat. On completion of the treatment (the works did not extend into the clay layers) 3m of surcharge was applied to the area as preloading in 3 stages (1.0+1.0+1.0m) over 30+30+120 days and after approx 6 months 1.0-1.5m of this was unloaded leaving the finished road level approx 1 m above the original bog level. Measured settlement on site was approx 10-15% of the peat depth compared to 25-35% in the laboratory. This is explained by field drainage paths being much longer than those in laboratory samples. This slower drainage in the field allowed water to be taken up in the concrete curing process and so lessened settlement through dissipation. This should be considered in design of mass stabilisation schemes.



Cross-section across new road embankment



Time-settlement curves for staged loading process

As part of the monitoring of the project a portion of the stabilised peat was exposed and an uneven surface of hills and valleys at approx 4m centres was discovered. This feature had no practical effect on the performance of the future road as the stabilised peat was within its design strength throughout. It was considered that a shear wave had been induced when the fill had been pushed out over the new 4m section and to minimize this effect any fill should placed carefully by an excavator taking particular attention to the joints between adjacent stabilised areas.

Photograph of exposed surface of the stabilised peat showing ‘hills and valleys’

In all 32,000m³ of peat was stabilised under the contract using with 6400 tonnes of SH-cement. The finished road was opened to traffic in June 2004. Mass stabilisation option was considered to offer good economy in this project. It was efficient in the use of fill material, did not require disposal of excavated peat and had limited effects on the surrounding peat bog and ground water.

Sources: Carlsten, P, Olsson, M,2004, “Masstabilisering av torv på riksväg 44”, Nordic Geotechnical Meeting, 14, Ystad, Proceedings, vol. 1

11 Discussion and summary

11.1 General

This report presents the results of an investigation into methods of road construction across peatlands in the Northern Periphery. The report describes problems that can arise in carrying works over peat, records details of solutions used and offers a ‘snapshot in time’ of practices in the Partner area for dealing with bearing capacity problems on roads over peat.

The formation and geomorphology of peat is discussed and how these can influence peat characteristics and properties. Ground investigation methods and laboratory testing of samples are described and summary tables presented showing their use in the Partner areas. Geotechnical design for embankments over peat is considered together with the risk management issues associated with constructions over peatlands.

20 techniques for road construction are described and a table of their distribution and usage in the Partner areas is presented (Table 11.1). These techniques are further classified generically into 3 groups of ‘accepted practice’, ‘developing technique’ and ‘out of use method’ (Table 11.2). 39 case histories of road construction projects within the Partner areas are offered as examples of working techniques in practice together with references for further contact.

Table 32. Summary of methods reviewed during Project.

Method	Derivative	Accepted Practice	Developing Technique	Out of use Method
Peat excavation		✓		
Peat replacement		✓		
Peat displacement	Progressive displacement	✓		
	Partial displacement	✓		
	Assisted displacement	✓		✓
Peat left in place	Preloading	✓		
	Surcharging	✓		
	Stage construction	✓		
	Profile lowering	✓		
	Pressure berms	✓		
	Slope reduction	✓		
	Lightweight fill	✓		
	Offloading	✓		
	Geosynthetics		✓	
	Timber raft			✓
	Concrete raft			✓
Vertical drainage			✓	
Piling		✓		
Mass Stabilisation			✓	

Table 33. Summary of construction techniques across the partner road districts.

Technique	Norway	Finland	Sweden	Scotland
8.3 Peat excavation	Used regularly	Used regularly	Used regularly	Used regularly
8.4 Peat replacement	Used regularly	Used regularly	Used regularly	Used regularly
8.5.1 Progressive displacement	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.5.2 Partial excavation	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.5.3 Assisted displacement	Used Occasionally	Not used	Used in the past	Used in the past
8.6.1.1 Preloading	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.1.2 Surcharge	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.1.3 Stage construction	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.2.2 Pressure berms	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.2.3 Slope reduction	Used regularly	Used Occasionally	Used Occasionally	Used Occasionally
8.6.2.4 Lightweight Fill	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.2.5 Offloading	Used Occasionally	Used Occasionally	Used Occasionally	Used Occasionally
8.6.3.1 Geotextiles & Geogrids	Used regularly	Used regularly	Used Occasionally	Used Occasionally
8.6.4 Vertical Drainage	Not used	Not used	Used Occasionally	Not used
8.6.2 Timber rafting	Used Occasionally	Used Occasionally	Used Occasionally	Used in the past
8.6.5 Piling	Used regularly	Used regularly	Used Occasionally	Used Occasionally
8.6.6 Mass stabilisation	Not used	Used Occasionally	Used Occasionally	Not used

The report concludes with a ‘Table of Improvement Methods’ (Table 33) that summarises the advantages, disadvantages, risks and relative costs of the different methods of construction discussed within the report.

11.2 ROADEX partner areas

Within the 4 Roadex partner districts the following statements can be made:

- 'Excavation' and 'Replacement' are considered to be the most reliable of the methods available today particularly for major roads;
- 'Displacement' and 'Partial Excavation' continue to be used where appropriate but their use is declining as newer methods prove more economic;
- 'Peat left in place' techniques are used where Excavation and Replacement are considered too expensive and Displacement is considered impracticable;
- 'Preloading' is the accepted technique for the improvement of bearing capacity of peaty subgrades and is normally carried out by Stage Construction to allow time for the subsoil to gain strength before the next layer is placed. Stage layers are generally 0.5m thick;
- Surcharge is reckoned to be the simplest and most cost-effective method for accelerating consolidation in peat once the embankment has reached designed height. Typical surcharge amounts for peaty soils range from 0.1 to 0.2 times the height of embankment;
- Vertical Drainage is not generally used on peat unless the deposit is seen to be contaminated or layered with less permeable soils such that it would benefit from the reduction in drainage paths;
- Stability of embankments is occasionally enhanced by widening the embankment base by means of berms or slope reduction to produce a more deep seated potential slip surface;
- Piling is considered to be too expensive for use as an everyday engineering solution for peat and is only used where settlement control is considered to be critical, eg on the approaches to bridge abutments;
- Geosynthetic applications in road construction over peat continue to increase in the Partner areas especially in the maintenance and improvement of existing roads over peat. The special case of high strength geosynthetic reinforcement in basal embankment reinforcement is considered to have too many inherent risks and is not recommended for national route construction;
- Offloading is considered to be a useful maintenance technique where a minimum of 50% of the existing load of the road can be removed, ie where a final load/unload ratio of 2:1 can be assured;
- Lightweight fills are being increasingly used to reduce problems of embankment instability and settlement. The technique is thought to be at its best when used in conjunction with a heavyweight surcharge or in an offloading scheme where the removal of the heavier material can be expected to have a proportionately greater effect on the lightweight material below.

All of the district offices visited had a common philosophy of applying low risk, standard techniques, such as Peat Excavation, Replacement and Displacement, to the construction of main national routes and restrict the use of the less developed and more innovative techniques such as soil improvement, geosynthetics, etc, to the lower classes of regional and district roads. The proven 'left in place' techniques such as preloading were only considered acceptable where there was sufficient time and flexibility in the construction period to allow the technique to produce the required improvement in strength.

Most engineers approached during the course of the project were aware of the range of techniques available for the construction of roads over peat but most had their own preferences of 2 or 3 alternatives that they tended to use regularly. All of those questioned however indicated that they would be prepared to use the more innovative techniques where they could be shown to be appropriate or cost effective for their particular sites.

The road maintenance schemes visited during the project were usually tackled empirically without the assistance of a detailed ground investigation. The 'offloading' technique was an exception to this and required a basic geotechnical input.

The choice of technique for a particular location was generally determined through a combination of the cost influencing factors:

- the complexity of the particular engineering works;
- the amount of soils investigation and testing necessary for each method;
- the required time for execution of the method;
- the type of budgetary control in force, eg rate of return, number of financial years, etc;.
- the amount of traffic disruption and additional traffic control required by the works;
- the expected future maintenance liability.

It was only after all of these construction and maintenance effects were examined that the most cost effective solutions emerged and final choice was made.

11.3 Minimum recommended practices for low volume roads

It is recommended that the following practices be considered as a minimum in low volume road construction and maintenance over peat

11.2.1 Ground investigations & laboratory tests

- A desk study, especially of records of similar works constructed locally in the past (Section 5.2);
- A site visit and 'walkover' to obtain a clear picture of the surface features of the peatland (Section 5.2);
- A probing exercise to establish peat depths and any layering This can be followed up by physical exploration measures if considered necessary that are suitable for the particular Works, eg trial pits, Swedish sampler, DCP, GPR, etc (Section 5.2.1);
- Peat classification and degree of humification (Section 3.3);
- Water content (Section 3.4.2).

11.2.2 Risk Management

- *A simple risk register (Section 7.5).*

11.2.3 Monitoring

- Make and retain site records. This is a direct plea from the author of this report. Many innovative projects are being carried out on sites without records being made. Any record is useful to future engineers, structured records are better. (Section 9.1);
- Use of settlement plates and/or hose gauges (9.2.1).

References:

Amaryan, L.S., Sorokin, G.V. & Ostroumova, L.V., (1973), "Consolidation laws and mechanical-structural properties of peaty soils", International conference on SMFE, Moscow.

Bell, F.G., (1978), "Roads on Peat", Civil Engineering magazine, January 1978

British Standards Institution, (1981), "Code of practice for earthworks", BS 6031:1981, BSI, 139 Chiswick High Road, London, W4 4AL

British Standards Institution, (1999), "Code of practice for site Investigations", BS 5930:1999, BSI, 139 Chiswick High Road, London, W4 4AL

Brouwer, J.J.M., (2002), "Guide to cone penetration", Lankelma, website www.conepenetration.com

Bruce, R., "The Great North Road over the Grampians", Paper 4812, The Institution of Civil Engineers, London

Carlsten, P., (1989), "Vägbyggnad på Torv", SGI Vägledning 2

Carlsten, P., (1988) "Peat Geotechnical Properties and Up-to-Date Methods of Design and Construction", State-of-the-Art-Report Preprint, Statens geotekniska institute Report 215

Carlsten, P., (1995) "Construction methods for roads in peatland areas", Bulletin 11, Danish Geotechnical Society, Lungby

Carlsten, P, & Olsson, M., 2004, "Masstabilisering av torv på riksväg 44", Nordic Geotechnical Meeting, 14, Ystad, Proceedings, vol. 1

Centre for Civil Engineering Research and Codes, (2001), "Guideline road construction over pear and organic soils", Report 2001-6, The Ministry of Settlements and Regional Infrastructure of The Republic of Indonesia in co-operation with The Ministry of Transport, Public Works and Water Management of The Kingdom of The Netherlands

FLAC 3D, ITASCA Consulting Group, Inc., 708 South Third Street, Suite 310, Minneapolis, Minnesota, 55415, USA, website www.itascacg.com

Hanna, T.H., (1985), "Field Instrumentation in Geotechnical Engineering", Trans Tech Publications

Hartlén, J. and Wolski, W. (eds), (1996), "Embankments on Organic Soils", Elsevier Science Ltd , New York

Hobbs, N.B., (1986), "Mire morphology and the properties and behaviour of some British and foreign peats", Quaterly Journal of Engineering Geology, London.

Irish Environment and Heritage Service, (2004), "Peatlands"website, www.peatlandsni.gov.uk.

Janbu, N., (1965), "Consolidation of clay layers based on non-linear stress strain", Proceedings 6th International Conference of Soil Mechanics and Foundation Engineering, Montreal

Jelisc, N., (2002), "Mass Stabilization, Stability and Settlement in Mass Stabilized Peat", Doctoral Thesis, Lund University, Lund Institute of Technology, Department of Soil Mechanics and Foundation Engineering.

Landva, A.O., (1980), “Vane testing in peat”, Canadian Geotechnical Journal.

MacFarlane, I.C., (ed.), (1969), “The Muskeg Engineering Handbook”, National Research Council of Canada

North Ostrobothnia Regional Centre, “The World of Plants, Aapa Mires”,

Oasys Limited, 13 Fitzroy Street, London, W1P 6BQ, website www.oasys-software.com

Office of Minnesota Road Research, Mn/DOT , “User Guide to the Dynamic Cone Penetrometer”.

Organisation for Economic Co-Operation and Development, Road Research Group, (1979), “Construction of roads on compressible soils”, Paris.

O’Riordan, N.J., and Seaman, J.W., (1994), “Highway embankments over soft compressible alluvial deposits”, TRL Contractor Report 341

PLAXIS BV, BTC II building, Delftechpark 19, 2628 XJ DELFT, website www.plaxis.com

Radforth, N.W., (1969), “The Muskeg Engineering Handbook”, National Research Council of Canada

Raeymaekers, G., (1998), “Conserving mires in the European Union”, LIFE-Nature Project

Raftery, B., (1990), “Trackways Through Time Archaeological Investigations on Irish Bog Roads 1985-1989”, Headline Publishing, Co. Dublin.

Rautaruuki Oy, “Geoprofiilien käyttö tierakenteen pohjanvakvistuksessa”

Roadex Project 1998-2001. Northern Periphery. CD-ROM

Roadscanners Oy, Rovaniemi, Finland, website www.roadscanners.com

SAGE-CRISP, The CRISP Consortium Limited, Amir Rahim, 2509 Robinhood Street, Houston, TX 77005, USA, website www.mycrisp.com

Skempton, A.W. and Bjerrum, L. (1957), "A Contribution to the Settlement Analysis of Foundations on Clay," Geotechnique, vol. 7, p. 168.

Scottish Development Department, Roads Directorate Technical Memorandum SH4/89 Amendment No1

Succow, M. & Jeschke, L, (1990), “Moore in der Landschaft, Entstehung, Haushalt, Lebewelt, Verbreitung, Nutzung und Erhaltung der Moore”, publisher Harri German, Graefstrasse 47, D-60486 Frankfurt/Main

Swedish Geotechnical Institute, SE-581 93 Linköping, Sweden

U.S.Army Corps of Engineers, (1992), Engineering and Design Guide, “Bearing Capacity of Soils”, Engineer Manual 1110-1-1905, 30 October 1992

Terzaghi, K. and Peck, R.B., (1967), “Soil Mechanics in Engineering Practice”, John Wiley & Sons, New York, NY

The Department of Environment, Transport and the Regions, (1998), “Eurocode 7 – A Commentary”, The Building Research Establishment and Arup

The Highways Agency, (2002), HD22/02 “Managing Geotechnical Risk”, Design Manual for Roads and Bridges

The Institution of Civil Engineers and The Department of Environment, Transport and the Regions, (2001), “Managing Geotechnical Risk, improving productivity in UK building and construction”

Transportation Research Board, Technical Activities Division, Design and Construction Group, Soil Mechanics Section, Transportation Earthworks Committee (AFS10)

Tressider, J.O., (1966), “A Review of Existing Methods of Road Construction over Peat”, Road Research Laboratory, Road Research Technical Paper No 40, HMSO

Vasander, H. (ed.), (1996), “Peatlands in Finland”, Finnish Peatland Society, Kuokkalantie 4, FIN-40420 JYSKA, Finland.

Von Post, L. & Granlund, E., (1926), ”Södra Sveriges torvtillgångar I”, (Peat resources in southern Sweden), Sveriges geoligiska undersökning

Appendix 1 – Table of Improvement Methods

Notes

The following notes may assist in reading the contents of Table 34.

Column	Comment
'Ref'	refers to the relevant section within the main Report
'Method'	gives a brief summary of the method being considered
'Advantages'	summarises the advantages of the method being considered as listed in the main section of the Report
'Disadvantages'	summarises the disadvantages of the method being considered as listed in the main section of the Report
'Risk'	gives an initial suggestion of the type of risk that could be considered within a Risk Register
'Cost rating'	gives an indication of the relative cost of the method being considered; '€' denotes a low cost, '€€€€'denotes a high cost

Table 34. Summary of improvement methods with costs.

Ref	Method	Advantages	Disadvantages	Risks	Cost rating	Comments
8.2	**Avoidance** (Realigning the route to go round the peatland on a sound foundation)	Avoids potential problems in dealing with peat and other soft soils. Should result in better long term road characteristics	Requires alignment revision. Possible reduction in alignment quality.	None, other than normal construction risks.	€ - €€	
8.3 8.4	**Peat excavation** (The removal of peat on the line of the route) and **Peat replacement** (The removal of peat and its replacement with non cohesive material.)	Proven technology. Should achieve a good bearing capacity using a standard embankment construction on a sound layer. Limited consolidation and settlement over the lifetime of the road. No additional time required for surcharge effects.	Significant quantities of excavated materials created. Land required for formation of sideslopes in peat and disposal of excavated materials. Difficulties in excavation and placing fill below water table. Normally demands high quality of fill material (low percentage of fines). Deep excavations may have effects on adjacent lands and structures. Unexcavated soft material below embankment may cause future settlements.	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.	€€ - €€€	Method generally limited to depths of up to 8m of peat. Cost range reflects limited depths of excavation and filling.
8.5.1 8.5.2	**Progressive displacement** (The displacement of peat from below an embankment using the weight of the embankment fill supplemented by a surcharge.) and **Partial excavation** (The displacement of peat from below an embankment by the weight of the embankment fill and surcharge aided by excavating out material ahead of the embankment.)	Well tried intermediate technologies. Should achieve a good bearing capacity on the displaced embankment construction. The displaced peat to the sides of the embankment can enhance the embankment stability. Good methods for constructing a high embankment above a peatland.	Better suited to amorphous peats. Fibrous peats may prove resistant to shear failure without assistance. Requires substantial quantities of fill material for the buried embankment. Requires longer construction time for displacement and surcharge affects to be effective. Normally demands high quality of fill material (low percentage of fines). Some limited consolidation and differential settlement can be expected over the lifetime of the road if peat pockets remain trapped below the embankment. The peat displaced during the procedure can cause heave effects on adjacent land and structures. Wide embankments may require significant materials to be displaced. Possible problem with culvert locations.	Excavation in peatland. Effect on adjacent structures. Possible trapped peat below embankment.	€ - €€€	May need proving cores to check for the presence of trapped peat below the embankment. Can be combined with surcharging for 3-6 months to limit the problem with future settlements.

Ref	Method	Advantages	Disadvantages	Risks	Cost rating	Comments
8.5.3	**Assisted displacement** (The displacement of peat from below the embankment by the weight of the embankment fill and surcharge assisted by blasting or water jetting.)	Used with progressive displacement and partial excavation methods. Established intermediate technology. Does not require peat excavation. Should achieve a good bearing capacity on the displaced embankment construction.	As mentioned for progressive displacement and partial excavation methods. Use of explosives. Can only be used in clear open sites with no utilities, etc.	Use of explosives. Excavation in peatland. Peat displacement. Effects on adjacent structures. Possible trapped peat below embankment.	€ - €€€	May need proving cores to check for the presence of trapped peat below the embankment.
8.6.1.1	**Preloading** (The use of load ahead of the main works to improve the bearing capacity of the subgrade so can it can be capable of supporting the planned load)	Minimises embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil. .	Embankment filling rate is limited by soil strength increase due to consolidation. Time needed for preloading can extend project construction times. Preloading materials may need to be brought on to site earlier than required and require double handling. Comprehensive site investigation and laboratory testing needed to establish the consolidation characteristics and anticipated increases in soil strength during construction. Should have an onsite monitoring system for consolidation and settlement to ensure that the required settlements are being achieved. Is a 'floating' road method and is best suited to thin embankments.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€ - €€	Stockpiles of construction materials can be used as preloading surcharges rather than importing additional fill materials. Stage construction normally needed in the case of higher embankments
8.6.1.2	**Surcharging** (The use of a temporary additional load to accelerate the rate of settlement and consolidation.)	Improves the bearing capacity of the underlying peat so that it can support the weight of the in-service embankment. The times for primary consolidation and secondary compression of the underlying peat can be accelerated. Stage construction normally needed in the case of higher embankments	Time needed for surcharging can extend construction time. Surcharge materials may need to be brought on to site earlier than required and require double handling as a consequence. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€ - €€	Cost range indicates additional cost over main method. Stockpiles of construction materials can be used as preloading surcharges rather than importing additional fill materials.

Ref	Method	Advantages	Disadvantages	Risks	Cost rating	Comments
8.6.1.3	**Stage construction** (The construction of an embankment in layers with the placing of each layer being dependant on a strength increase in the previous layer)	Produces sequential gains of strength in the peat. Minimises future secondary compression settlement of the new embankment. Higher embankments can be constructed without shear failure in the underlying peat. Does not require peat excavation, disposal or the need for additional land for storage of spoil.	The time needed for the various stages to take effect can extend the embankment construction time. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved before the next layer is placed.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€ - €€	Cost range indicates additional cost over main method. Site monitoring system required for stages
8.6.2.1	**Profile lowering** (The lowering the intended embankment height to suit the strength of the underlying peat.)	Reduces the quantities of fill material required. Reduces embankment loadings on the underlying peat. Reduces the amount of land required.	Requires a modification of the designer's preferred alignment. May not be possible if bridge clearances or waterway areas are critical. May give problems with bearing capacity of road embankment	Loading of peatland. Bearing capacity	None, design only	
8.6.2.2	**Pressure berms** (The widening of the base of an embankment to increase the factor of safety against potential slip circle failure.)	Improves stability. Increases the depth and length of the critical slip circle. Low grade fill material (even peat) can be used as fill mass in berms.	Requires additional fill material and additional land for the wider construction. Increases the overall weight of the embankment. Consolidation settlements may be increased as a result of the spread of load from the pressure berm.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€	Cost range indicates additional cost over main method. (This could be very small if there is surplus material on site.)
8.6.2.3	**Slope reduction** (The flattening of sideslopes to widen an embankment to increase the factor of safety against potential slip circle failure.)	Improves stability. Increases the depth and length of the critical slip circle.	Requires additional fill material and additional land for the wider construction. Increases the overall weight of embankment. Consolidation settlements may be increased as a result of the spread of load from the wider slopes.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€	Cost range indicates additional cost over main method. (This could be very small if there is surplus material on site.)
8.6.2.4	**Lightweight fill** (The use of low density fill materials to reduce the overall weight of an embankment and lower the permanent stresses on the subgrade.)	Does not require as high a bearing capacity from the peat foundation Usually does not need the underlying peat to be strengthened. Lighter embankment construction generally means less future settlement.	Purchase price and transport of the specialised lightweight materials. Design and placing of lightweight materials may require special arrangements. Environmental considerations particularly with groundwater. Bearing capacity of the lightweight embankment may be limited	Placing lightweight material below ground water table. Bearing capacity.	€€ - €€€€	

Ref	Method	Advantages	Disadvantages	Risks	Cost rating	Comments
8.6.2.5	**Offloading** (The removal of existing heavyweight material from an existing road construction and its replacement with a low density fill materials to reduce the overall weight of the in-service embankment and lower the permanent stresses on the subgrade.)	Does not require as high a bearing capacity from the peat foundation Usually relies on the underlying peat to have generated a sufficient bearing capacity to support the planned in-service embankment. The reduced embankment weight generally means minimal future settlement. No additional time required for surcharge effects.	Purchase price and transport of the specialised lightweight materials. Design and placing of lightweight materials may require special arrangements. Environmental considerations particularly with groundwater. Bearing capacity of the lightweight embankment may be limited	Placing lightweight material below ground water table. Bearing capacity.	€€ - €€€	The higher end of the cost range shown reflects the cost of specialist lightweight fill material
8.6.3.1	**Geotextiles & geogrids** (The use of a geosynthetic layer within an embankment, usually at the interface with the peat subgrade.).	Limited site disturbance. Easy to install. Provides reinforcement effect to the base of embankment for the short and medium term. Aids stability. Can reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. No excavation, disposal or need for additional land for storage of spoil.	The overall settlement of the embankment is not reduced. The geotextile/greogrid can be damaged by construction equipment. Creep may affect the long term performance of the geotextile. Use of geogrid may need higher quality fill material (interlocking).	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€ - €€€€	The cost range shown indicates the variation in costs between a separator grade geotextile and a high strength geosynthetic
8.6.3.2	**Timber rafting** (The use of a brushwood or timber platform on the peatland surface to support and distribute the loads of the embankment)	Limited site disturbance. Relatively easy to install. Provides reinforcement effect to the base of embankment for the short and medium term. Aids stability. Can reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil.	The overall settlement of the embankment is not reduced. Can be damaged by construction equipment during placing of embankment fill. High element of manual labour required for fabrication of the raft. Timber raft must be submerged.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€€ - €€€€	

Ref	Method	Advantages	Disadvantages	Risks	Cost rating	Comments
8.6.3.3	**Concrete rafting** (The use of a concrete platform on the peatland surface to support and distribute the loads of the embankment)	Limited site disturbance. Provides long term stiff foundation for the embankment. Aids stability. Reduce differential settlements and lateral stresses on the peatland surface. Minimises need for embankment fill material. Does not require peat excavation, disposal or the need for additional land for storage of spoil.	Overall settlement of the embankment is not reduced. Curing time for concrete. High element of manual labour required for fabrication of the raft.	Loading of peatland. Bearing capacity. Effects on adjacent structures.	€€€€	
8.6.4	**Vertical drainage** (The use of additional drainage conduits to shorten drainage paths and cause an acceleration of the primary consolidation process.)	Reduction of time for primary consolidation and secondary compression to happen.	Acceleration of primary consolidation and secondary compression results in significant settlements during construction period. Performance of drains affected by buckling, heave, smear.	Loading of peatland. Bearing capacity. Effects on adjacent structures. Altering existing drainage paths.	€€€	Cost range indicates additional cost over main method.
8.6.5	**Piling** (The use of piling units to transmit the embankment load to a suitable load bearing layer.)	Does not require peat excavation, disposal or the need for additional land for storage of spoil. Limited site disturbance. Minimal settlement. No additional time required for surcharge effects .	Does not rely on strength of insitu peat. No support assumed from surrounding soil. Usually needs a continuous concrete slab or geotextile load transfer platform.	Piling operations. Vibration. Effects on adjacent peatland and structures. Design sophistication.	€€€ - €€€€€	
8.6.6	**Mass stabilisation** (The improvement of a soil by mixing n a designed binder in order to increase its bearing capacity.)	Does not require peat excavation, disposal or the need for additional land for storage of spoil. Reduces settlements and adds to bearing capacity of the peat. Smaller demand of fill material compared to other preloading techniques. Could be suitable for high standard roads with high demands on differential settlements and bearing capacity. Could be suitable when there is soft clay beneath the peat.	The time needed for preloading can extend construction time. Surcharge materials may need to be brought on to site earlier than required and require double handling as a consequence. Needs to have a system in place on site for monitoring of consolidation and settlement to ensure that the required settlements are being achieved.	Loading of peatland. Stabilisation operations. Bearing capacity. Effects on adjacent structures.	€€€ - €€€€€	

Roadex Publications

ROADEX II

- ROADEX II - Focusing on Low Volume Roads in the Northern Periphery DVD
- User Perspective to ROADEX II Test Areas' Road Network Service Level
- Permanent deformation
- New material treatment techniques
- Managing spring thaw weakening on low volume roads
- Socio-economic impacts of road conditions on low volume roads
- Dealing with bearing capacity problems on low volume roads constructed on peat
- Drainage on low traffic volume roads
- Environmental guidelines
- Environmental guidelines, pocket book
- Road management policies for low volume roads – some proposals
- Structural Innovations
- Monitoring, communication and information systems & tools for focusing actions

ROADEX I

- Roadex Multi-media CD-ROM
- Road Condition Management of Low Traffic Volume Roads in the Northern Periphery
- Winter Maintenance Practice in the Northern Periphery,
- Generation of 'Snow Smoke' behind Heavy Vehicles

Roadex II Lead Partner: The Highland Council, Transport, Environmental & Community Service, HQ, Glenurquhart Road, Inverness IV3 5NX Scotland.