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GEOTECHNICAL ASSET MANAGEMENT FOR UK RAILWAY EMBANKMENTS

This thesis is submitted to the University of Nottingham for the degree of

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by

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Abstract

The British railway system is one of the oldest in the world. Most railway embankments are aged around 150 years old and, the percentage of track disruption due to embankment failure is frequently higher than other types of railway infrastructure. Remarkable works have been done to understand embankment deterioration and develop asset modelling. Nevertheless, they do not represent a sufficient way of managing assets in detail.

One of the biggest challenges that geotechnical asset managers and railway operator face is the detection of embankment failure at an early stage. Unplanned disruptions compromise safety for passengers, reliability of railway operators and require emergency budget deployment. To guarantee good system performance and meet costumer's expectations, industries would benefit efficient and pro-active management activities and adoption of Geotechnical Asset Management (GAM) programs.

To support the challenge, this research improves the understanding of the interaction between causes of embankment instability and visible signs of embankment instability. In this thesis, the signs of embankment instability are identified thanks to the use of a new metric called Embankment Instability Metric EIM developed by AECOM in 2018. The EIM measures the worsening of track geometry that is likely due to embankment instability.

This research work presents the results of the analysis aiming to evaluate whether a link existed between track deterioration, due to embankment instability, and the geotechnical parameters known from literature as playing a role in the embankment disruption.

Results of this analysis proved that, based on the specific analysis undertaken, different levels of correlation between causes and symptoms can be assessed and that some parameters show a better link with the EIM than others.

The final outcome of this research work was the development of a decision-making tool based on a Multi-Criteria Decision-Making MCDM approach. The

novel tool supports the decision-makers in the process of selecting the most appropriate intervention to be undertaken for a specific embankment asset given its current geotechnical conditions.

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Lastly, a gigantic thank you to my grandparents. Every kid deserves a childhood like the one you donated me.

To my 23-year-old self

Declaration

I declare that the contents and the work described in this thesis were performed at the University of Nottingham, Faculty of Engineering from October 2017 to September 2020. I hereby certify that this thesis is my own and has not been submitted in whole or in part to any other university or any other educational association for a higher degree.

Giulia Siino

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CHAPTER 1. INTRODUCTION

1 Introduction

1.1 Background

The railway system represents an important part of the transportation infrastructure of a country. The performance of a railway track system results from a complex interaction of the system components, therefore, it is necessary to understand how each part of the track structure works and interacts with the rest.

In the past, the track superstructure, comprising the rails, fastening system and sleepers were the focus of attention of railway engineers. Less attention was given to the substructure even though it provides the foundation to support the superstructure and to help it reaching its quality performance. Embankments and cuttings form civil engineering structures known as earthworks. They are an important means of physically supporting the trafficked surface of the transport infrastructure maintaining the natural ground alignment. Railway earthworks require constant maintenance and the need to undertake it has become increasingly apparent as the materials within these structures age, leading to instability, which in turn has implications from a financial, safety and reliability point of view.

The British railway system is one of the oldest in the world with most of earthwork structures in excess of 150 years old and therefore built without a detailed understanding of slope stability and soil or rock mechanics. There are several geotechnical hazards across the network that are still difficult to detect with available technology. Also, the way in which the infrastructure owner's data is structured rarely provides a prioritisation of risk. Nevertheless, earthwork failures will continue regardless, and it is simply not economically viable to reconstruct of all embankments to the levels of capability and resilience offered by modern engineered slopes. A solution needs to be found involving the concept of Geotechnical Asset Management (GAM).

The biggest challenge faced by railway operators and asset managers in the UK is the detection of geotechnical asset failure by means other than train drivers or lineside staff, desirably at a time when prevention of further deterioration and/or economical repair are still possible. Geotechnical failures are indeed often first reported by train drivers, for example through rough rides on embankments. This happens too late for preventative measures to be put in place. Stopping trains from finding failed earthworks that have rapidly lost the ability to perform is the current top geotechnical challenge. At present, there is no reliable method of prediction of when or where earthwork failures may occur.

If user’s expectations of safety, reliability and affordability are to be met, this can only happen if the transport infrastructure continues to function in the desired manner. Maintenance, repair and renewal are required to ensure the infrastructure’s continuing performance and, if this is to be done efficiently and pro-actively, these activities will require a management system.

The annual cost to construct and maintain a viable track system forms a significant element on companies’ financial statement. The funds optimisation is a challenge which demands novel techniques and cost-effective technologies to ease early failure detections and to consistently acquire and store information across the network. When the condition of an asset worsens until repair operation is necessary, the railway infrastructure operator will incur significant costs. The capital invested for the repair may indeed extend beyond the direct costs of employing designers and contractors and it often includes the provision of temporary access track, temporary speed restriction, line closures, re-routing users and reduced revenue. The cost of unplanned repairs is generally high and certainty higher than planned interventions.

1.2 The Research Project

The research on which this thesis is based was a collaborative research project between the University of Nottingham (Nottingham Transportation Engineering Centre) and AECOM (Nottingham offices, Rail Asset Management team). It aimed to determine the parameters playing a role in leading to geotechnical assets deterioration and how their symptoms may be detected before that deterioration impacts the train traffic. Moreover, during a meeting with Network Rail, it emerged how, from an operational point of view, it is more challenging to detect an ongoing failure within an embankment than a cutting. For this reason, the analysis in this research work is narrowed down to only embankment assets.

1.3 Aim and Process

Within the context described in Sections 1.1 and 1.2, the aim of the thesis is to explore how quantitative data sets can be collected and combined so that relevant embankment instability information can be abstracted and, thus, prioritised intervention can take place before failure.

In this regard, the specific objectives of this thesis are:

- To assess causes of distress (geotechnical features) from literature and then link these to detectable instability symptoms (track geometry displacement).
- To identify whether correlations between these data sets exist.
- To set critical values of the relevant factors and establish priority of intervention on the assets.
- To develop a Multi-Criteria Decision-Making tool for supporting decision-makers in the pro-active detection of embankment failure.

Track geometry data are available in the AECOM database, collected and analysed for the "Embankment Instability Modelling Research" project (which the author of this thesis did not participate) commissioned by Network Rail in 2018. The project had demonstrated that track geometry data is a viable data source to consider for detection of railway embankment instability and a new metric, referred to as the Embankment Instability Metric (EIM), was developed as an output. This metric is used in this thesis to correlate with parameters that potentially cause instability. The process will set critical values of the relevant parameters and establish priority of intervention on the assets. In this way the thesis attempts to find answers to the following research questions:

- Do the EIM and the parameters leading to embankment instability show correlation?
- How can these correlations improve the Geotechnical Asset Management?

The outcome of this thesis aims to bring an improved technical understanding of the interaction between all the parameters involved in the instability process and to then develop a multi-criteria intervention decision-making MCDM tool for railway embankment stability. The tool developed is based on the Analytic Hierarchy Process (AHP) approach chosen as best fitting the problem of several possible MCDM methods determined as available from literature review. It will be possible to input the current embankment asset condition (from monitoring data) into the tool so as to finally obtain a suggestion on the most suitable intervention for the site examined. Railway geotechnical infrastructure asset managers are then supported in their decision-making process and thus in the identification of the best action to undertake before actual failure occurs.

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CHAPTER 2. GEOTECHNICAL ASSET MANAGEMENT – AN OVERVIEW

2 Geotechnical Asset Management – An Overview

The economic prosperity of a country is greatly dependent on a safe and reliable transport network. People's expectations to travel for long distances with reliable journey times have increased remarkably over recent decades. Passenger research from Transport Focus [1] constantly shows how important for users is "an affordable, punctual, reliable, frequent service". Such expectations can only be met if the transport infrastructure continues to function in the desired manner. Maintenance, repair and renewal are required to ensure the infrastructure's continuing performance – and, if this is to be done efficiently and pro-actively, these activities will require a management system.

Due to the importance of geotechnical assets in the support and performance of the transportation infrastructure, organisations have been managing them for centuries. However, it is only recently that focus has been sharpened on Asset Management (AM) and, hence, on Geotechnical Asset Management (GAM) recognising the need to be carry out maintenance within a systematic framework, better if integrated into a risk-based Transportation Asset Management (TAM) program.

In this chapter, an overview of the current practice of GAM will be given with particular attention to UK practice.

2.1 Why implementing GAM?

At the simplest level, asset management is about managing physical objects with associated value. Geotechnical assets are the retaining walls, embankments, slopes and constructed subgrades within a transportation system right-of-way (ROW) or easement. Like other asset categories, geotechnical assets are features that are designed, constructed, and maintained by a transportation agency and their performance contributes to the continuous operation of a transportation network. Geotechnical assets are also subject to deterioration and exposed to natural hazards similar to other assets.

According to Vessely et al. [2] the benefits of GAM in the transportation field are real and measurable and are increasingly being recognised by both public and private infrastructure organizations. Performing GAM brings benefits that include:

- Financial savings across the geotechnical asset life cycle, with values reported to be greater than 30 percent by the U.S. Army Corps of Engineers [3] and 60 percent to 80 percent per unit length of embankment in the United Kingdom [4].
- A process to measure and manage involuntary safety risk exposure across the entire asset class.
- Reduced transportation delay and line closure times, resulting in improved network performance.
- Enhancement of data-driven decisions that support asset managers.
- An improved understanding of risk exposure levels and distribution, and the ability to manage those risks.

Public agencies receive public funds and have a fiduciary responsibility to be good managers of government-provided budget. Consequently, agencies establish policies and procedures to ensure that money is used effectively, waste is minimised, and investments can withstand the test of public scrutiny.

In 2014 during the "*GE's* Slope Engineering and Geotechnical Asset Management Conference in London" it was stated how a more proactive approach is needed for effective GAM [5].

A reactive approach to earthwork management has largely prevailed over a proactive approach up to now. This turned out to be inefficient and uneconomic; by adopting a GAM, organisations will better manage risks to passenger safety, mobility, and economic vitality, and will be able to make knowledge-based life-cycle investment decisions [6] .

The ISO 55000 standard for asset management [7] provides an overview of the subject of asset management. ISO 55000 notes that "asset management capabilities include processes, resources, competences and technologies to enable the effective and efficient development and delivery of asset management plans and asset life activities, and their continual improvement."

Even when implemented at a simple level, GAM processes can provide the decision makers with data that enables them to make better-informed choices and reduce expenses by optimising investments for geotechnical assets at any point in the asset life-cycle. Also, a systematic GAM programme reduces broader economic impacts associated with asset failures, (e.g., injury, loss of life, or property damage to citizens, businesses, and other governmental agencies) and with mitigation measure as, in the case of railways, speed restriction or line closures [8]. Agencies that embrace GAM generally move away from reactive approach to failures as they start taking advantage from proactively and systematically prioritising work, keeping valuable assets well maintained, and finding cost-effective measures that allow for long-term management of the assets' useful life [9]. Agency executives who are able to authorise a GAM plan are more likely to understand asset measures that indicate what the asset can do in terms of system performance. Network Rail geotechnical asset performance, for example, is assessed with respect to the following measures [10]:

- Train derailments,
- Train delay minutes,
- Temporary train speed reductions, and
- Earthwork failures.

2.2 GAM Taxonomy

A taxonomy is a means for classifying and describing the hierarchical order or relationships for the components of a system. The practice of TAM also uses a taxonomy to help enable common understanding among professionals and maintain consistency in and across asset management processes and data. The chances of a successful GAM incorporation of the eventual plan into an agency-wide TAM plan [11] improve when the taxonomy adopted at the outset of the GAM program has been kept consistent with this integration in mind.

Anderson et al. [12] researched and presented a geotechnical taxonomy (Figure 2.1) for transportation infrastructure assets with the goal to facilitate communication and advancement in GAM and TAM. This taxonomy also resembles the general GAM taxonomy used by Highways England (now National Highways) and Network Rail.

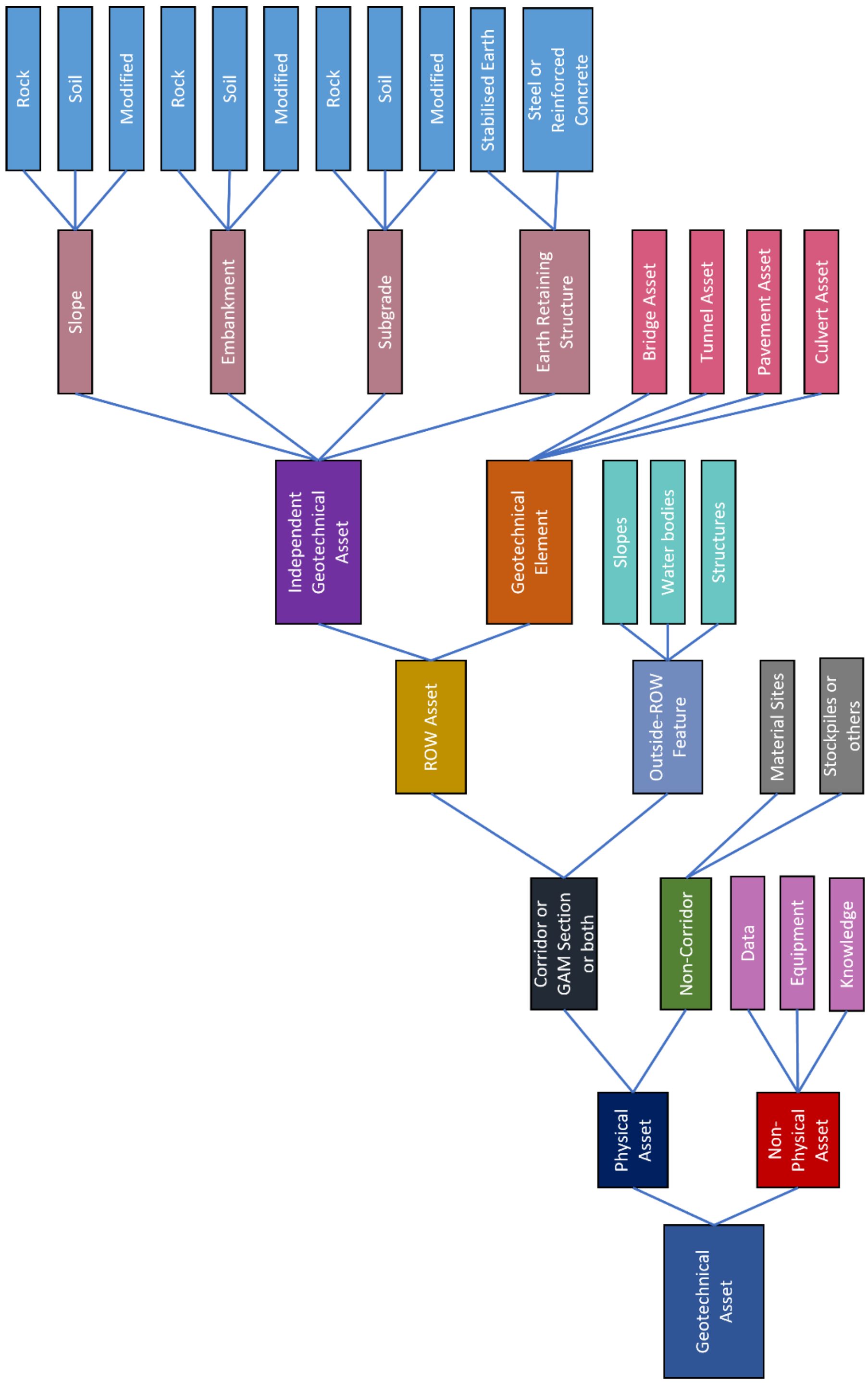


Figure 2.1 Taxonomy of geotechnical assets, elements, and features (after Anderson et al. [12])

2.3 GAM in the UK

In the UK, the management of physical assets was aided by the publication of the Publicly Available Specification document, PAS55 (British Standards Institute, 2004). This was followed by the International Standard ISO 55000 [7], developed under the leadership of the Institute of Asset Management (British Standards Institute, 2014). This last document lists the key aspects of AM best practice including:

- Definition of the levels of performance provided to customers.
- Identification of lifecycle and associated risks of the assets (including deterioration modelling).
- Understanding of cost of ownership.
- Conversion of data into useful information.
- Evidence-based decision making.

In this way, AM should provide the basis for a holistic and coordinated approach ensuring that the infrastructure receives the suitable investment and has the appropriate resilience capability to meet performance requirements. AM covers the whole asset's life cycle: design, construction, operation, renewal and disposal. Each organisation will have its own strategy and approach, nevertheless among all the key principles of AM, safety is fundamental to everything organisations do. Reducing passenger, public and workforce safety risk underpins all of them. However, with climate change and, hence, increasingly frequent severe weather condition, maintaining a high level of safety performance is challenging. This is especially true when managing earthworks [13].

In the UK, responsibility for the various transport networks is divided between Network Rail, Highways Agency, Canal & River Trust, Transport for London and local authorities, and also Transport Scotland, Transport Wales and Northern Ireland Roads Service. Even though all approaches are based on the same basic principles and are underpinned by their statutory and regulatory responsibility for safety, performance and the environment, asset-management strategies vary in detail between the various asset owners and managers. There are no commonly agreed standards that specifically relate to the asset management of geotechnical assets including earthworks [14].

More recently there has been the intention of adopting a more proactive asset management approach. This requires the implementation of a reliable system of monitoring, maintenance, and remediation actions (Sections 2.5, 2.6, 2.7), so that existing earthworks can meet an acceptable level of safety risk, avoid service loss and minimise expensive unplanned remedial works.

In the UK, about 20,000 km of railway earthworks (including embankments and cuttings) support the road network and railway network [15] at various stages of performance and deterioration, mostly managed by Network Rail. Some of this infrastructure is beyond the limit of its intended design life. The network includes many cut slopes and embankments that were developed between 1830 and 1880. Network Rail has established a GAM system that consists of risk-based inventory, assessment, and intervention processes that have resulted in documented improvements in safety and delay risk for the system since implementation 15 years [10]

The Network Rail system has matured with regard to several processes, with recent changes made to the risk assessment process based on asset performance data that enables informed model calibrations. Further, studies of the proactive management of embankment assets supporting railroad lines and motorways in the United Kingdom demonstrated realised life-cycle cost savings of 60 percent to 80 percent per unit length of embankment [4].

2.4 Geotechnical Asset Risk Assessment

Practicing sound asset management requires knowledge of the assets owned, including both the current condition of those assets “today,” how they are likely to deteriorate over their useful life, and the risks their failure or bad performance will pose on the organisation from a financial and also reliability point of view [16].

An important element of the asset management process is the risk assessment which aims to maximise the probability of maintaining a safe and reliable network. At a strategic level, transport infrastructure risk assessment involves a system of procedures and tools capable of evaluating the risks that are tied to the various types of threats to the system. Such a risk assessment will support stakeholders and decision makers to define policies, strategies and investments that aim to reduce vulnerability and/or remove residual risks to valuable assets [17].

The concept of risk provides a rational means for assessing both unfavourable events and conditions as it includes consideration on likelihood of an unfavourable event occurring as well as the consequences of the event itself. Including both likelihood and consequences prevents misleading assumptions: consideration of likelihood alone would tend to overrate probable but minor events, whereas consideration of consequence alone would tend to overemphasize severe events that may be quite unrealistic. The combination of the two gives therefore a more realistic context [18].

For geotechnical assets, physical failure (due to deterioration, overloading etc.) and geologic or natural hazard events (rockfall, landslides, extreme weather events) are primary sources of risk. Deterioration-based risks are fundamentally characterised by consequences associated with continuous deterioration of all assets. Natural hazard risks are a result of events that occur at unique points in space and time, likely not affecting most assets. The analysis of these different failures follow different management approaches. Management of physical failure risks typically is accomplished using deterioration curves (i.e. following Markov-chain model [19], [20]) whereas natural hazards management is based on probabilistic assessment of the hazard events [21].

The current condition of an asset under risk assessment is crucial information [17]. By regular inspections, the condition of the asset is determined, and the information is coupled with historical information, to provide an overall perspective on the evolution of the condition of the asset so to provide information on the potential failure [22]. A risk assessment is performed alongside the safety and commercial risks to develop a funding plan for investment in maintenance and, if required, remediation of the route. Site inspection and risk assessments are used to determine the most appropriate mitigation measure. If remediation is deemed necessary, these measures will be carried out to reduce the consequence of the threat (hazard) event, should it occur, by considering both the severity and duration of the potential impact [23]. For example, a temporary speed restriction may be applied to a section of railway line when a major storm is forecasted, so that in case of a slope failure the trains may be able to stop in time, or at least the speed of impact would be considerably reduced, reducing the severity and increasing the safety. Mitigations may be used to manage the risk for a relatively short period of time until it can be permanently reduced by an intervention. Mitigation may also be used to manage risks that cannot be treated by intervention, such as the installation of a rock fall alarm system along the boundary fence to manage the risk posed by a third-party rock slope. The cost of the mitigation measures and an assessment of the residual risk once those measures have been adopted are evaluated so as to decide on the best strategy and treatment option (example of treatment options are: “Do Minimum”, “Maintain”, “Rehabilitate”, “Reconstruct”, “Restore”) [24].

A distinctive aspect of earthwork assets is their inherent variability. Even if information and knowledge of the change in condition and performance were perfect (an impossibility), there would still be a variability associated with future predictions, related to the uncertain behaviour of geological materials [25]. The behaviour of each earthwork asset is also affected by environmental conditions [26] especially surface water and groundwater consequential upon rainfall events, which are also uncertain. Moreover, in developed countries, and in particular in the UK, many of the earthworks that support the transport networks are suffering because of their ageing. Asset owners must meet users’ expectations of minimal delays while under constraints of ageing assets, imperfect knowledge of their condition, increasing volumes of traffic travelling at higher speeds, increasing environmental pressures and limited resources [26]. Thus, a risk-based approach is essential to characterise the impact of geotechnical variability, and to assign probabilities to future earthwork behaviour [24].

2.4.1 Evaluating Risk (Network Rail approach)

In In the United Kingdom, Network Rail has a risk-based management system for the nearly 200,000 cut slopes, embankments, and rock slopes supporting the nation's rail system. In 2013, following a series of six derailments resulting from historic rainfall in 2012, the agency updated its methodology for estimating hazard index scores, a surrogate for likelihood of failure, from inspection data. The initial hazard score methodology had been established in the early 2000s, based on expert judgment. This initial hazard score was used to assign a Soil Slope Hazard Index (SSHI) based on presumed correlations between various visual observations of distress and five types of failure modes (e.g., deep rotational, shallow rotational, and so forth). Greater values of SSHI were intended to correspond to greater likelihoods of failure, with slopes assigned classifications of "Serviceable," "Marginal," "Poor," or "Top Poor" based on the SSHI score. After a review prompted by the 2012 derailments, the initial SSHI methodology was deemed unsatisfactory based on the observation that approximately 70 percent of slope failures were occurring in slopes deemed "Serviceable" or "Marginal". NR have updated and upgraded the hazard index as output of a recalibration process of the hazard system using observations of approximately 1,000 failed slopes [27]. These Hazard Indices allow each earthwork to be placed into an Earthwork Hazard Category (EHC) ranging from A (lowest Hazard Indices, lowest likelihood of failure) to E (highest Hazard Indices, highest likelihood of failure). The five EHCs A to E are shown in the Earthworks Safety Risk Matrix (Figure 2.2) plotted against Earthworks Asset Criticality Band (EACB). The EACB can be related to a statistical measure of safety consequence that is used throughout NR, allowing comparison across asset types to be undertaken. The EACB is segmented into five bands, from lowest to highest safety consequence designated 1, 2, 3, 4 and 5 [10].

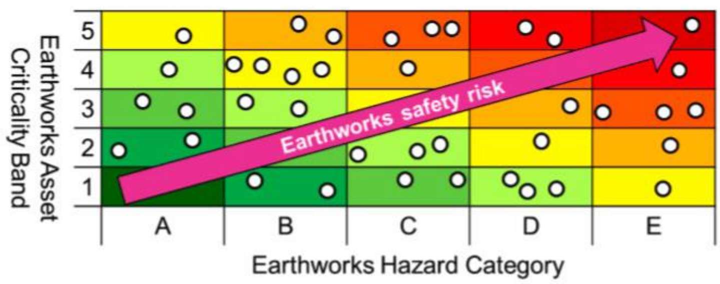


Figure 2.2 Earthworks Safety Risk Matrix (NR Earthworks Asset Policy, 2018)

Many agencies have adopted two-dimensional graphics with likelihood on one axis and consequence on the other. Figure 2.2 shows an example of such an approach, which is used by Network Rail ("Hazard Category" is a term used for likelihood in the Network Rail program while "Criticality" relates to consequence). For the two-dimensional graphics ("risk matrices"), assets that fall farther from the origin are associated with greater risk exposure [28].

2.5 Data for GAM

Smart infrastructure provides digital availability of data and information which lays down the basis for helping infrastructure owners to plan effectively, save money by reducing disruptions to services and interpret the deterioration characteristics of the infrastructure [29]. However, the term deterioration describes a change leading to a loss of performance with respect to a requirement. This change in performance can be driven by multiple mechanisms which can act independently or simultaneously. The way in which data are recorded varies from organisation to organisation and even from individual to individual. Thus, data significance may be difficult to interpret. The scale of the problem is difficult to assess as different elements of the works may be accounted for under different budget headings within organisations [30].

For geotechnical assets the complex relationship between what drives the deterioration (soil mechanics properties (Section 3.4), geotechnical issues (Section 3.5), external factors (Sections 3.5.5 and 3.5.6)) and the mechanism of deterioration affecting the requirements (Sections 3.2.4 and 3.3) makes challenging the interpretation of information describing the behaviour of the structure. The expectation of performance can be interpreted broadly to include considerations such as the level of service (e.g., for road traffic flow or rail ride quality), the cost of ownership and maintenance, financial utilisation or regulatory compliance. Together, such considerations can be combined to assess the deterioration of geotechnical assets. Deterioration is driven by human actions (i.e. physical loading due to traffic) and environmental actions (i.e. seasonal deformation cycles and extreme weather conditions) which change the properties of the asset at the soil scale (i.e. changes in strength, stiffness, permeability) and at the asset scale (i.e. changes in slope geometry or structural integrity). The deterioration of geotechnical assets can cause localised transport disruption, which then propagates throughout the entire network [31].

To efficiently contextualise the available information on geotechnical assets deterioration and, in parallel enlarge the understanding with new information, the relationship between key factors and asset properties needs continuous investigation. The way these elements are indirectly linked to the expectations of performance is a crucial point for the geotechnical asset management [11].

2.6 Rail assets monitoring and maintenance

Rail asset monitoring and maintenance remain critical challenges facing railway infrastructure in terms of cost-effectiveness, workforce safety and operational efficiency [32]]. The lack of appropriate and timely information for efficiently plan maintenance action, resulting often in delayed response to failing assets, therefore in unscheduled disruption of operations and train delays, is a major cause of this critical challenge [33]. Moreover, the way data is acquired for understanding the condition of rail assets remains risky, rail assets operatives still enter danger-zones for conducting rail assets inspection and maintenance [34]. The development of modern approaches is necessary to guarantee a safer working condition for rail assets operations and deliver a more reliable service to rail customers.

2.6.1 Monitoring

Smethurst et al. [31] provide a comprehensive review of current and future technologies for monitoring the performance of transport infrastructure slopes. A report by Network Rail [10] comprises a high-level review of the ways in which new monitoring and surveillance technologies can provide enhanced approaches to slope management. It draws upon recent literature and case studies, and highlights that new technologies are rapidly developing in this area. The combination of new, low-cost sensors; novel terrestrial, aerial and space-based platforms; improved instrumentation; rapidly developing, powerful algorithms; and high-performance computing provides ample opportunities for innovation. The report emphasises that the greatest advances will come from the use of multiple technologies together. There is a need to adopt reliable methods of monitoring which can inform engineers of the condition of the more critical geotechnical assets and of any significant changes occurring.

Monitoring instrumentations can be used for failure detection (reaction via alert alarm systems) or to provide data on performance and condition of earthwork and possible signs of failure [35]. Smethurst [31] found that the majority of the technologies for surface and sub-surface deformation monitoring, are not suited to respond to rapid or instantaneous failure. Nevertheless, many of these technologies are of great help when it comes to detect movement over a period of time and indication of ongoing movement can be obtained for either slopes or embankments.

In view of the very large number of earthworks sites, as in the UK, the impracticability to have widespread monitoring instrumentations is well recognised [36]. Traditional measurement, for example, of pore pressures and soil deformations, by installing piezometers and inclinometers in boreholes, is essential, however, to obtain a more detailed understanding of the behaviour of a particular slope or embankment, that is judged to be critical when the readings are taken manually.

There have been some recent advances in the updating of monitoring and surveillance methods. A notable example is the recently trialled wireless tiltmeter system [37]; this is an extremely promising application of innovative sensor development to the management of earthworks assets. Such monitoring systems, installed on earthworks that are judged as potentially critical, can provide failure detection and reaction via alert alarm systems, as well as providing data on the performance and condition of a slope or embankment, and possible precursors to failure. Recent methods are:

- Surveillance technologies with helicopters and drones for inspections of earthworks, particularly after especially intense rainfall, serve as a key impact mitigation by providing warning or identification of an earthwork failure. Despite some existing limitations, particularly in respect of current regulations and privacy limits, drone technology usage is a rapidly increasing tool for locating obstructions on the track, identifying changes, and mapping features [35], [38].
- InSAR is an established satellite technology which is also developing rapidly, with substantial promising developments in AI and machine learning. Routine analysis of track geometry data is a potentially valuable technique for the early detection of embankment instability; but there is yet no fully developed procedure to establish automatic data processing [35], [37].
- International experience indicates that the most promising surveillance technologies for slope and landslide management are LiDAR and photogrammetry (both aerial and land-based).
- Wireless sensors have been shown to be effective for monitoring of slope movements, provision of warning systems and detection of flexible barrier deformations. There is significant potential for these advanced monitoring technologies; they need to be managed centrally, replacing several aspects of the well-established visual examination procedure [37].

Visual inspections, indeed, rely heavily on data collected by technicians in the field subjected to individual interpretation and are prone to human error neglecting certain indicators to slope failure [27].

2.6.2 Maintenance

Maintenance forms a critical part of the asset management regime and is undertaken in order to maintain the assets in their current condition, preserving the ongoing safety and serviceability of the transport network, and to minimise future asset deterioration and degradation, thereby preventing or postponing the need for major remediation [39]. Ideally preventative maintenance should be carried out on a routine basis, e.g. through vegetation management and cleaning drainage systems. The frequency of preventative maintenance is governed by the severity of any potential problems and their consequences [40]. Corrective maintenance is undertaken where defects are observed during inspection or site walkovers. Prioritisation based on risk is used to determine the maintenance programme.

Maintenance is governed by different industry-specific internal guidance and commonly involves the intervention activities shown in Table 2.1.

Table 2.1 Main maintenance activities on earthworks

Main maintenance activities on earthworks
Drainage maintenance
Vegetation management
De-vegetation of drains and of minor retaining structures
Debris and refuse clearance
Servicing Engineered support
Clearing catch fences/ netting
Servicing GIA or alert/ alarm systems

Asset owners have increasingly recognised the necessity to progressively adopt a more integrated approach to the management of Earthworks, Drainage and Vegetation. A key element of this new tendency is the improvement required of the current earthworks vegetation maintenance regime in order that the positive effects of vegetation on slope stability (reduced surface erosion, greater root reinforcement, avoidance of channelling of flows, maintenance of surface pore water suctions) may be enhanced while, at the same time, the detrimental effects (blocked ditches and pipes, clay shrinkage and desiccation cracking) can be minimised [41].

2.7 Remediation

Almost all methods of repair or refurbishment aimed at improving the structural condition will also result in an improvement in the service condition as the asset has to be cleaned in order to carry out the repair or refurbishment [39].

Remediation is required when an embankment fails to meet its performance target. Performance may be defined differently depending on the mode of transport, but it will surely include the requirements for safety, speed and ride quality. Failure may be defined in terms of serviceability, and ultimate limit state, or catastrophic failure. Remediation methods vary. The choice of method used is obviously dictated by the underlying causes of failure and the failure mechanism [10]. However, there are many other factors that influence the decision, which may be in mutual conflict. These include safe procedure requirement, acceptable method statement, availability of line (route possessions), cost, site access, size and mass of equipment and future maintenance requirements. For example, the cost of closing a railway, or renting a lane on a motorway, is very expensive, so if the best technical solution is recommended to be carried out from the top of the railway or the base of the highway, that solution is highly unlikely to be used even if, in other respects, the solution has the lowest cost and the longest design life. Furthermore, the cost of unplanned repair can be considerably higher than the cost of routine maintenance—costs of ten times higher have been recorded when compensation payments are considered [42].

2.8 Conclusions and Next steps

A generic asset management process begins with a clear idea of the goals and objectives of the organisation, coherent with the policies and strategies for effectively managing their assets (in this case geotechnical). Subsequently, the condition of the assets must be identified to compare the existing and expected performance of the asset. The next step is to undertake feasibility studies, which include undertaking adequate financial, risk and resource analysis, in order to develop optimised solutions. This is followed by delivering the service as per the asset owner's requirements while also monitoring the performance of the delivery. This chapter has briefly (not exhaustively) reviewed the recent strategies and technological developments available to support organisations' improvement of the understanding and safety of earthworks along the railway lines. Organisations are investing significantly in new technologies and instrumentations to try and move from an old reactive philosophy to a more pro-active one. The greatest opportunities will likely come from bringing together multiple asset parameters contributing to its stability addressed by various technologies and integrating multiple approaches to examine the problem.

One of the biggest challenges of GAM is to detect slope failure before this happens. Identifying the parameters that play a key a role in good or bad geotechnical asset performance and in slope instability, is one of the objectives of this research work undertaken in Chapter 3. The literature review will help to find both factors affecting embankment stability and signalling potential failure. The study of these factors will deliver a more comprehensive understanding of the problem faced by asset managers which will assist the long transition process from the common reactive approach to a more efficient and effective proactive approach.

CHAPTER 3. LITERATURE REVIEW

3. Literature Review

As highlighted in the previous chapter, one of the objectives of this research work is to identify the geotechnical parameters playing a role in good and bad embankment asset performance. Understanding earthwork deterioration is well-established with much work carried out previously on asset condition assessment [15], [35], [43]. Although these works are valuable, more focus is required to move from the day-to-day management of embankments and reactive maintenance to a predict-and-prevent approach. Indeed, existing models are not capable of scanning the high number of embankment sites that exist around the UK rail network so as to identify a manageable number of at-risk assets.

This literature review describes typical factors playing an active role in railway embankment failures and issues in the current Geotechnical Asset Management (GAM) approach [44]. This section constitutes the starting point of the research and aims to advance the objectives of this thesis by covering the following knowledge areas:

- Railway infrastructure construction history in the UK
- Factors affecting slope stability
- Track geometry
- Current geotechnical asset management
- Multi-Criteria Decision-making MCDM methods (Chapter 6)

3.1 British Railway Embankment Infrastructure History

The British railway system is one of the oldest in the world [45]. To help to understand a railway system's good and bad performance, and in particular railway embankments' performance, an overview of some of the main steps of its development follows in the next paragraph.

3.1.1 Change in British Railway

The railway infrastructure was born around the end of the 16th century. Many wagonways were built between the 17th and 18th centuries used just on short distances, mostly for mining purposes.

The first public railway line for passenger transportation was introduced only in 1830 and ran from Liverpool to Manchester. Between 1835 and 1841 nine main lines of railway were built in the United Kingdom for a total length of about 1,060 km involving the excavation in cuttings of 54 million m^3 of material, most of which was used for making embankments [45]. This was the period in which long distance transportation started for both goods and passengers. In the United Kingdom the development increased rapidly during the ‘40s of the 19^{th} century reaching a network of 10.715 km the end of the decade [46].

3.1.2 Railway Embankments – Description, History and Construction

Embankments, together with cuttings, are part of those civil engineering structures known as earthworks. They are an important means of physically forming the trafficked surface of transport infrastructure.

Embankments are made from materials placed on natural ground that are commonly composed of soil or rock excavated from elsewhere. Infrastructure embankments carry railways, roads and canals, maintaining their vertical alignment by raising their level above the surrounding ground. Cuttings are excavations in existing ground, with side slopes; they also provide passage for rail, road and canal traffic across natural ground to maintain the vertical alignment.

Like other engineering structures, embankments require maintenance, and the need to undertake it has become increasingly apparent as the material within these structures age. Deterioration under the effect of traffic load, weather, animal burrow, vegetation, time etc. can lead to strength reduction, microstructural change, alteration of asset geometry and instability. Unstable trafficked surfaces bring immediate safety consequences for users and cost implications for owners.

At the beginning of their construction, the excavation of a single earthwork was easy but expensive. The total cost required to construct an embankment , alone, would have been double the cost involved in building that embankment formed of material from an adjacent cutting. Moreover, extra land was required outside the normal boundaries useless pits would be left alongside the line. Clearly it was desirable to balance “cut” and “fill”, both in total and over relatively short distances [47]. As general rule then, embankments were formed from cuttings not too far distant. In this way a combined system of cutting and embankment was achieved, and large quantities of material were managed rapidly [48].

The legacy of the construction methods, as explained in the following paragraphs, is reflected in the performance of earthworks and hence in the need for current maintenance.

3.1.2.1 History and construction of infrastructure embankments

Old railway earthworks were built without a detailed understanding of slope stability and rock mechanics [49].

Most old embankments were, thus, built with much steeper slopes than modern ones. As slopes become older, their strength reduces because of the migration of pore water, with time, toward areas where the soil has, previously, remained in the condition when it was excavated. Often, this causes soil structure softening and lead to increasing slope instability. Nowadays, slope angles are constructed to allow for such softening [47], but this future decay was not recognized when the early embankments were being constructed.

The history of embankment development in railways started around the 1830s. Horse-drawn wagons (Figure 3.1) transported the material excavated from soil cuttings to fill areas where the soil or rock was end-tipped or side-tipped to form poorly compacted embankments. The best way to build an embankment, especially in clay, would have been to form the bank in shallow layers (between 0.6 and 1.2 meters of thickness) for the full length, and give each layer and foundation enough time to compact before placing a new layer. Despite its reliability, this method was little used to construct as it was too slow and often didn’t fit with the balanced cutting-embankment process.

Where feasible, material excavated from cuttings was placed in adjacent embankments. Therefore, an indication of the material in an embankment can be based on the geology of the cutting nearby. Fill for the embankments was generally placed by end-tipping to full height and so with no formal compaction and with little breakdown. The greater rate of construction achieved by high end-tipping, as opposed to building in shallow layers, was gained at the expense of increased settlements [50]. Following this procedure, the weakest soil was excavated first from the surface to create the cutting, then placed first to elevate the embankment. Consequently, the embankment was placed on the weakest fill. Moreover, Skempton [50] provides a very informative document on the construction of the main railway lines in England between 1834 and 1841 and no information is given on foundation preparation, in terms of drainage or removal of unsuitable material. Failures wholly within the embankments occurred during construction and at one or two years after construction. These were usually in uncompacted clay fills: Upper Lias, Oxford Clay, Weald Clay and London Clay (all high plasticity clays). As much of the fill was dug by pick and shovel, clay fill remained in the form of lumps within the embankments. The delayed failures are attributed to softening of the clay lumps as rainwater entered the bank. Settlement of the uncompacted fill in the embankments occurred and was compensated for by placing additional ballast. Ash from coal-fired steam trains and power stations has also been used for this purpose. At some locations borrow areas were dug immediately adjacent to the embankments that were under construction, a procedure referred to as "side cutting". The presence of these pits could affect the stability of the bank [45].

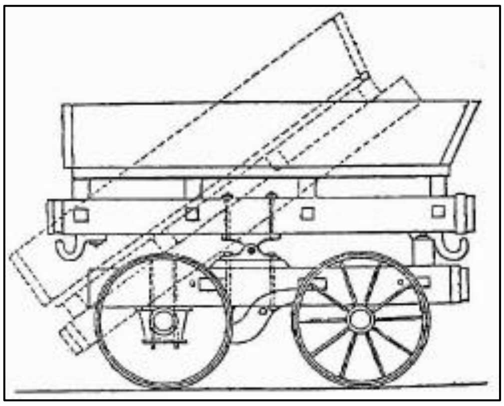


Figure 3.1 Earth wagon (after [51])

Nearly all railway embankments were constructed of relatively un-compacted material. Before the 1930s, little or no compaction was possible as the process and benefits of compaction were poorly understood and heavy plant was, in any event, unavailable. Soil compaction is nowadays recognised as an important component of soil management. Compaction of the soil and removing air voids increases the soil's sheer strength, decreases its compressibility and its permeability. It also reduces the voids ratio making it more difficult for water to flow through soil [14]. There are several objectives to soil compaction:

- Increase bearing capacity
- Increase durability
- Increase resistance to deformation
- Decrease frost damage
- Increase stability
- Decrease permeability

Regardless of the soil type (cohesive or non-cohesive), failure to remove the air between soil particles before building any construction structure can later cause unexpected and unwanted movement of the soil and penetration or absorption of water beneath the embankments.

The legacy of these poor construction methods led to large settlements and slope failures commonly occurring during or soon after construction [52]. Failure commonly occurs at or about the interface between the natural ground and the embankment fill. This can be exacerbated by seepage in the natural ground. As for railways, the embankments were not benched into the existing ground, and in many cases the original topsoil would have been left in place, forming a potential rupture surface. Some embankments were constructed on soils that contain pre-existing rupture surfaces, which can promote deep failures and failures on sidelong ground. Slope failures that begin as shallow failures may progress to deep failures. Shallow failures are often serviceability limit state failures, whereas deep failures are almost always ultimate limit state failures [53].

Also, the embankment slope angle was based on short-term angles of repose attained during construction as this minimised the amount of soil needed per metre length of embankment. The slopes to the embankments developed at the angles of repose of the material being placed and were trimmed to slopes of 1.5

or 2 (horizontal) to 1 (vertical). These would be considered over-steep in modern practice [47].

3.1.2.2 Modern Embankment Construction

The main goal when designing earthwork construction, is to build a stable and safe structure for the long-term. Specific performance requirements are different based on the final use of the structure (dam, bank, cutting, embankment and so on). The Eurocode [54] provides comprehensive information on all actions that should normally be considered in the design of buildings and other civil engineering works. Section 7 of the Eurocode is dedicated to geotechnical design [54]. To give an idea of how embankments are constructed nowadays, the main steps of construction method are briefly described in the following paragraphs.

The embankment consists of a series of compacted layers of suitable material placed on top of each other until the level of the subgrade surface is reached.

For embankment structure in transportation field, generally, the first step is the exclusive use of adequate soils with specific natural features to enforce quality control measures. These characteristics are previously defined through laboratory and in situ procedures applied to samples collected the procedures include grain size distribution tests, evaluation of consistency limits, analyses of natural water content, oedometer tests and triaxial tests. Some examples of test and related property investigated can be found in the following Table 3.1 [55]:

Table 3.1 Some examples of measurement of soil properties

Type of Test	Property evaluated
Sand-cone Method	Density of Soil
Rubber Balloon Method	Density and Unit Weight of Soil
Nuclear Method	Density of Soil and Soil Aggregate
California Bearing Ratio (CBR)	Strength and Stiffness
Dynamic Cone Penetrometer (DCP) Index	Strength and Stability of Compacted Soil
Moisture tester	Optimum Moisture Content

The second step is the preparation of the surface between the embankment and the underlying original ground surface; the vegetation is removed, and the surface is replaced with compacted coarse grain material with anti-capillary function. The construction of the embankment itself establishes the materials chosen in layers which thicknesses previously designed according to the test results on the materials. After the compaction, the soils used need to guarantee adequate value of density and compressibility [56]. The surface slope depends on the material features; generally a gradient of 1:1.5 (height:width) for each layer is sufficient to guarantee the global stability of the embankment. The surface between the embankment and the railway sub-structure must show high stiffness (low deformations under cycle loading) and need to be sufficiently flat to receive the railway sub-structure, although some cross-fall may be desirable in order to help shed water that percolates down through ballast and sub-ballast. Material for this surface should show better mechanical characteristics than the material used for the underlying part of the embankment and should receive a more intense compaction.

Highway embankments are more recent structures than railway embankments (Figure 3.2), therefore they have been constructed following modern building techniques. Compared to railway embankments, highway embankments have better maintained road and toe drainage, combined with a largely impermeable surface, whereas railway embankments have a permeable ballast surface and often poorly maintained drainage [57]. These characteristics combine with variations in vegetation and climate, to influence the seasonal deformation behaviour. In addition, where track problems and progressive deformation of the subgrade are ongoing, track drainage can be further impeded and lead to concentrations of water and exacerbation of trackbed problems [58].

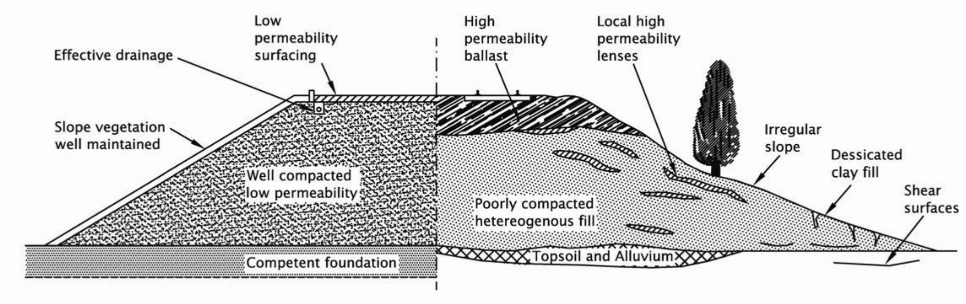


Figure 3.2 Difference between Highway and Railway Embankment (after [15])

3.2 Railway Track Condition

When embankment instability starts developing, the track follows the movement of the earthworks and this movement is recorded as a deterioration of the vertical and horizontal alignment, as shown in Figure 3.3 [59], [60]. Specific railway performance requirements are strictly related to track geometry quality. The European Standard EN 13848-1 "Railway applications/Track - Track geometry quality" [61] defines track geometry quality as "assessment of excursions from the mean or designed geometrical characteristics of specified parameters in the vertical and lateral planes which give rise to safety concerns or have a correlation with ride quality". It also defines the minimum requirements for the quality levels of track geometry, and specifies the safety related limits for each track geometry parameter. As speeds have risen, the quality of track geometry has become increasingly important: a small irregularity will hardly be noticed in a slow-moving train whereas passenger comfort in a high-speed train might be significantly compromised and the dynamic load applied to the track might be damaging to both track and train [60].

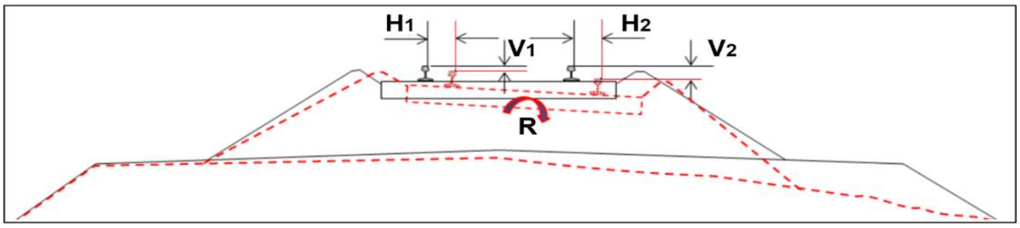


Figure 3.3 Track movement on an unstable embankment (after [59])

3.2.1 Track Geometry

Five parameters are typically used by railway agencies to assess track geometry (Figure 3.4): vertical alignment (top), horizontal alignment (line), cross level (cant), gauge and twist. Standards prescribe minimum and maximum allowable values for these parameters based on the type of railway line. BS EN 13848 [61] states the existence of three indicators of track quality: extreme values for isolated defects, standard deviation (SD) in a typical length (e.g. 200 m) and the mean value. Depending on type of line and speed, there are three main limits for these indicators, above which different actions need to be undertaken [18]: the Immediate Action Limit (IAL); the Intervention Limit (IL); and the Alert Limit (AL). More details can be found in Section 3.2.3.

A track recording vehicle (TRV) is used to measure track parameters, details of the track recording vehicle can be found in Section 3.2.2.

3.2.1.1 Principal track geometry parameters

There are five measurable track parameters (Figure 3.4); vertical alignment (top), horizontal alignment (line), cross level (cant), gauge and twist. The alignments are evaluated along a space domain called wavelength λ.

According to Hamid and Gross (1981), Bing and Gross (1983), Sadeghi and Askarinejad (2008), Shafahi and Hakhamaneshi (2009) [62]–[65] these parameters, used correctly and described in terms of standard deviations, can give a track quality index (TQI) [66]. Within Network Rail, the vertical alignment (top) for the short wavelength (35m) is considered to be a good overall indicator of track quality (Network Rail - NR/L2/TRK/001/C01) [22]. This view is also supported by Thom and Oakley (2006) [67] who state that while gauge variation and horizontal alignment play a part in the operational quality of a railway track bed, poor vertical alignment results in poor ride quality for passengers and unwanted dynamic forces for track and vehicle components.

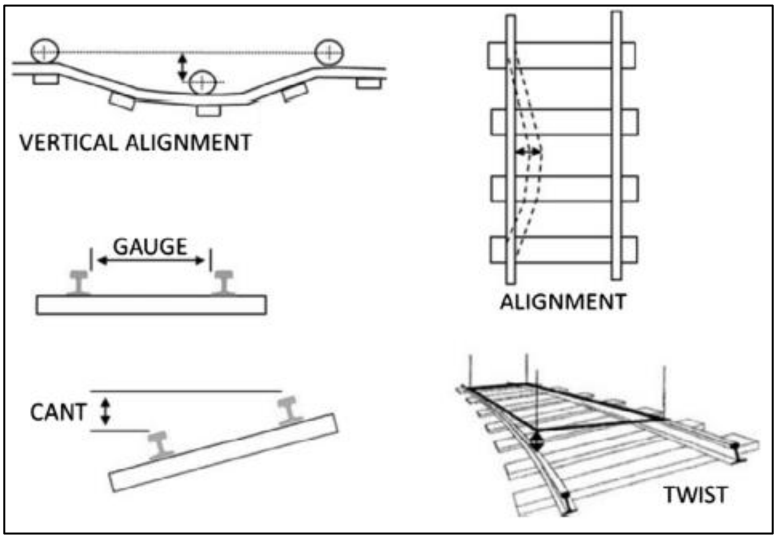


Figure 3.4 Track quality parameters (after [68])

- Alignment

Alignment of track is made up of two components namely horizontal alignment and vertical alignment expressed as an excursion from the main respectively horizontal and vertical position (reference line) of the track referred to a defined wavelength chosen according to the line speed (usually 35m o 70 m wavelength). The vertical alignment is used as the main indicator for track quality, the key contributor to poor vertical alignment is differential settlement [58]. For the purpose of this research, the alignment parameters have fundamental importance as it was observed [59] that simultaneous deterioration in the vertical and horizontal alignments indicates potential earthworks instability in most cases. When this is corroborated by rotational movement of the track, it is almost certain that the problems can be attributed to earthworks weakness.

- Cant or Cross level

Track cross level is the elevation of the outside rail minus the elevation of the inside rail. It is also known as super-elevation or cant. It is used to increase the maximum safe speed of a train through a curve [68].

- Gauge

Rail gauge is the distance between the inner sides of the two load bearing rails, measured 14 mm z_p below the running surface. Track in Britain is laid at standard gauge (1435 mm); this is the most common gauge world-wide. Other common gauge dimensions used around the world are metric gauge (1000 mm) or (1067 mm) and broad gauge (1524 mm (Russia), 1668 mm (Spain and Portugal), 1600 mm (Ireland), 1676 mm (East Asia)).

There are also many different gauges used mostly for low-speed trains, industrial and mountainous lines [68].

- Twist

This is the algebraic difference between two cross levels taken at a defined distance apart, usually expressed as a gradient between the two points of measurement. Twist may be expressed as a ratio (% or mm/m) [68].

3.2.2 Track geometry measurement and quality

The track geometry routine inspection, when accurate and detailed, allows the identification and rectification of geometry faults before an unacceptable level of risk is presented. Routine inspection includes visual inspections, detailed examinations, cab riding and sampled manual measurement of track geometry [36]. The use of Train Recording Vehicles (TRVs), wherever practicable, is planned on the following:

- passengers running line, goods running lines, carriage lines and loops.
- high speed (60mph and above) crossover routes.
- long crossovers with more than five sleepers between the through timbers on the crossover road.
- crossovers between lines with different cant.

The frequency of measurement depends on the track category (Figure 3.6), assigned based on the track speed and the Equivalent million gross tonnes per annum EMGTPA (a measure of the annual tonnage carried by a section of track, so as to take into account variations in the damaging potential of normal traffic). The track geometry tends to move from the design geometry with the continuous passage of vehicles. Defects in the geometry are caused by track support settlement and local irregularities associated with dipped joints, wheel burn, corrugation, etc [69].

TRVs accurately measure track geometry and provide the magnitude and locations of changes in geometry so to inform the track maintenance team. With TRV observations, both sudden and longer-term changes in track geometry can be detected [70].

BS EN 13848 [61] states the existence of three indicators of track quality: extreme values for isolated defects, standard deviation (SD) over a typical length (200 m) and mean value. In order to plan and/or predict maintenance interventions, rail authorities and practitioners often use the standard deviation as a convenient means of quantifying the geometric quality of a track section [66]. According to the European track geometry quality standard, longitudinal level is measured for individual rails and defined as the deviation of consecutive vertical alignment of rail levels from the mean vertical position. The vehicles (Figure 3.5) use a variety of measuring systems (sensors and lasers) mounted on a bogie to detect and translate characteristics of track geometry into quantities. Before starting a run, track features such as track identification and mileage are input either manually or automatically into the TRV. As well as track geometry parameters, other parameters are recorded such as the distance ran by the TRV in order to aid geo-referencing of recorded measurements. Location is either obtained automatically by use of a satellite positioning system or manually using mile posts. Twist, curvature, horizontal alignment and vertical alignment are either directly measured or calculated by the TRV. The data processing involves calculation of standard deviation for track segments. Outputs from TRVs are used to plan maintenance, track quality monitoring and safety assurance as related to track geometry [71].

Track geometry quality is expressed as achieving a particular status by categorising Standard Deviation (SD) values for Vertical Alignment (Top) and Horizontal Alignment (Line) (for 35 m and 70 m wavelengths) (Table 3.2). In the UK SD values are calculated for 8th mile (220 yards or 201.168 m) section of track.

The track quality bands are: Good; Satisfactory; Poor; Very poor; Maximum (35 m wavelength filter only).

Table 3.2 Track geometry quality band Standard Deviation SD in mm (after [71])

Speed Range [mph]	35m wavelength filter										70m wavelength filter							
	Vertical Alignment					Horizontal Alignment					Vertical Alignment				Horizontal Alignment			
	Good	Satisfactory	Poor	Very Poor	Minimum	Good	Satisfactory	Poor	Very Poor	Minimum	Good	Satisfactory	Poor	Very Poor	Good	Satisfactory	Poor	Very Poor
10 -20	5.2	7.4	8.3	9.9	>9.9	3	5	5.6	9.9	>9.9								
25 -30	4.3	6.1	7	7.7	>7.7	2.7	4.5	5.2	8.6	>8.6								
35 - 40	4.1	5.8	6.7	7.2	>7.2	2.5	4.1	4.7	7.9	>7.9								
45 – 50	3.8	5.4	6.3	6.7	>6.7	2.2	3.7	4.5	7.3	>7.3								
55 – 60	3.5	5	5.9	6.3	>6.3	2	3.3	4.2	7	>7								
65 – 70	3	4.3	5.4	6	>6	1.7	2.9	3.6	6.7	>6.7								
75 - 80	2.7	3.8	4.8	5.7	>5.7	1.5	2.5	3.1	6.3	>6.3	3.7	5.7	6.3	>6.3	3	5.2	5.7	5.7
85 - 95	2.5	3.2	4	5.3	>5.3	1.3	2.1	2.7	6	>6	3.3	5.1	5.6	>5.6	2.6	4.5	5	>5
100 - 110	1.9	2.7	3.4	5	>5	1.1	1.8	2.3	5.7	>5.7	2.9	4.5	5	>5	2.2	3.8	4.3	>4.3
115 - 125	1.7	2.4	3	4.7	>4.7	1	1.6	2	5	>5	2.4	4	4.4		1.8	3.2	3.7	>3.7

Table 3.3 Frequency of Track Geometry Measurement (after [71])

Track Category	Frequency (Nominal Planning Interval)	Maximum Interval between measurement
1A	4 weekly	10 weeks
1	8 weekly	18 weeks
2	12 weekly	26 weeks
3	16 weekly	36 weeks
4	24 weekly	52 weeks
5	24 weekly	52 weeks
6	24 weekly	52 weeks

Figure 3.5 Track recording vehicle (after [72])

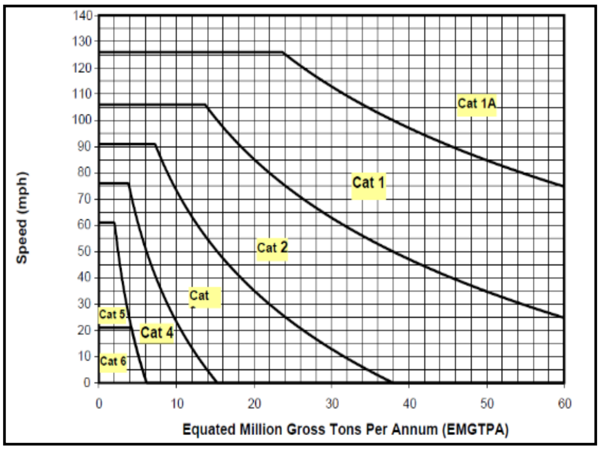


Figure 3.6 Track category (after [71])

3.2.3 Maintenance Plan

As defined in BS EN 13848-1 [61], there are three *limits* for each geometry parameter, depending on the type of line and the speed. Different actions need to be undertaken accordingly to the specific limit:

- Immediate Action Limit (IAL): refers to the value which, if exceeded, requires taking measures to reduce the risk of derailment. This can be done either by closing the line, reducing speed or by correction of track geometry.
- Intervention Limit (IL): refers to the value which, if exceeded, requires corrective maintenance in order that the immediate action limit shall not be reached before the next inspection.
- Alert Limit (AL): refers to the value which, if exceeded, requires that the track geometry condition is analysed and considered for the planned maintenance operations.

The output from track geometry measurement is used to create maintenance plans so as to, among other things, prevent Immediate Action Level faults occurring and to reduce the number of Intervention Level Faults occurring or repeating. A maintenance plan also allows track quality and geometry targets to be attained and management of sites where there is ground instability (from desiccation, animal burrowing, rotational or sliding ground movement, mining subsidence, or other cause).

3.2.4 Soil Mechanics properties

Slope failure occurs when the downward active forces on material due to gravity and train dynamics, induce shear stresses in the soil that exceed its shear strength. The shear strength is a consequence of the geotechnical properties of the rock or soil mass and its state of effective stress. Therefore, factors that tend to increase the shear stresses or decrease the shear strength increase the chances of a slope failure [73]. Different processes can lead to reduction in the shear strengths of the mass. Increased pore pressure, cracking, swelling, decomposition of clayey rock fills, creep under sustained loads, leaching, strain softening, weathering, erosion and cyclic loading are common factors that decrease the shear strength of mass. In contrast to this the shear stress may increase due to additional loads and increase of water pressure in cracks at the top, increase in soil weight due to increased water content, excavation at the bottom of the slope and seismic effects

[52], [73]. The mechanical properties affecting slope failure are discussed in following paragraphs.

3.2.5 Soil properties

The soil or rock shear strength affects the stability of a slope. This, in turn, is related to the soil or rock particle size distribution, density, permeability, moisture content, plasticity. Many of the geotechnical properties of soils have an influence on each other.

3.2.5.1 Specific gravity

The specific gravity of a certain material is defined as the ratio of the weight of a given volume of the material to the weight of an equal volume of distilled water. It is an important index property of soils that is closely linked with mineralogy or chemical composition [74] and also reflects the history of weathering [75]. Roy et al. [76] found that increase in specific gravity can increase the shear strength parameters (apparent cohesion and angle of shearing resistance). In general, higher values are associated with soils with greater particle strength. However, it is unrelated to the quality of the packing together of the particles so is, at best, only a partial indicator of soil quality. Typical values of specific gravity are given in Table 3.4.

Table 3.4 Typical values of specific gravity (after [77])

Type of soil	Specific gravity [t/m^3]
Sand	2.65 – 2.67
Silty sand	2.67 – 2.70
Inorganic clay	2.70 – 2.80
Soil with mica or iron	2.75 – 3.00
Organic soil	1.00 – 2.60

The moisture content of a soil is not a direct indicator of the soil's strength because more clayey soils can hold more water and remain strong. Therefore, the Consistency Index, CI, was introduced by Atterberg (1911) [78] and is defined as:

$$CI = 1 - LI = 1 - \frac{w_N - PL}{LL - PL} = 1 - \frac{w_N - PL}{PI}$$

Where:

w_N is the natural moisture content

PL is the plasticity limit

LL is the liquidity limit

LI is the liquidity index

PI is the plasticity index

This indicates how far a soil's condition lies between its Liquid Limit where the strength will be very low, and the Plastic Limit where it will be much higher

3.2.5.2 Compaction

Compaction of soils is the process in which a soil sustains mechanical stress and densifies by the exclusion of interstitial air, so that the particles are redistributed into a closer state of contact with each other. The mechanical stress may be applied by kneading, or via dynamic or static methods. The degree of compaction is quantified by measuring the change of the soil's dry unit weight relative to the maximum change that can be achieved under reference conditions and loading [79].

Within the framework of engineering applications, compaction is particularly useful as it results in:

- An increase in strength of soils
- A decrease in compressibility of soils
- A decrease in permeability of soils

The durability and stability of a structure are related to the achievement of proper soil compaction; this is one of the most critical components in the construction of railway embankments.

Serviceability and ultimate state problems can often be traced back to the failure to achieve proper soil compaction. Indeed, compaction became a standard procedure in the construction of embankments and earthworks in general; no other processes applied to natural structure soils produces such relevant positive changes in the physical properties [80].

When soil particles are forced together by compaction, the reduction in voids reduces the permeability, thus the seepage of water in reduced. At the same time, the movement of capillary water is minimized, this reduces the tendency to take up water and suffer later reductions in shearing resistance.

For every soil, there is an optimum amount of moisture at which it can experience its maximum densification under a particular mechanical compaction. For a given compactive effort, a soil is reaches its maximum dry unit weight $\gamma d,max$ at an optimum water content level wopt [81].

The compactibility of a relatively dry soil increases as water is added to it. That is, for water content levels dry of optimum (wopt), the water acts as a lubricant, enabling soil particles to slide relative to each other, thus leading to a denser configuration. Beyond a certain water content level (wet of optimum, w>wopt), excess water within the soil results in pore water pressure increase that tends to keep the soil particles apart. A typical correlation between the dry unit weight and the water content is presented in Figure 3.7. Also, it is worthwhile to note that, as it can be seen in Figure 3.7, for a given soil, the highest strength is achieved just dry of optimum, while the lowest hydraulic conductivity is achieved just wet of optimum. The effect of the compactive effort on the maximum dry unit weight ($\gamma d,max$), and the optimum water content level (wopt) can be also observed. With increased in compactive effort, $\gamma d,max$ increases, while wopt decreases. That is, a smaller water content level is sufficient to achieve a denser sample.

3.2.5.3 Density Index

Density index is a measure of the degree of compaction and hence, indirectly, the stability of a stratum [76]. Compaction degree of fine-grained soil is measured in relation to maximum dry density for a certain compaction effort. But in the case of coarse-grained soils there are two extreme states of compaction: the loosest and densest states (the values of which will depend on the shape, size and grain-size distribution of the soil grains). Any intermediate state of compaction can be compared to these two extreme states using the so-called density index:

$$DI = \frac{e_{max} - e}{e_{max} - e_{min}}$$

Where

e_{max} = voids ratio in the losest state

e_{min}= voids ratio in the densest state

e = natural voids ration of the deposit.

Empirically, the soil characteristics based on the density index have been observed as shown in Table 3.5 [82].

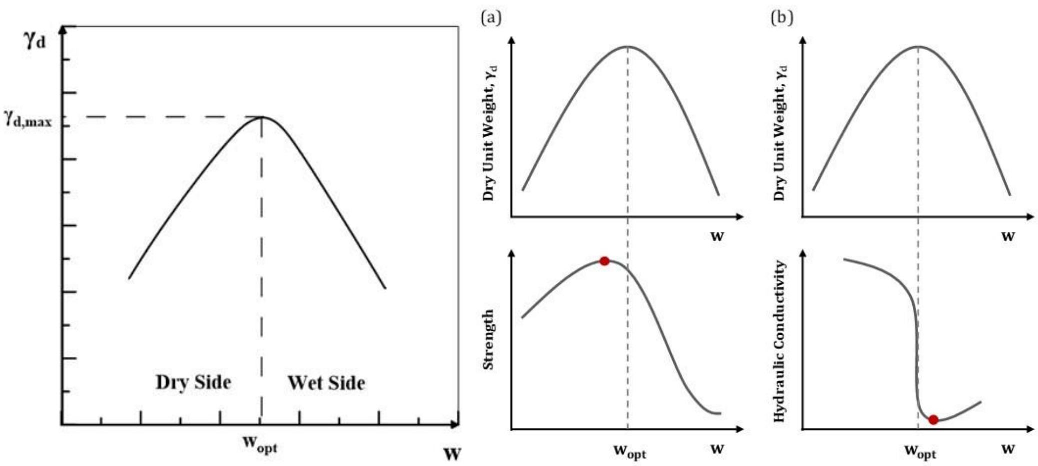


Figure 3.7 Correlation between the dry unit weight and the water content (after [79])

Table 3.5 Density Index (after [82])

Density Index (%)	Soil compaction	Angle of shearing resistance
0 - 15	Very loose	< 28
15 – 35	Loose	28 – 30
35 – 65	Medium	30 – 36
65 – 85	Dense	36 – 41
85 - 100	Very dense	> 41

3.2.5.4 Consolidation

When compression of soil (volume reduction) is in response to changes in effective stress, it is termed consolidation. This is the process by which the soil volume changes by the expulsion of water with consequent rearrangement of particles, crushing of particles and elastic distortion. It may occur due to the imposition of a higher continuous total stress under unchanged pore water pressure conditions or due to reduction in pore water pressure under unchanged total stress; or to a combination of these [83].

In an engineering situation, consolidation may result from the loading imposed by a building or a fill. If the deposit is saturated, the imposed stress causes an increase in pore pressure. Dissipation of this pore water pressure by flow of pore water into neighbouring soil masses, or into drains, leads to an increase in effective stress and a reduction in volume as the water is squeezed out and the particles rearranged. Thus, the reduction in moisture content results in an increase in strength [84].

Both compaction and consolidation bring about a closer arrangement of soil particles, but densification by compaction prevents later consolidation and settlement of an embankment. This does not necessarily mean that embankment settlements would be completely avoided; the weight of the embankment may anyway cause consolidation of compressible soil layers that form the embankment foundation [79]

Settlement

During construction, surface loads from foundations or earth structures are transmitted to the underlying soil profile. As a result, stresses increase within the soil mass and the structure undergoes a time-dependent vertical settlement [85] (Figure 3.8). The total settlement, S, is calculated as the sum of three components: Si is the immediate settlement, Sc is the consolidation settlement and Ss is the secondary compression settlement.

$$S = S_i + S_c + S_s$$

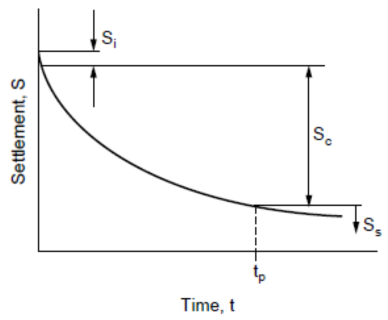


Figure 3.8 Time–settlement curve showing total settlement components (after [85]).

If saturated soil is loaded, as happens in embankment building, on overall increase in mean total stress occurs (Figure 3.9). In a fine-grained soil (like clay) the viscous resistance to pore water expulsion prevents the soil structure from rapidly contracting. In the short-term loading condition there is a change in effective stress due to shear strain only, together with an increase in pore pressure (Figure 3.9). With time this excess pore pressure is dissipated by drainage away from the area of increased pore pressure into the surrounding are of lower pore pressure unaffected by the construction. This flow of pore water causes a time dependent reduction in volume in the zone of influence the soil consolidating and the soil structure stiffening, giving rise to decreasing settlement and increasing strength. The minimum factor of safety occurs in the short-term undrained condition when the strength is lowest (Figure 3.9) [86].

However, the water flowing away from the originally stressed area can increase the pore pressure in an adjacent area. In that receiving area the effective stress would decrease and, hence, strength would also fall. For this reason, the response of actual embankment depends on many factors (including soil type, drainage routes and in-situ stress conditions) so that the pattern of stability illustrated in Figure 3.9 may be overly simplified.

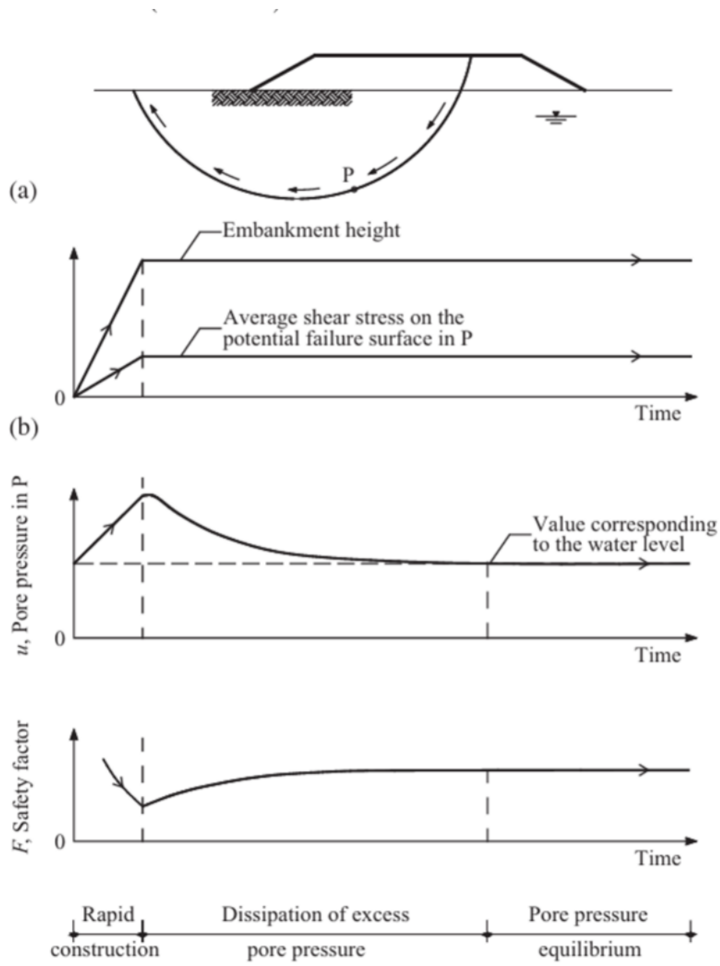


Figure 3.9 Embankment on soft clayey soil: a) layout and failure type; b) change of shear stress, pore pressure and safety factor during and after construction (after [86])

3.2.5.5 Permeability

Soil permeability is the property of the soil describing the ease by which it can transmit water through its pore defined as the velocity of flow under a unit hydraulic gradient [87]. The permeability of soils has a decisive effect on the stability of foundations, seepage loss through embankments and drainage of subgrades. Water flowing through soil may exert considerable seepage forces, which have direct effects on the safety of structures. Shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure, and thus increase in strength, is controlled by the soil’s permeability. Table 3.6 reports typical soil permeability and effective strength values, based on soil type.

Table 3.6 Effective stress strength parameters and permeability values for soils (after [73])

Soil	Permeability m/sec
Rockfill	5
Gravel	5 x 10-4
Medium sand	-
Fine sand	1 x 10-6
Silt	3 x 10-7
Normally-consolidated clay of low plasticity	1.5 x 10-10
Normally-consolidated clay of high plasticity	1 x 10-10
Over-consolidated clay of low plasticity	1 x 10-10
Over-consolidated clay of high plasticity	5 x 10-11

3.2.5.6 Shear strength

Strength is determined by carrying out test in which samples are subjected to increasing shear stress until it fails. Depending on the test (i.e. triaxial compression test, unconfined compression test, direct shear test etc.) compressive, shear and tensile strength can be defined. The boundary of permissible stress states defines a failure criterion. Failure for soils is defined either according to the undrained condition (i.e. in terms of total stress) or according to the drained condition (i.e. in terms of effective stress). The Tresca and Mohr-Coulomb criteria are used, respectively, to define these mathematically [73].

The Mohr-Coulomb criterion describes the shear stress, τ, as:

Equation 1

$$\tau = c' + \sigma_n' tan\varphi'$$

Where: σ_n' is the effective stress acting normal to the potential failure surface, equal to the difference between the total normal stress acting on the potential rupture surface under consideration and the pore water pressure $(\sigma_n - u)$; φ' is the angle of shearing resistance of the soil (the inclination of the failure envelope in Figure 3.10); and c' is the apparent cohesion of the soil.

The intercept of the straight line on the shear stress axis (Figure 3.10) is variously called cohesion (e.g. [10], paragraph 103), effective cohesion (e.g. [10], paragraph 539) and apparent cohesion (e.g. [88]). However, it usually arises as a consequence of forcing a straight line to fit through measured values of (τ, σ')

even though the data is most certainly representative of a curve. The resulting intercept depends on the range of stresses considered; therefore it is not a fundamental soil property.

Nevertheless, it is commonly used in engineering practice, including the rail industry as shown by the references given in the previous paragraph, as a measure of the stress that holds together particles within a soil. In this thesis, when soil properties will be considered for analysis, this property will be referred to as "apparent cohesion", to acknowledge that c' has no physical basis as it is just the computed intercept on the y-axis of the best-fit envelope [89].

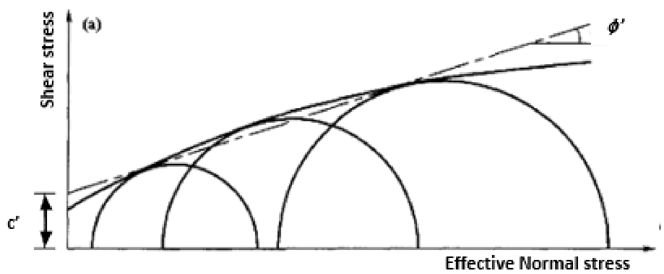


Figure 3.10 Failure criterion (after [89])

The following Table 3.7shows typical values of c' and ϕ' for different rocks and soils.

Table 3.7 Typical c' and ϕ' values for rocks and soils (after [90])

Classification	Friction Angle (°)	Apparent Cohesion [kN/m^2]
Hard rock	45	300
Soft rock	40	200
Hard soil	30	60
Medium soil	30	30
Soft soil	20	30
Very soft soil	20	23

The shear resistance of soil is the result of friction between, and the interlocking of, particles and possibly cementation or bonding at the particle contacts. Soil containing particles with high angularity tends to resist displacement and hence possess higher shearing strength compared to those with less angular particles.

The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain and direction of the strain. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, cracks, and cementation [90].

3.2.6 Soil mechanics principles for slope stability

In the classic limit equilibrium model used for most calculations of slope stability, the strength of the soil, and hence its resistance to failure, is given by the Mohr- Coulomb criteria seen before (Equation 1)

$$\tau = c' + (\sigma_n - u)\tan\varphi'$$

The value of τ thus depends on the value of u, the pore water pressure. If u increases, the [(σ_n - u) tan ϕ'] term will decrease, which may lead to failure of the slope in soils with low values of c' and ϕ' relative to the stress level applied by the slopes geometry [73].

In soils with high clay content, ϕ' may be lower than the angle of the slope and the influence of c' is significant. For low-clay-content soils and granular materials, c' will be low or zero and the strength will depend principally on ϕ'. Most over-consolidated clays, when freshly excavated, have negative pore water pressures, or low positive values. The soil, indeed, is not immediately able to drain water so that the appropriate volume change can take place [91]. Embankments constructed with these materials will be stable in the short term (about ten years) at slope angles of up to 1 (vertical) to 2 (horizontal) (1:2 or 26 degrees) if the material is well compacted, and 1:2.5 (22 degrees) to 1:3 (18.4 degrees) if loosely tipped [80]. Hence, at the construction stage the embankment appears stable, although this stability is sometimes marginal, and failures have occurred [92].

With time, the negative pore pressures decrease (i.e. get close to zero), or the positive pore water pressures increase, as water percolates through the fill. Table 3.8 shows typical as-built slope angles for clays most subjected to failure as results of Perry’s [4] survey on slope condition in the UK.

The majority of slips are caused by issues related to water; either surface or subsurface water (often referred to as groundwater). The presence of groundwater in a slope can reduce effective stresses when positive pore water pressures develop, causing a reduction in shear resistance. Groundwater can also increase the destabilizing forces acting on the slope due to the additional weight associated with a soil mass becoming saturated or seepage forces [4].

Table 3.8 UK Clays with high percentage of failure (after [73])

Geology	Percentage of failure	Predominant slope angle (v : h)
Embankment		
Gault Clay	8.2	1:2.5
Reading Beds	7.6	1:2
Kimmeridge Clay	6.1	1:2
Oxford Clay	5.7	1:2
London Clay	4.4	1:2
Cuttings		
Gault Clay	9.6	1:2.5
Oxford Clay	3.2	1:2
Reading Beds	2.9	1:3

An embankment performance assessment must include the influence that the pore water pressure regime has on the stability. According to Briggs et al. [15] the increase of pore water pressure refers to changes in effective stress due to surface water infiltration, imposed stress changes, or to the loss of soil suction within the embankments. As highlighted before, compaction and type of fill also have an important role in defining the performance.

No matter if the embankment is built following a modern or an old technique, an increase in pore water pressure in excess of historic values can potentially reduce the shear strength of the soil and trigger a slope failure. An increase in pore water pressure, within the soil near the slope surface, probably induced by rainfall, can trigger a shallow slope failure [15]. Increased pore water pressures can also trigger the deep-seated failure of embankments when weakened by progressive failure or when there are old shear surfaces from historic instability. For this reason, slope drainage has been recognised as an important slope remediation measure.

A study across all Network Rail territory between 2000 and 2003 [93] showed that the highest percentage of railway service delays (>8h duration), due to geotechnical causes, occurs in winter months (242 incidents) rather than in summer months (44 incidents). The delays were attributed to the ultimate failure of earthworks driven by elevated pore water pressures or washout due to the weather condition.

Supporting hydrological modelling by Scott et al. (2010) [94] showed that the increased permeability of railway embankments (often greater than 5 × 10-8 ms-1) compared to highway embankments (often less than 1 × 10-8 ms-1) made railway embankments more susceptible to rainfall infiltration and to increase in pore water pressure during wet winter weather. Surface run-off was more likely in highway embankments, which generally have, and/or are supplied with, specially designed drainage layers/trenches of lower permeability core.

3.3 Railway Embankment Performance Requirements

Embankments must meet specific performance requirements when supporting overlying railway transportation infrastructure [15]. Railway embankments must guarantee safe rides and track quality requirements according to the specified line speed and loading. The failure to meet performance requirements can range from an ultimate limit state failure, which may stop or severely restrict traffic flow, to a serviceability limit state failure which does not disrupt traffic flow but prevents the embankment from operating as intended [4].

Many railway embankments may suffer serviceability limit state failure, SLS, defined as "a state at which the condition of an asset would be such that the level of service it provided would be unacceptable: condition may be defined, for example, in terms of appearance or deflection under load". The loss of performance is generally slow and insidious and is associated with excessive movement rather than overall instability [95].

Ultimate limit state failure, ULS, is defined as “a state of collapse, instability or other form of failure of an asset that may endanger its users and/or the general public” [96] is less frequent but the consequences are dramatic, usually resulting in traffic being halted or severely restricted. Ultimate limit state failure may develop if serviceability limit state failure is not addressed.

Failure is said to have reached or exceeded an ULS when soil rupture is caused by shear stresses in the embankment exceeding the shear strength of the soil [97]. When such a situation arises, it is often necessary to impose a temporary speed restriction on railway embankments [98]. In such circumstances, the cost of delays and disruption can be high and, where safety regulations have been breached, the consequences of prosecution by the courts may be severe. When serviceability or ultimate limit states of an embankment are exceeded, and repair becomes necessary, significant costs may be incurred. These costs may extend beyond the direct costs of employing designers and contractors for repair works to the provision of access tracks, temporary speed restrictions, line and route closures and reduction in revenue.

So it’s much better to act to prevent ULS ever occurring than to fix a ULS after the event, for example, typical direct costs for London Underground Limited LUL embankment remedial works fell from £3000–5000 per metre to £1000–2000 per metre when part of a proactive maintenance and renewal strategy [98]. Planned repair and renewal clearly benefit owners as they permit more efficient use of resources and, importantly, predictability of expenditure and hence, greater control of costs.

3.4 Slope Failure Mechanisms

Several types of slope failure can affect infrastructure embankments, ranging from small-scale shallow translational slides to major deep rotational slips that run from the crest through the embankment and the underlying foundation material to emerge beyond the toe. Examples of common embankment failure mechanisms are shown in Figure 3.11.

All types of slope failure can be analysed in terms of soil mechanics principles which will be briefly described in Section 3.2.6.

The subgrade is the platform upon which the track structure is constructed with the main function of providing stable foundation. The influence of the traffic stress extends five meters under the sleeper [58]. This means subgrade is an important part of the structure, influencing both track performance and maintenance.

Although not specifically relating to embankments, according to Mott MacDonald [95], Li and Selig's consideration about subgrade failure mechanisms [99] would apply equally to embankment fill. The two most important failure mechanisms related to repeated train loading are:

- progressive shear failure (Figure 3.12) and
- excessive plastic deformation (Figure 3.11).

Plastic flow of the material occurs during progressive shear failure, as a result of distressing by repeated loading cycles. The soil then squeezes out and upwards where there is least resistance. This type of failure has been observed in fine soils with a high clay content causing a reduction in track levels. Operational maintenance, generally, can lift the track and eventually re-ballast, but this may leave a depression in the subgrade surface (a so-called "ballast pocket") which may lead to ponding of water with consequent larger-scale failure later.

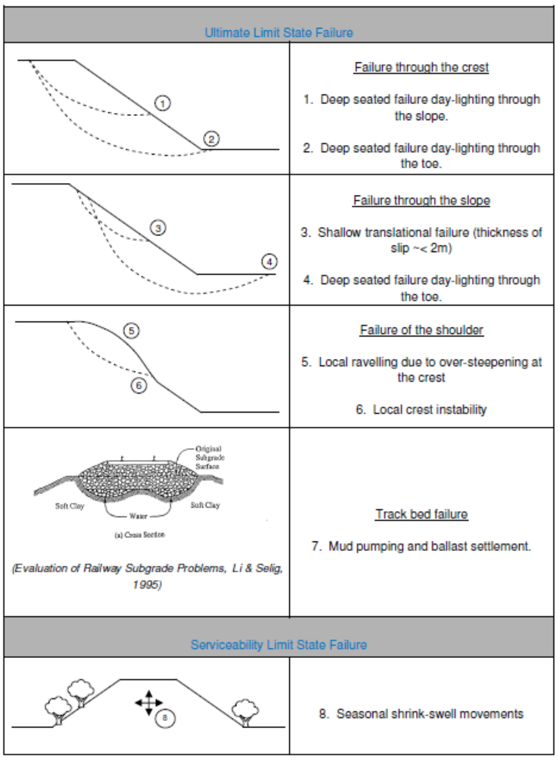


Figure 3.11 Summary of common failure mechanisms (after [10])

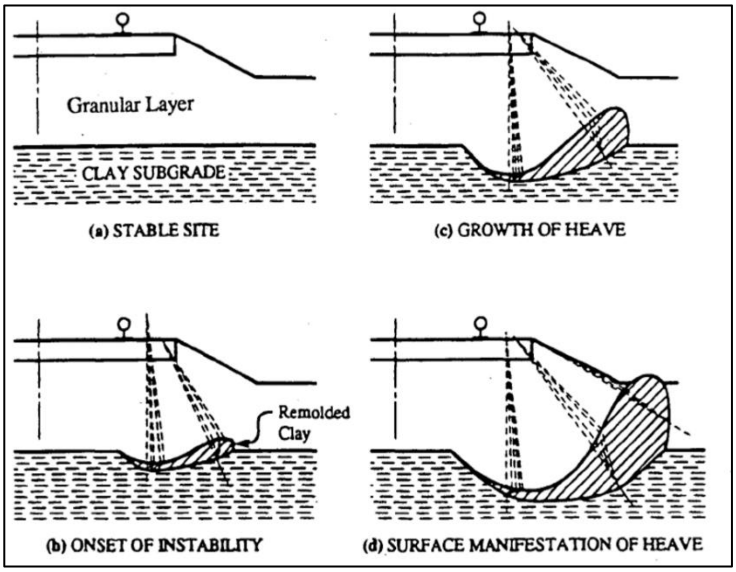


Figure 3.12 Development of progressive shear failure (after [10])

Also, as train loading distribution will never be uniform, due to irregularities in the structure and heterogeneous soils, depressions will develop in the ground, more in one location than another. The effect will be greater plastic deformation in some places than others, leading to rougher track profiles. Furthermore, water will be attracted to the resulting lower points, further softening them and, thereby, increasing further movements. In this way track quality deteriorates. Although this can be temporarily maintained by re-ballasting; with water still present in the soil, deterioration will continue.

Both these mechanisms, plastic flow and plastic deformation, will cause changes to the line and level of the track, potentially resulting in the imposition of speed restrictions and increased maintenance frequencies [95].

3.5 Geotechnical issues

This section sets out some of the main geotechnical issues affecting the performance and, hence, the maintenance of infrastructure embankments. The aspects covered are:

- Seasonal deformation
- Drainage
- Vegetation
- Wildlife
- Traffic damage
- External factors.

3.5.1 Seasonal deformation – Shrink and Swell

Soil properties change by repeated drying and wetting cycles due to environmental influence [100]. When the water attempts to leave the soil (drying), the water flows from the deeper regions to near ground surface due to capillary action caused by surface evaporation and plant-based evapo-transpiration; the air-water menisci act to resist the loss of water from soil pores causing soil to shrink. As a result, dry density, effective stress and strength increase rapidly. On the other hand, as water attempts to enter pores (wetting), the menisci act to draw water into pores, suction reduces, and effective stress and strength reduce rapidly. This happens without much flow of water; hence two different strengths can occur at similar moisture contents depending on whether the soil is wetting or drying. Sivakumar et al [101] reported from their experiments on compacted kaolin clay that the wetting and drying curves are significantly different as shown in Figure 3.13.

The process can result in the build-up or breakdown of soil particles and of the bonds between particles. The volume change of unsaturated soil due to wetting and drying causes enormous damage in embankments [102].

At the same moisture content, the drying phase has higher shear strength, lower stiffness, more ductility and contraction under shearing; on the contrary, the wetting phase has lower shear strength, high stiffness, more brittleness and dilation under shearing; that may explain the rapid decrease in stability of slopes after rainfall infiltration [103]–[105].

3.5.2 Cracking

The swelling and shrinking cycle cause problems to the subgrade bringing undesirable cracking and movement [106]. There are three factors responsible for the fatigue of expansive soils due to wetting and drying cycles:

1. reorganization of the soil particles leading to a progressively more intense destruction of internal structure,
2. loss of lateral confinement due to new cracks and
3. type of clay minerals presents in the soil.

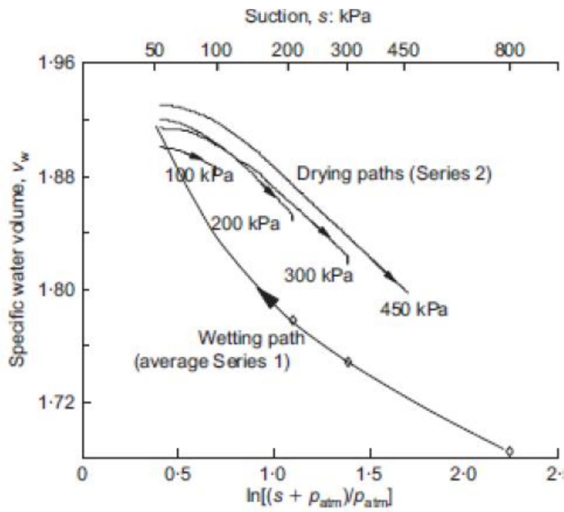


Figure 3.13 Suction characteristics during wetting and drying (after [101]**)**

During the dry spell, desiccation cracks often form. They can evolve in a wide range of forms and dimensions, and the continuous cycles of opening (when drying) and closing (when wetting) tends to modify the permeability of the soil.

Once a soil cracks, the cohesive strength is compromised and can't be re-bonded. As well as strength reduction, this leads to an increase in pores which in turn increase the permeability of the soil.

A particularly critical time is when a period of intense rain follows a long dry spell. Then desiccation cracks tend to have significant effects on the stability of embankments as, when rain comes, they can fill with water imposing hydrostatic loads on the crack walls that can have significant effects on the stability of embankments [107].

Analyses of datasets of embankment performance showed that train delay minutes owing to geotechnical causes during the dry summer months are primarily located in areas of high-plasticity soil, with almost two orders of magnitude more delays being attributed to these soils than areas of lower plasticity soils. This was linked to the shrink-swell deformation of embankments, leading to track defects and hence speed restrictions [107] (Figure 3.14).

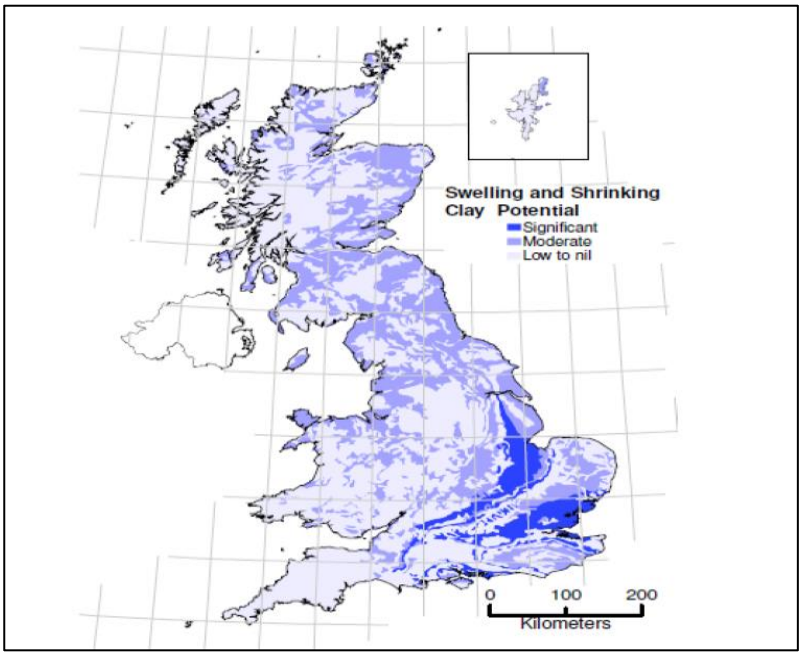


Figure 3.14. Swell and shrink clay potential map, based on volume change potential (after [107] **)**

3.5.3 Drainage

The main purpose of the drainage is to divert the water from the subgrade beneath track in order to keep it safely supported by its subgrade. This has relevance when considering track performance and maintenance [39], [108], [109]. The stability of the earthworks is dependent to a large part on the drainage system, which is, in the UK, also often over 150 years old and was installed to a pre-set 'design', which did not take account of catchment areas, run-off and water flow. The drainage system was also not "designed" as a slope stabilisation measure. Replacement over the years has generally been on a like-for-like basis, so the drainage system has not been enhanced.

The achievement of appropriate drainage does not involve simply digging a cross trench and draining the water from the track. If not used properly, inappropriate drainage can allow water into the subgrade, thereby causing more softening rather than preventing it.

The railway drainage system (Figure 3.15) includes all components designed to collect surface and groundwater which runs towards, falls onto or issues from the railway asset, and deliver it to a suitable outfall, whether that be a river or stream, a public sewer or a soakaway. The drainage assets are defined as:

- Earthwork drainage (of both surface and groundwater)
- Track drainage (of both surface and groundwater)
- Structures drainage in relation to tunnels, culverts etc

Assets of various types within each group have a similar form and function, and similar mechanisms of degradation.

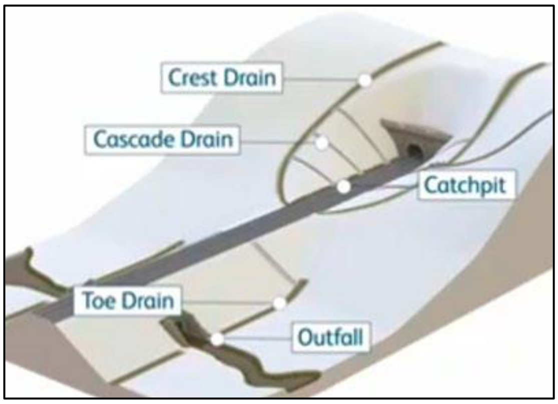


Figure 3.15 Drainage Systems (after [10])

The purpose of track drainage assets is to remove water from the track support system (see typical cross section in Figure 3.16). Track drainage is not required where the infiltration capacity of the support system exceeds the rate of infiltration from all sources of water. For much of the railway network, track drainage relies on infiltration of rainwater into the underlying ground and so infiltration is an important component of track drainage. Ballast is provided to give support, load transfer and drainage to the track and thereby keep water away from the rails and sleepers. Track drainage can be differentiated between track drainage assets and off-track drainage assets as illustrated in Figure 3.16.

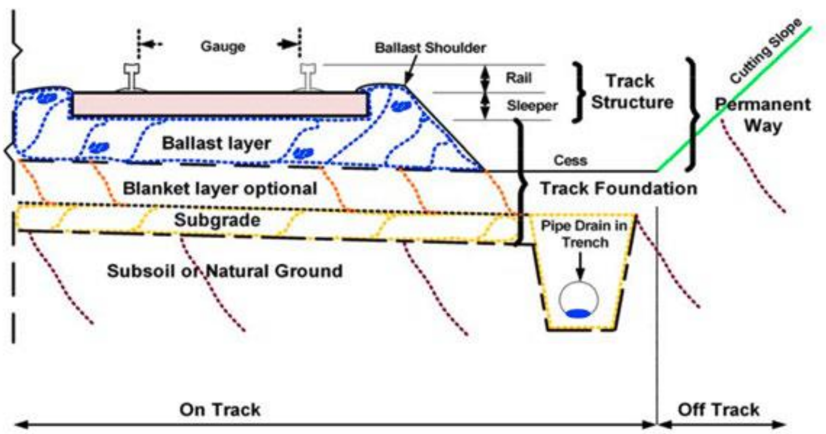


Figure 3.16 Typical track drainage cross section (after [10])

Earthwork drainage forms the majority of off-track drainage assets (Figure 3.17). Drainage ditches are often added along the edges of cuttings and embankments. In the UK, fences are always provided along the boundary line of the railway to protect the public from wandering onto the track. Even so, there are a few accidents every year when trespassers are killed or injured by trains or electric conductor rails.

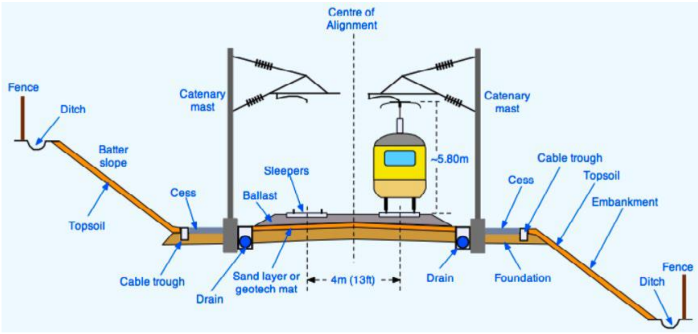


Figure 3.17 Off-track drainage assets (after [10])

3.5.4 Vegetation

Vegetation on an embankment slope is generally the most aesthetically pleasing form of surface protection. Trees covering many of the UK's railway earthwork slopes provide a natural habitat for wildlife and biodiversity while creating a visual and acoustic screen for residential areas. Some types of vegetation can also improve the stability of an embankment to a shallow depth.

From an engineering perspective, trees aid slope stability through mechanical root reinforcement and by the establishment of soil suctions. However, trees can also cause serviceability problems such as excessive track movement due to root growth and due to evapo-transpiration driven soil shrinkage, resulting in delays for passengers and a substantial maintenance cost for infrastructure owners [94], [110]–[112].

Originally, engineering practice saw vegetation as a hazard to be removed. With improved knowledge, positive key impacts on engineering performance due to changes in slope hydrology [93] and mechanical reinforcement of roots can now be appreciated and exploited where possible.

The Canopy can intercept rainwater and so prevent infiltration and minimise slope erosion, while roots absorb water from the slope. These effects can initially be seen as beneficial as they will reduce pore water pressures in the slope and increase stability. However, in addition, roots can provide higher permeability pathways, increasing the amount of water entering the slope [113].

Change in the soil water content, as a result of seasonal water uptake by roots, causes seasonal volume changes in the soil and consequent three key effects:

- Deformations to the slope.
- Cracking of the slope surface in the summer, leading to infiltration pathways available for the autumn.
- Strain softening of high plasticity materials in the slope as a result of shrinkage and swelling which may lead to progressive slope failure.

Vegetation may also prevent collapse; the tensile action of the roots can reinforce the slope and thereby inhibit soil movement. So, mechanically, roots provide reinforcement that can be seen as an additional apparent cohesion of the near-surface material similar to the one produced by fibre reinforcement. This reinforcement will also help prevent erosion [93].

The amount of mechanical reinforcement provided by a root system strongly depends on the spatial distribution of the roots, on their mechanical strength and it is also likely to depend on the soil and interface properties. However, tall trees should be avoided where they may impose high loads as a result of wind loading or their own weight.

Different species will provide different levels of mechanical support through root strength, spatial distribution and by extracting water over different zones of influence [49].

Large mature trees provide valuable screening benefits. However, if such trees are classified as high-water demand, then the impacts of deformation due to shrinkage and swelling are likely to be significant. In contrast, low water demand trees are likely to have a minor impact on slope deformation.

Past experiences [93] show that the risk of excessive seasonal slope and track deformation is relatively high when high water demand trees are located close to the crest, while the risk of slope instability is higher for grass covered slopes than those covered by trees. The risk of instability is relatively low if high water demand trees are located close to the toe of the slope.

Finally, wildflowers and grasses tend to be less deeply rooted than trees or shrubs and will therefore only affect the hydrology and mechanical properties of the near-surface soil. However, such species are likely to be effective in preventing erosion and surficial failure because of their interconnected, fibrous root system that will mechanically improve the top 20–30 cm of the soil [93].

It is important to underline that vegetation may take several seasons to become fully established and the reduced effect on embankment stability during the interim period should be considered. Wherever possible, mature vegetation should be retained, as this assists in reducing the visual impact of remedial works until full vegetative cover is established. Where erosion in this interim period is potentially a problem, temporary protection to the slope may be required, and products such as coir or geo-synthetic blankets impregnated with seed can be used [114].

3.5.4.1 Influence of vegetation on shrink and swell

The purpose of an investigation into the effect of vegetation on slope stability and behaviour is to estimate changes in soil water content that are related to the changes in the volume of water stored within the soil. This water content is usually expressed as the soil moisture deficit (SMD), calculated in mm as a volume of water per unit area.

It is possible to estimate SMD through the full water balance [114]:

$$\sum(R - RO) - \sum ET + SMD - RE \approx 0$$

where R is the rainfall, RO is the runoff, ET is the actual evapotranspiration, SMD is the change in stored water within the soil, and RE is the net recharge from the surrounding soil. Rainfall is simple to measure, and it can also be very site-specific since records are available for the UK. Runoff is likewise site-specific and measurable [115]

$$Q = CIA$$

Where:

Q = Peak rate of runoff in cubic feet per second

I = Average intensity of rainfall in inches per hour for the time of concentration (Tc) for a selected frequency of occurrence or return period

C = Runoff coefficient, an empirical coefficient representing a relationship between rainfall and runoff

A = The watershed area in acres

Tc = The rainfall intensity averaging time in minutes, usually referred to as the time of concentration, equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.

Evapotranspiration is a function of the interactions between the elements of the plant–soil–atmosphere system. It depends on plant type, climate, soil characteristics and soil moisture conditions. It is more difficult to quantify owing to the variability of climate, soil and plant types [116].

A soil with zero SMD is at ‘field capacity’ (the water content the soil can hold against gravity). For many soils a value of SMD = 0 mm usually occurs 1–2 days after rainfall and corresponds to a suction of about 10 kPa [117]. The soil moisture deficit changes dynamically in response to the inflows and outflows of water in the field, and if the SMD calculation covers too long a period, the results will be meaningless.

Glendinning et al.’s study [93] shows that winter heave of the embankment occurs during periods of low SMD (zero) while embankment settlement occurs during periods of high SMD (>300 mm) in summer months. The study shows that shrink and swell process is also influenced by vegetation. The amplitude of shrink-swell movement of parts of slopes adjacent to Oak and Poplar tree covers (SMD 50–55 mm) was an order of magnitude greater than the adjacent parts of the slope covered by grass (SMD 5–8mm). Piezometer measurements also indicate higher suctions (up to -90 kPa) and greater range of seasonal pore water pressure variation (80 kPa) for parts of the embankment covered with the Oak and Poplar tree than for areas with grass cover (20 kPa variation). Glendinning [93] attributed the high effective stress (up to 500 kPa), as measured in undisturbed samples of Gault clay embankment fill, to high suctions induced by adjacent Oak and Hawthorn trees. Damaging deformations due to seasonal shrinkage and swelling can be reduced by removing high water demand tree species (Oak, Willow, Poplar and Hawthorn) from embankment slopes [118].

Reliable analytical descriptions of the effects of trees on embankment movement are not yet established.

3.5.5 Wildlife

As well as becoming corridors for vegetation, infrastructure embankments can become havens for wildlife, in particular small burrowing rodents, amphibians, reptiles and mammals [119].

In nature, there are several types of rodent animals that damage embankment and cause various seepage or stability related problems. Almost all types of rodents affect these structures and their components in an undesired way.

The common ones are rabbits, badgers, beavers and muskrats (the last two less common or absent in the UK). These types of burrowing animals have different effects on embankment due to their varying digging properties. These rodents produce different hole diameters, lengths and depths, and the influence of their actions on the embankment varies.

Numerous states in the United States reported failures in earthen structures due to nuisance wildlife intrusions. Bayoumi [120] on wildlife living in earthworks in Connecticut and Northern states shows that muskrats can dig large burrows up to 3 m below the water surface on the upstream face of the dam. Moreover, their digging direction changes according to the location of the phreatic surface of the seepage. When the phreatic surface level rises, they dig towards the upward direction. Similarly, beavers are active in the upstream side of the earthwork. They have a relatively large body; their length varies between 0.60-0.75 m whereas their weights are around 20 to 27 kg. They can dig holes between

0.3 and 1.2 m below the water surface with a diameter around 30 cm. Typical burrows of badgers basically have a plugged entrance. In contrast to muskrats and beavers, badgers dig for shelter from the downstream side of an earthwork. Their burrow lengths range from 1.5 m to 9.0 m with a diameter of 20 to 30 cm.

The presence of wildlife may affect both routine maintenance and remedial works activities.

The Wildlife and Countryside Act (as amended) 1981 is still the major legal instrument for wildlife protection in Britain. This legislation covers the protection of a wide range of protected species and habitats and provides the legislative framework for the designation of Sites of Special Scientific Interest (SSSIs) [121].

It is illegal to destroy the nest of birds, and hence works need to be programmed to avoid nesting periods in the spring. Other wildlife to consider is amphibians and reptiles. Remedial works have to be planned according to when it is possible to move them and obtain access to the slope [120].

3.5.6 Traffic damage

As seen so far, owing to their complex construction history, together with a lack of understanding of their mechanical behaviour, UK railway embankments have inherited several factors which make them vulnerable to damage from railway traffic loading. Such damages would manifest themselves through increased maintenance and poor trackbed performance.

The passage of an individual train over a railway embankment will induce both elastic and plastic deformations within the earthwork. Although the embankment deformation is largely elastic and fully recoverable, irrecoverable plastic strains will accumulate gradually over a large number of cycles. The magnitude of permanent deformation increases with train axle load [122].

RSSB research project [123] on the effect of railway traffic on embankment stability has highlighted the potential mechanisms for failure, although the evidence of embankment failures around the UK railway network does not provide a link with failure due to increased railway traffic loading. According to the study, it is likely that the development of embankment failures induced by railway traffic loading will be a slow progressive process which will initially become evident through increasing frequency of track maintenance. In conclusion, without a reliable means of prediction, it is possible that works to strengthen the track sub-grade to carry increased railway traffic loading, will not address the root cause of the problem within the embankment below the track.

3.5.7 Human activity

Human activities frequently affect the stability of infrastructure embankments. Trenches dug at the toes of embankments, either to install services or to be left open for drainage, have a major adverse effect on the stability of the embankment. Similarly on a larger scale, excavations for quarrying, eg brick, sand and gravel pits, can seriously destabilise a slope [124]. Even if the slope does not fail, some movement is likely to occur, leading to settlement at the crest and weakening of the embankment. There is increasing business pressure to install services on the shoulders of embankments. Excavations for these services, on the edge of the embankment crest, may themselves be unstable and lead to minor failures. If services have to be installed in embankments, they should be kept as far back from the slope as possible [10].

Problems can arise where a new embankment is to be constructed alongside an existing one.

Foundation preparation is required to make sure that the new embankment is properly benched into the old one and that unacceptable differential settlement or lateral movement does not occur. This type of construction is common in many recent railway improvement schemes, as the emphasis has changed from constructing new routes on green field sites to maximising the use of existing corridors. Problems are likely where the modern embankments are to be built adjacent to older one in areas of soft foundation soils. The imposed loading from the new structures could cause a large increase in pore water pressure (and consequential reduced shear strength) in the ground under the old embankment and could also cause settlement of the old embankment as a result of the settlement trough produced by the new embankment [123].

In some cases, vandalism can reduce embankment stability, for example:

- cutting of materials, such as geogrids, used to reinforce railway and highway slopes.
- arson, particularly setting fire to vegetation.
- fly-tipping, that can lead to an unsightly appearance and can block drainage.

CHAPTER 4. AECOM'S EMBANKMENT INSTABILITY METRIC PROJECT

4. AECOM's Embankment Instability Metric project[1]

As highlighted in Chapter 2, a geotechnical asset management system, when tailored to the needs of the industry and adopted in a proactive way, may provide several benefits to an infrastructure asset owner in terms of money, safety and reliability. To guarantee good system performance, geotechnical asset management (GAM) aims to reduce uncertainty through informed, data-driven decisions and optimisation of resources. Track geometry data has been routinely collected by Network Rail, over many years, to identify track defects and subsequently to plan track maintenance interventions. Additionally, in 2018 Network Rail commissioned AECOM to undertake a study, described in this chapter, to investigate the use of track geometry data in the detection of embankment instabilities. In this study, track geometry data for over 1800 embankments were processed and parameters offering the best correlation with embankment movements were identified and processed by a novel algorithm to generate an embankment instability metric (EIM). With the study, AECOM successfully demonstrated that the instability of railway embankments is clearly visible in track geometry data and the metric gives an indication of the worsening of track geometry, that is likely due to embankment instability.

The project, which is entirely property of AECOM, is described in this chapter as the metric upon which this thesis' project is founded. The EIM is used in this thesis as a measure of track geometry degradation due to embankment instability. It is, thus, a crucial input for the work undertaken in this research work, however, the author of this thesis gave no contribution to the EIM study. The author had only access to the data results of the study; specifically, 1 metric value was provided at each asset per each year considered for the analysis.

[1] This chapter is after: Detecting Embankment Instability Using Measurable Track Geometry Data; D. Kite, G. Siino, M. Audley; Infrastructure 2020.

4.1 Track Recording Vehicle TRV

In the past decade, various studies [69], [70], [125] have been carried out on developing techniques to determine the individual elements of track geometry using inertial sensors fixed to in-service trains. Measured vehicle vibration signals are influenced by track features such as rail irregularities, corrugation, vertical alignment, track stiffness, changes in rail bending characteristics due to the presence of welds, and cracks.

Various techniques are used to monitor the track for these defects. Most of those employ a frequency-based analysis so that short and long-wave defects can be differentiated [36].

Before starting a run, track features such as track identification and mileage are input either manually or automatically into the TRV. Basic location is obtained during the survey journey either automatically, by use of a satellite positioning system, or manually using mile posts. As well as track geometry, other parameters are also recorded, such as the distance run by the TRV, in order to aid geo-referencing of recorded measurements (essential in tunnels, for example, where satellite positioning is impossible). Twist, curvature, horizontal alignment and vertical alignment are either directly measured or calculated by the TRV.

Data are often processed on board the TRV: graphs are produced, track quality indicators are calculated, and line parameters are drawn up. The data processing involves calculation of standard deviation with a base-length, where the base-length depends on the intended use [126] These standard deviations are calculated for the twist, horizontal alignment and vertical alignment parameters and are calculated for all the collected TRV data.

Outputs from TRVs are used to prioritize investment in track repairs and renewals, to plan track maintenance activities and to confirm (or not) whether the track is safe. In order to measure the parameters under track loaded conditions, the sensors, placed under the vehicle's frame, are positioned as close as possible to one of the vehicle's loaded axles to meet the measurement conditions indicated in EN 13848 [71]. Different track recording vehicles should be calibrated and standardised so as to give comparable results, in the same format, when measuring the same track under the same conditions.

4.2 Background to the analysis – Sharpe and Hutchinson's Study

In 2014, AECOM undertook a study for Network Rail, where the objective was to review TRV records on selected sites on the Midland Main Line, to assess whether geometry data could be used to detect the early stages of embankment failure. The sites were identified by the Geotechnical Route Asset Management team, as earthwork sites susceptible to failure. Failures had occurred during the heavy rainfall of Winter 2013/14 in that area. The study was completed by Dr Phil Sharpe (AECOM), who worked closely with Dave Hutchinson (Geotechnical Route Asset Manager, Network Rail), and later published a joint paper on the methodology [127].

Track geometry records, obtained on a monthly basis, were processed, allowing trends to be identified in data collected for the period between November 2010 and May 2014. For the purposes of using track geometry data to detect ground movements, the most useful parameters appeared to be vertical alignment (Top) and lateral alignment (Align), as these can be directly attributed to earthwork movements. It was also found that a combination of the two parameters was representing the rotation of the track and could be used to indicate whether deterioration in track geometry was due to earthwork movement: lateral alignment (Align) SD and the difference between Left (rail) Top SD and Right (rail) Top SD (referred to as differential Top or dTop).

The use of Standard deviation SD is due to the poor quality of data from TRV runs [Track-Recording-And-Usage]. Also, location information between TRV runs is extremely poor, which is why the track geometry runs need to be aligned with each other (further explanation in Section 4.3.1). The calculation of the SD (after the run alignments) helps to smooth the data out, which mitigates against the poor longitudinal alignment of track geometry runs and helps with trending the runs over time. In an ideal world, where perfectly aligned data exist, it would be possible to move away from SD.

When embankment instability starts, the track can be expected to follow the movement of the earthworks as shown in Figure 3.3.

In fact, earthwork movements appear to be characterised by excessive deterioration in both lateral alignment and difference in vertical alignment. Looking at the train direction of travel (on the UK network trains typically travel on the left hand tracks), a shallow rotational slip would be indicated when the Left Top SD is greater than the Right Top SD; in this case the dTop will be a positive value. A deeper rotational slip, however, would be indicated when the Right Top SD is greater than the Left Top SD, thus the dTop will be a negative value (Figure 4.1). In both cases, horizontal movement away from the embankment centreline (i.e. for UK railways a leftward movement) would also be indicated.

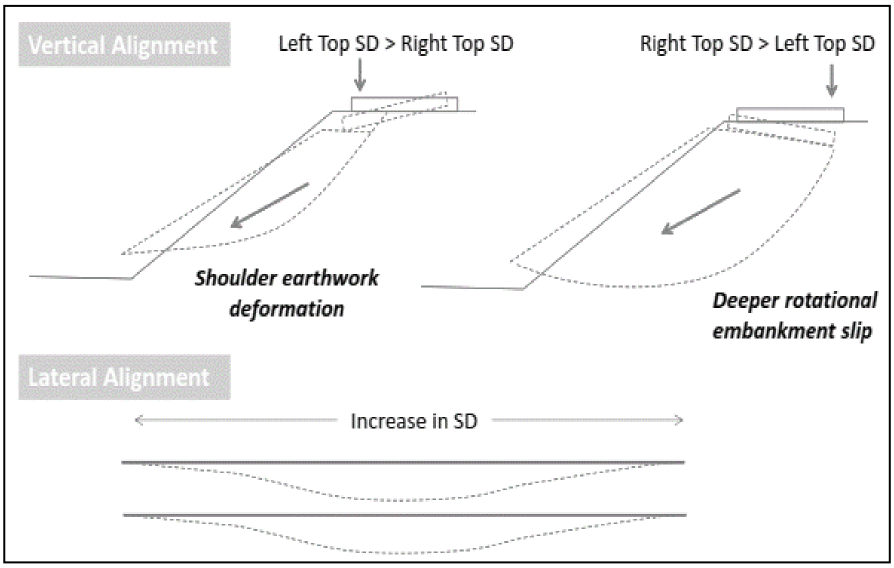


Figure 4.1. Effect of typical rotational slip-on track geometry (after [6])

As a result, the study suggested that the track geometry measurements contain strong indicators of earthwork movements and, for those assets deemed to have failed in 2013/14, there was evidence of instability in the track geometry data at least three years before the point of failure. A change in vertical alignment on one or both rails is shown, accompanied by differential settlement between the two rails, and lateral track movement associated with movement along the slip plane. Major areas of earthwork movement were known in advance, but there were many areas, not previously identified as experiencing movement, that were showing indicators of minor earthwork movements [127]. By taking an overview of the whole range of earthwork movements observed in the data, Sharpe and Hutchinson [127] proposed a classification (Table 4.1) of the severity of earthwork movements from "Negligible" (where any track roughness would be addressed during the course of routine maintenance and would not therefore be identified as an earthwork problem) to "Failure" (corresponding to the point at which there is a serious earthwork issue that requires regular maintenance):

Table 4.1. Classification of severity of earthwork movements (after [127]**)**

Negligible	<1 mm deterioration in SD per year
Minor	1 to 2 mm deterioration in SD per year
Moderate	2 to 4 mm deterioration in SD per year
Failure	>4 mm deterioration in SD per year

A reasonable interpretation of the study outcomes was that if a site was presenting both dTop and alignment deterioration, the greater of either of these parameters should be used to classify the severity of earthwork movement.

4.3 Embankment Instability – Modelling project

In 2018, Network Rail instructed AECOM with an embankment instability research project, described in this chapter, after releasing a challenge statement titled "Detection of Geotechnical Asset Failure by Means Other than Train Drivers or Lineside Staff" [128]. The challenge statement set out the research needs related to the improved use of analysable datasets to assist with the monitoring of geotechnical assets, particularly embankments. Furthermore, the challenge statement also suggested the use and integration of datasets from different disciplines, with geotechnical datasets and referenced track geometry data as a potential data source.

The aim of the embankment instability project was to refine and test the concept of using track geometry data to perform an analysis of embankment instability on a large sample of assets presenting known issues and so to develop an algorithm to quantify the level of instability. The output of the algorithm has been used in a parametric study to establish whether there is enough confidence to consider wider use of this technique as a risk marker for the prioritisation of asset vulnerability to failure, driving further inspection or prioritisation of other remedial actions.

AECOM were provided with the locations of the embankment assets identified for either renewal or refurbishment during Control Period 6 (CP6) from 1 April 2019 to 31 March 2024. The assets per category, identified along nine routes, in total are: 274 “renewal”, 783 “refurbishment”, 577 “maintain” and 38 “mitigation only”. The analysis, though, could not be completed for some sections of track, with the main limitations being the quality and frequency of the track geometry data recorded by the TRVs.

4.3.1 Gathering Data

Approximately 60,000 track geometry runs, provided by Network Rail, containing approximately 5 billion data points were analysed for the embankment instability project. Some of the downloaded track geometry runs were found, for no clear reason, to contain no data. In order to maintain integrity of the analysis, geometry runs containing no data were manually removed from the analysis exercise.

Moreover, the positional element of the individual track geometry runs were not accurate enough to carry out long-term trending in their original state. Hence, to allow trending to be undertaken, each of the track geometry runs required aligning through a semi-automatic web-based tool built by AECOM. The semi-automatic tool displays all the available geometry runs for a given section of track in a web browser. Next, a user identifies and selects the location of a rail weld that can be seen in each of the individual geometry runs—rail welds show up on the geometry trace with a recognisable signature. Finally, once the user has selected the same weld throughout all the geometry traces, the alignment tool stretches and compresses the data so that all the welds line up.

Figure 4.2 shows the difference between the track geometry runs prior to and following the alignment procedure. Once the raw track geometry data has been aligned, the data is processed (at 10-yard intervals) and transferred to AECOM’s linear asset management tool TAMP [129], so that it can be visualised and analysed.

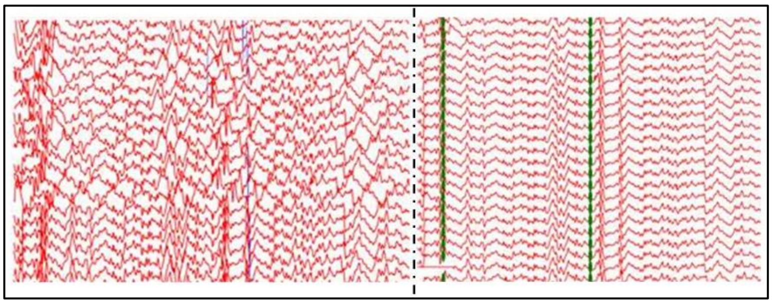


Figure 4.2. Track geometry data, prior to (left) and following alignment (right) (after [130]**)**

For the initial study (Sharpe, 2014), a base-length of 36.6 m (equivalent to two rail lengths) was used for the computation of SD (i.e., the total length over which points are used for the SD calculation). In the embankment instability project (2018), a base-length of 18.3 m (20 yards, equivalent to one rail length) was used for the SD base-length. This provides an objective measure to determine the rate of deterioration of track geometry by limiting the influence of dipped welds into the readings.

Figure 4.3 shows how, even though the development of key indicators of earthwork movement is evident for both base-lengths, SD data processed using the 18.3 m base-length give more localised detail, while the 36.6 m base-length data show a smoothed profile. Hereafter, these base lengths are, for brevity, referred to as 18m and 36m respectively.

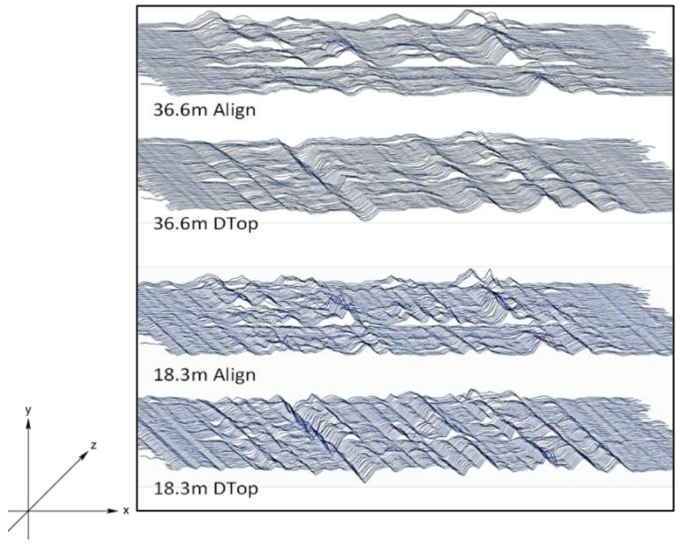


Figure 4.3 Visualised track geometry data comparing 36.6 m and 18.3 m SD base lengths (after [130]**)**

In Figure 4.3 each grey line represents a new TRV transit, with the oldest transit plotted at highest z and the most recent at the lowest z. The following parameters are used for embankment analysis. Two of these parameters are composite parameters, produced using a combination of parameters that were output from the runs.

Align = Alignment 18 m SD

Top = Minimum of (Right Rail Top SD) or (Left Rail Top 18 m SD)

dTop = (Right Rail Top 18 m SD)–(Left Rail Top 18 m SD)

4.3.2 Methodology

This paragraph describes the conceptual development of the methodology for the embankment instability project and discusses its assumptions and limitations.

An algorithm, coded in SQL, was developed to calculate a new metric. The algorithm was underwritten by the main assumption that track geometry cannot improve without a maintenance intervention. In addition, the algorithm was developed using the 35m wavelength raw data alignment (Align) and the differential top (dTop) parameters only provided by NR [126] based on the previous findings [127]. The 35m wavelength raw data filtering is used by NR to filter the raw data collected from the track recording vehicles, which would otherwise be unsuitable for use. Indeed, raw data are too affected from track geometry defects (roughness) and the filtering process eliminates ground-borne vibration and noise. This filtering is achieved by employing a Butterworth filter [126] and the filtered data are used as input in the algorithm.

Top is not a direct input for any iterations of the algorithm; as a result, this study only considers the deterioration of Align and dTop as indicative of embankment instability. Calculating the deterioration of the Align SD follows a logical statement: increases in Align SD values between measurement runs is considered deterioration, while decreases in Align SD values between measurement runs infers improvement, assumed to be due to track maintenance activity (although this can in some cases be due to seasonal effects). For the purpose of calculating an embankment instability metric (EIM), periods of deterioration and improvement are based upon the behaviour of the Align parameter only, as it is not possible to infer deterioration from dTop alone. Therefore, dTop is assumed to deteriorate only when Align SD deteriorates and improve only when the Align SD improves.

The algorithm detects a temporal variation in the deterioration of track geometry by breaking down the track geometry data into year-long periods. The year-long deterioration periods used do not follow a calendar year. Through the analysis of track geometry data, it has been noted that embankment behaviour occasionally shows seasonal variability. Hence, to encompass one full dry season and one full wet season for each period analysed, it was decided to run the annual period of analysis from 01-May to 30-April. This was named a "deterioration year" (DetYr).

4.3.3 Algorithm and Development of Two New Metrics

As reported in the previous paragraph, the algorithm only considers deterioration in the dTop and Align parameters. Starting from data filtered at 35m wavelength (Butterworth filter [126]), it combines the deterioration rates found in the 18 m dTop SD and 18 m Align SD parameters by averaging them, and outputs two metrics initially referred to as AvGrad18 and MaxGrad18. MaxGrad18 is the maximum combined deterioration rate found between two sequential recording runs during a given deterioration year, and AvGrad18 is the average combined deterioration rate over the whole deterioration year.

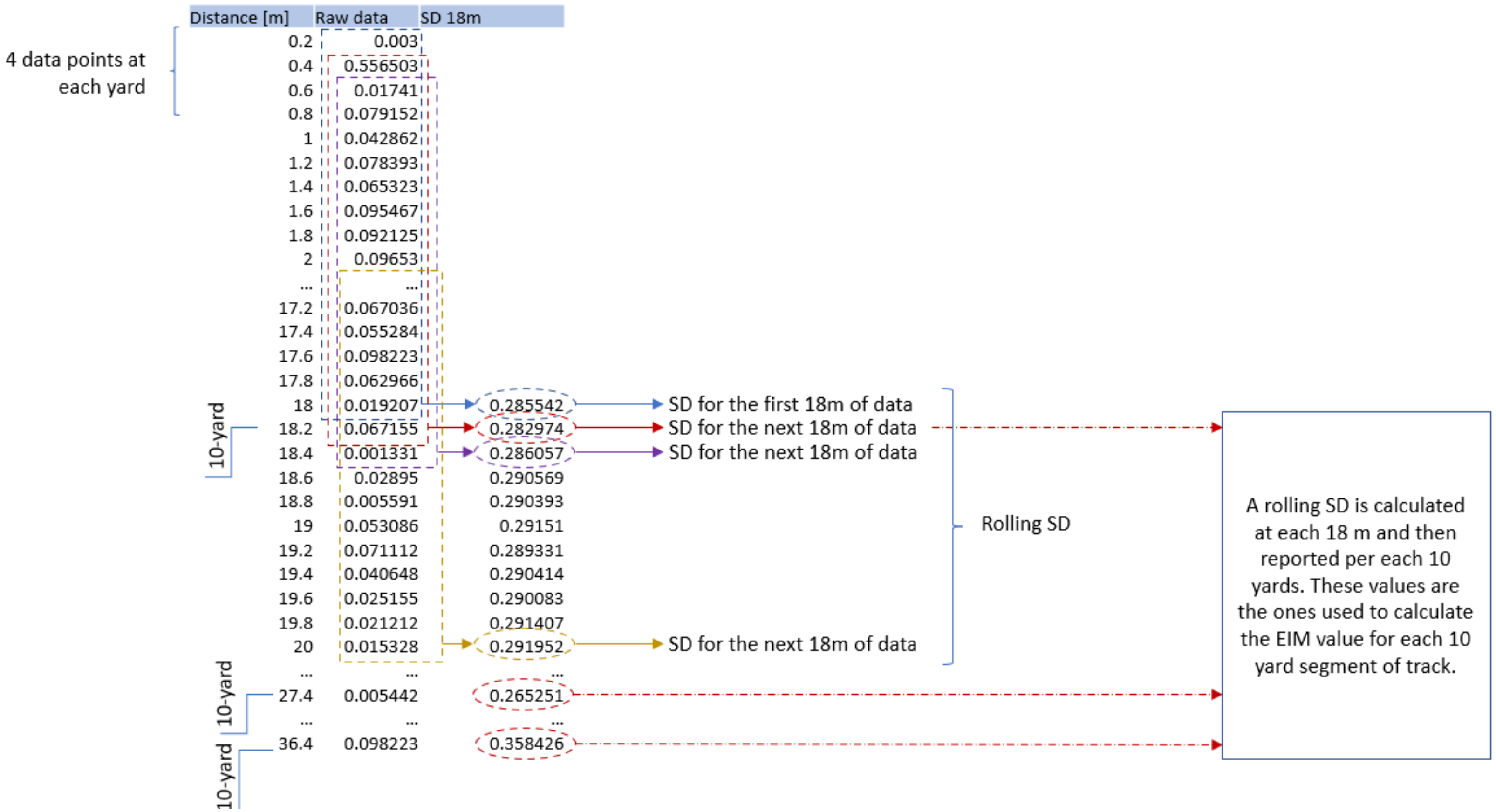
4.3.3.1 Calculating the metric

Both MaxGrad18 and AvGrad18 are calculated on a 10-yard basis.

Figure 4.4 schematically shows the steps to be followed for SD calculation for the 3 measures that are input into the algorithm: Alignment 18m SD, Right Rail Top 18m SD, Left Rail Top 18m SD.

1. NR provides one datum every 20cm, hence four data points are available for each yard. The rolling SD is calculated for each (rolling) 18m (SD 18m).
2. The SD rolling value corresponding each 10-yard measurement is input into the algorithm for calculating the EIM value for the specific 10-yard segment of track, at each run (at least 2 runs per year).
3. The EIM is calculated by averaging the deterioration over a year. The reason why the matric is averaged lies in the high volume of data. The embankment is, by definition, a 110-yard asset which means 11 EIMs each year, therefore a 10-year analysis results in 110 EIMs. From the analysis and interpretation point of view, this number of data is impractical for the final user to interpret. Averaging the data overtook the issue and therefore delivered a more practical value to the client (i.e. Network Rail).

It's to be noted that the data in the scheme (Figure 4.4) is randomly generated for demonstration purpose only; the scheme and the steps above are generic and apply for calculation of the 3 measures. In a real spreadsheet, column 'Raw data' would contain Alignment, Right Top, Left Top and column 'SD 18m' would respectively show values for Alignment 18m SD, Right Rail Top 18m SD, Left Rail Top 18m SD.



Distance [m]	Raw data	SD 18m
0.2	0.003	
0.4	0.556503	
0.6	0.01741	
0.8	0.079152	
1	0.042862	
1.2	0.078393	
1.4	0.065323	
1.6	0.095467	
1.8	0.092125	
2	0.09653	
...	...	
17.2	0.067036	
17.4	0.055284	
17.6	0.098223	
17.8	0.062966	
18	0.019207	0.285542
18.2	0.067155	0.282974
18.4	0.001331	0.286057
18.6	0.02895	0.290569
18.8	0.005591	0.290393
19	0.053086	0.29151
19.2	0.071112	0.289331
19.4	0.040648	0.290414
19.6	0.025155	0.290083
19.8	0.021212	0.291407
20	0.015328	0.291952
...	...	...
27.4	0.005442	0.265251
...	...	...
36.4	0.098223	0.358426

Figure 4.4 Schematic steps to calculate rolling SD as input of the algorithm

4.3.4 Numerical Simulation

Track geometry must be inspected frequently to determine track condition, to ensure safety and suitable ride quality. Hence, due to the discrete nature of the TRV runs, the behaviour of the track is unknown between inspection runs. Therefore, to better understand the performance of track geometry and the measurement frequencies, a numerical simulation of the deterioration-maintenance cycle of an idealised section of track was carried out. The primary purpose of the simulation is to understand the relationship between sampling frequency and apparent rate of deterioration for a given actual rate of deterioration. Random number generation was used to introduce some variability into the maintenance regime, in order to represent realistic conditions.

The Top SD parameter is used to simulate the geometry deterioration and track maintenance events, since the track maintenance and inspection thresholds are predominantly determined based on the Top SD values. The first stage in the development of the numerical model was to set out five assumptions:

4.3.4.1 Deterioration rate.

To simplify the problem in the first instance, the rate of deterioration was assumed to be constant for each simulation. To complete the deterioration-maintenance cycle, the only other values required are the SD value at which maintenance is triggered and the SD value achieved after maintenance.

4.3.4.2 Maintenance trigger level.

As a guide to maintenance trigger levels, reference is made to the relevant Network Rail Standard (NR/L2/TRK/001/mod11, 2015). The alert levels give values for the Top SD of 2 mm, 3 mm, 4 mm, 5 mm and 6 mm, corresponding to 125 mph, 100 mph, 75 mph, 50 mph and 25 mph, respectively. The Standard states that no immediate action is required at these alert levels, but that the fault should be corrected during the next period of planned maintenance. These intervention levels are set for maintenance due to any cause not necessarily associated with earthwork issues.

The primary method of maintenance is assumed to be tamping. While the trigger levels for maintaining the track are set by Network Rail and are dependent solely on speed, the standard of geometry achieved by maintenance should be independent of line speed. Observation of maintenance cycles recorded during the study suggests that Top SD value is reduced to a post maintenance value of between 0.5 mm and 1.5 mm by tamping, further bolstered by the work of Audley and Andrews [133], which concluded similar post maintenance Top SD values. This is incorporated into the simulation assuming “Top SD post maintenance” equals to 0.5 + rnd, where rnd is a random number between 0 and 1.

4.3.4.3 Planned maintenance frequency.

The planned maintenance frequency is based on a simple calculation of the difference between the maintenance alert level and the average SD after maintenance, which is assumed to be 1 mm. This is likely to represent a maintenance frequency slightly higher than is necessary, to ensure adequate opportunity to maintain.

4.3.4.4 Simulation of recorded SD time history.

The simulation proceeds by calculating the planned maintenance periods. The true Top SD time-history is computed according to the criteria described. However, if the Top SD at a given planned maintenance event does not reach the alert level, it is assumed that no maintenance is undertaken until the next planned date.

4.3.4.5 Numerical Simulation—Results

The numerical simulation model was run for line speeds ranging from 25 to 125 mph and for deterioration rates ranging from 0 mm to 6 mm SD per year. The output of the simulation is a set of minimum and recommended threshold values for the track geometry recording frequency. The minimum threshold is the theoretical recording frequency pertaining to a given combination of rate of deterioration and line speed, below which it will not be possible for any two successive recordings to fall within two successive planned maintenance interventions. The recommended threshold is the minimum frequency in order to have a reasonable chance of observing the actual deterioration rate.

The indicative threshold values from the simulation are shown in Figure 4.5, which plots the minimum annual track geometry recording frequency thresholds against the line speed and true deterioration rate. The chart shows that as the line speed and deterioration rate increase, the minimum number of annual geometry recording runs required to accurately calculate deterioration rate increases.

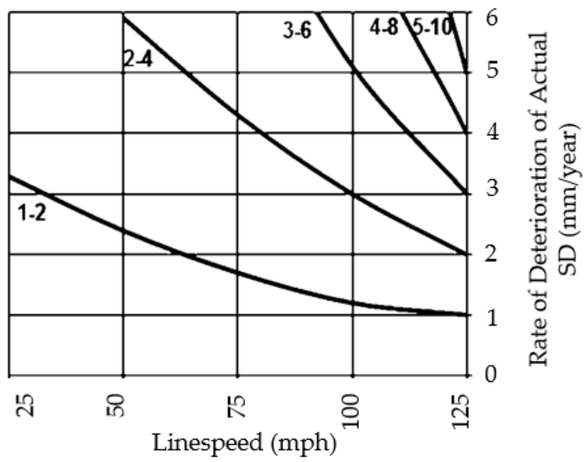


Figure 4.5. Indicative value of minimum required recording frequency

The numerical solution outlined demonstrates the theoretical impact of the track geometry recording-run frequency on the observed rate of deterioration of track geometry parameters. In addition, it outlines the minimum recording frequencies required to compute a reliable deterioration rate.

Figure 4.6 shows the average MaxGrad18 and average AvGrad18 values for track geometry recording-run frequencies between 4 and 23 records per year. All the results for each DetYr for each 10-yard section have been used to calculate these average metric values. Figure 4.6 shows the strong positive correlation between the frequency of the recorded track geometry data and the calculated AvGrad18 and MaxGrad18 metrics, confirmed by the high R2 values. Thus, the higher the run frequency, the more likely that a high EIM value would be observed. The impact of run on the EIM makes it difficult to compare the relative performance of embankment assets, particularly when the recording frequency can vary markedly between different sections of track. Therefore, further work needs to be carried out on quantifying the impact of run frequency on EIM and propose a method for mitigating this impact. Moreover, the use of the R^2 measure has some limitations: it gives an estimate of the relationship between the two variables, but it does not indicate whether the data and predictions are biased. In general, a high or low R^2 isn't necessarily good or bad as it doesn't convey the reliability of the model nor whether the chosen regression is the correct one.

The graph also shows that the influence of the recording run frequency is less significant for AvGrad18 than for MaxGrad18. Therefore, although the MaxGrad18 metric was calculated in an attempt to measure the maximum rate of deterioration occurring during a year, with the assumption that it was this maximum rate which would indicate the level of risk related to failure, it can be seen that the value is highly sensitive to the recording run frequency. This level of sensitivity to the recording frequency is seen as too high to assert its validity, or to reliably correct the measured value based on the frequency. Based, therefore on this consideration, the AvGrad18 metric has been chosen as the better measure to assess earthworks instability using the track geometry data, due to its reduced sensitivity to recording frequency. It is referred to as the "Embankment Instability Metric" (EIM) from this point onwards in this document.

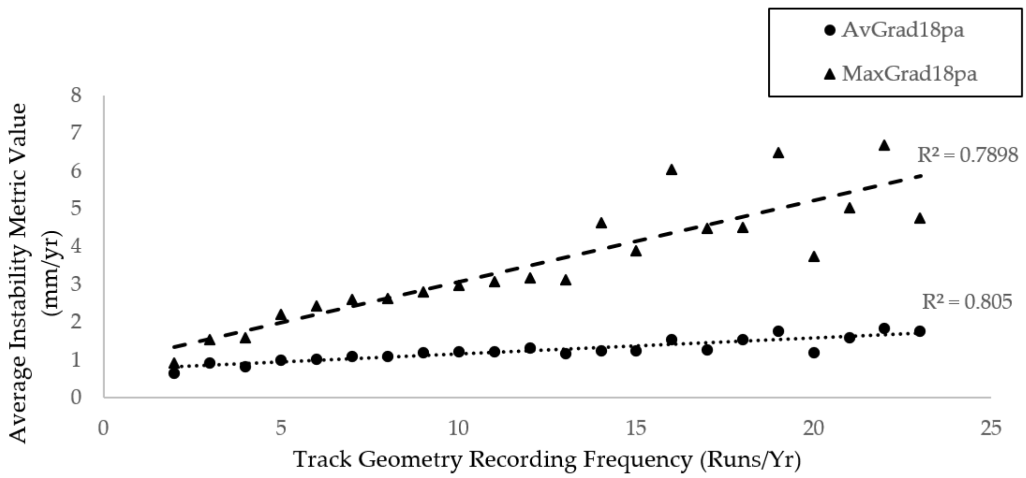


Figure 4.6. Track geometry recording frequency vs. EIM (after [130]**)**

4.3.5 Limitation and Exclusion in Data Processing

The EIM was calculated for 10-yard sections of track throughout each asset for all of the years in which there was available and sufficient data (at least two track geometry recordings per deterioration year). However, some exclusions were made in this calculation of the EIM. EIM results have been discounted for specific DetYr in specific 10-yard sections in cases where:

1. The track geometry data is older than 2010, as this data is not reliable enough to be analysed. Therefore, the first deterioration year for which results are given starts on 01-May-2010;
2. The track geometry data is more recent than 30-April-2018 (since at the time of the analysis there was not a complete year of data yet);
3. Tracks are listed as bi-directional, as left and right rail labelling couldn't be verified and this consistently affects the calculation of differential Top;
4. No data exists during the deterioration year (this is the case in a reasonably large number of cases, mostly in pre-2010 data, but some more recent ones, likely to be due to temporary speed restrictions TSRs or even line closures);
5. Fewer than two track geometry runs are recorded during the deterioration year.

4.3.6 Embankment Instability Metric Values (EIMs)

The following Table 4.2 presents the values generated for the EIM for all of the processed assets, subsequent to the exclusions indicated in Section 4.3.5.

Table 4.2. Summary of results of EIM from population of 10-yard segments (after [130])

Measure	EIM	%
Average Value	1.03	-
SD of Value	1.06	-
Count of Total	98,692	-
<1 mm	64,236	65%
1–2 mm	22,886	23%
2–4 mm	9354	9.5%
>4 mm	2129	2.2%
>8mm	218	0.2%

4.3.7 Results

4.3.7.1 Embankment Instability Metric Thresholds

As suggested, in previous work to establish this technique [129], thresholds of geometry deterioration have been suggested based on observations from failure sites. The study presented in this paper examined a further 51 known failure sites; of these, there are 28 sites which have sufficient data in the reported time period of failure, and examination of the track geometry data suggests clear signs of failure in 19 of these cases. The remaining 9 locations were excluded from the examination as they showed low data frequency and therefore it was not possible to establish a reliable metric from them. For these 19 sites, the maximum EIM values range between 4.2 mm/yr. and 12.8 mm/yr. for all years examined over all 10-yard sections of the failure sites.

This assessment of the rate of deterioration for these earthwork failures, combined with an understanding of the likely rates of deterioration due to trackbed failure and the effect of maintenance, confirms the assertion of the following suggested risk level thresholds in Table 4.3. In this project, and therefore in this thesis, "risk" is defined as the effect of an embankment problem on track system performance, with the purpose of highlighting the appropriate maintenance action to be undertaken. These thresholds apply to the calculated EIM and are the same values as developed in Sharpe's study.

It should be noted that these risk thresholds are only intended as guidance. Presently, there has been limited calibration of these values and they are based on limited observations from known failure sites. It occasionally happens that high readings are obtained from sites that have not been identified as problematic. The prevalence of these erratic readings is low, and also depends on the level of purging to remove erroneous runs completed by the AECOM team. There are two main reasons why this could happen: 1) the sites are actually problematic, but this is not detected by other monitoring means; 2) the record of longitudinal alignment is poor. This last point 2 could be due, for example, to the presence of a bridge or a crossing on or near the embankment which artificially increases the EIMs. When this occurs, it generally shows as a random high EIM whereas problematic assets experiencing real issues show high EIMs repeatedly over at least two years. Therefore, through visual inspections along with any contextual information (location of track assets, satellite views of the area etc.) it is easy to spot over-reporting embankment problems. On the other hand, low readings are sometimes given for sites that are reported as problematic. This can happen when the cause of the issue does not reflect on the track movement and is therefore not recorded by TRV data.

Although an average metric value for an embankment asset (110-yard) has been considered as a measure in this study, it is recommended that the risk classification thresholds apply to each in-year 10-yard metric value of the asset, rather than an averaged metric value. This is to avoid overlooking isolated segments of asset that may present higher values of EIM which would be smoothed by the use of the average metric.

The thresholds below have been used to show the split of observed EIM values for the population of all embankment assets analysed in this study. Considering the maximum EIM value generated (for all years and 10-yard sections) of each embankment asset, the thresholds showed 4% negligible, 23% minor, 41% moderate and 32% high risk.

Table 4.3. Embankment instability metric threshold descriptions

Risk Level	Metric Value	Description and Intervention
Negligible	<1 mm/yr.	Negligible infers that any track roughness would be addressed during routine track maintenance and would not therefore be identified as an earthwork problem.
Minor	1 to 2 mm/yr.	May or may not be identified as an earthworks issue, could be dealt with through track maintenance assuming rates of deterioration do not increase.
Moderate	2 to 4 mm/yr.	Moderate movement which is more likely to be identified and related to a potential earthwork issue.
High	>4 mm/yr.	High risk is judged to be the point at which it is obvious that there is a serious earthwork issue that requires regular track maintenance (very regular for high line speed) to maintain track geometry and will require a long-term earthwork remedial solution.

4.3.7.2 Recommended Track Geometry Recording Frequency Threshold

For the results presented along with this project, a minimum of two track geometry recordings are required in any one deterioration year, to calculate an EIM value for that year in that section. However, the study has suggested that a higher frequency of recording is recommended to calculate metric values.

The numerical simulation used to model changes to track geometry recording frequency suggests minimum and recommended thresholds for the recording frequency in order to reliably calculate the EIM. The thresholds are shown in Figure 4.5, and it is suggested that the recommended thresholds are applied. These thresholds should be calculated by assuming a deterioration rate of 4 mm/yr (as this is the high-risk level). For example, a line speed of 110 mph would require a frequency of six records per year.

4.3.7.3 Sensitivity Analysis

A sensitivity analysis has been conducted to consider the effect of other variables on the metric (such as track curvature, tonnage of rail traffic, etc.).

<u>Influence of track curvature</u>: since the EIM considers the deterioration rate of alignment and differential Top in the calculation, it is worth considering whether the curvature of the track, directly related to the track cant, has significant influence on the deterioration of the alignment. The figure below (Figure 4.7) shows the variation of the EIM in comparison to the change in track cant. As it can be seen, there is no evidence suggesting any correlation between the alignment deterioration rate or EIM with track curvature. There are lower EIMs at higher cants, but this may be a consequence of line speed: the highest cants are found on higher speed lines which are, consequently, more frequently tamped so as to ensure passenger comfort.

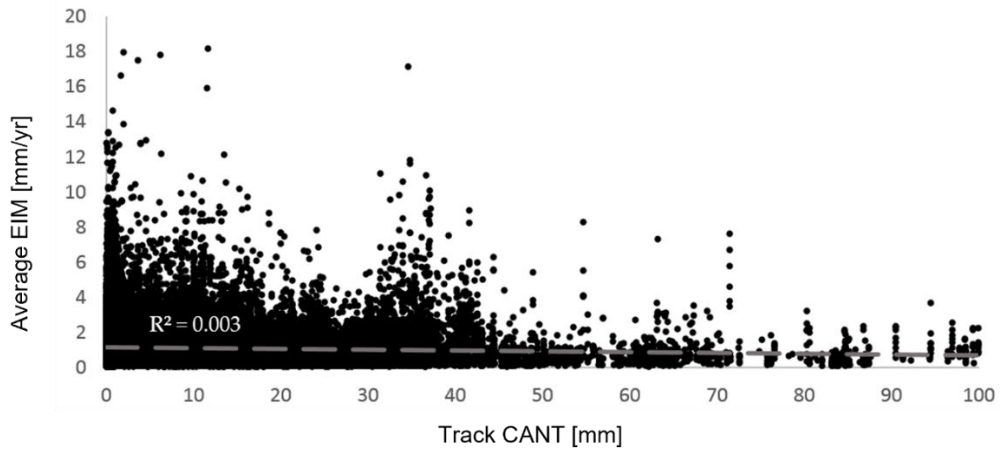


Figure 4.7. Graph of track CANT vs. Average EIM value (at each measure of CANT) (after [130]**)**

<u>Influence of tonnage</u>: this chapter has suggested that the parameters being measured, from which the EIM is derived, are mainly influenced by movements in the supporting embankments, rather than being affected by deterioration of

the track and trackbed. Across the whole network, the predominant factor influencing track and trackbed deterioration is the tonnage of the rail traffic. Therefore, it has been considered if any potential correlation between the tonnage and the EIM values existed. Figure 4.88 presents the values of the average EIM for increasing tonnage (known as MGTPA, million gross tonnes per annum). There is some weak correlation between the tonnage and the EIM, suggesting that either:

(1) there is some effect which an increased tonnage has on instability of the embankments, possibly that increased tonnage may be exacerbating the rate at which the instability develops; or

(2) that the EIM is influenced to a small extent by the general deterioration of the track and trackbed.

This relationship is not significant enough to invalidate the EIM or to require an adjustment of EIM based on route tonnage.

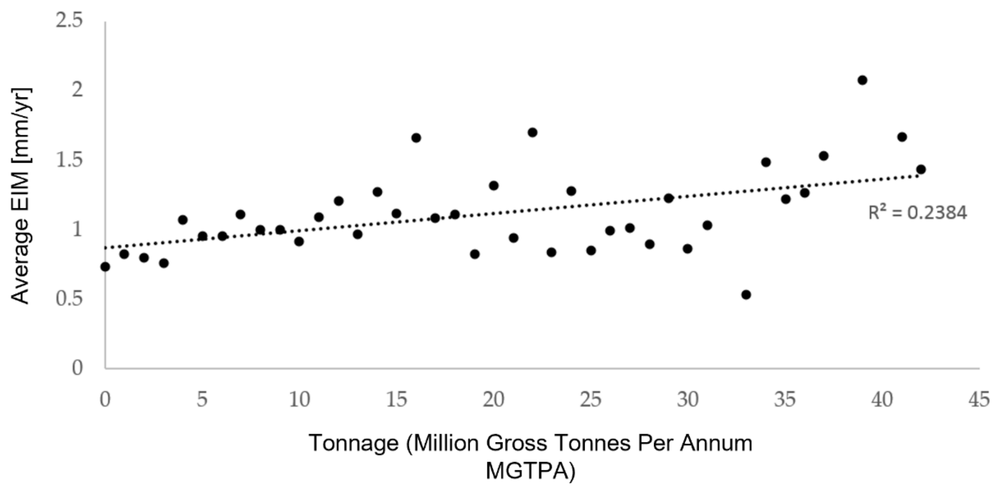


Figure 4.8.Graph showing variation in tonnage vs. Average EIM at each MGTPA value (after [130]**)**

<u>Influence of line speed</u>: consideration should be also given to the effect of line speed on the EIM, which is displayed in Figure 4.9. There is negligible correlation between the line speed and the EIM value.

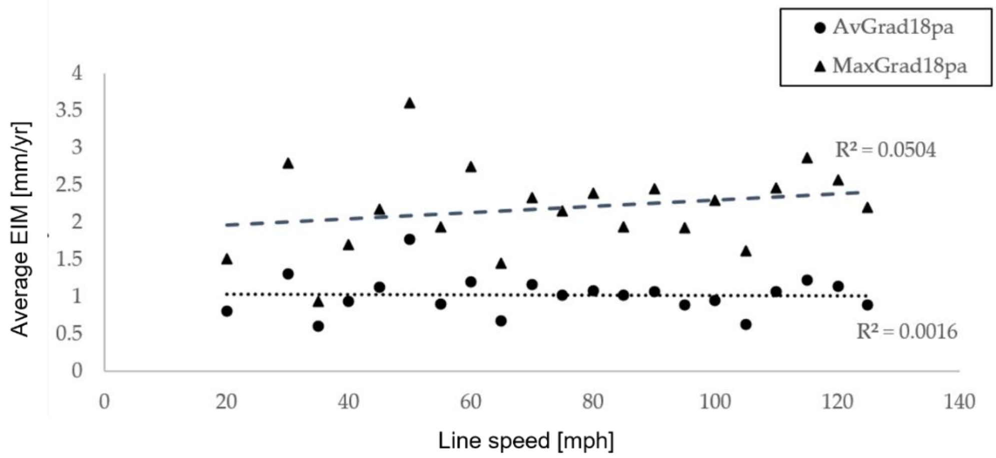


Figure 4.9. Graph showing variation in line speed vs. Average EIM at each Line speed value (after [130]**)**

It can be concluded that the metric has been shown to be independent to these other variables.

4.4 Discussion

The aging earthworks that Network Rail manage pose a challenge to asset managers. Most of the British rail network was constructed largely before the development of modern geotechnical practice, and the modern network still runs on a foundation of earthworks constructed before 1900. Moreover, the service history of the NR earthwork assets is almost completely absent before the last few decades. The majority of stability interventions pre-date have no record and where any information exists it is usually in a form that is not readily accessible today [128].

Regulated industries, such as NR, are constantly challenged to demonstrate continuous improvement to their management processes. Today, the biggest aspiration is to run a safe, reliable, efficient and sustainable infrastructure that is continuously improving. This requires a well-developed capability in asset management, with an appropriate and proportionate management of risk, whilst recognising there is a degree of risk considered tolerable.

Stopping trains from finding failed earthworks that have rapidly lost the ability to perform is one of the top geotechnical challenges [59].

The studies described in this chapter [127], [129], demonstrated that the deterioration of the trackbed performance can be measured through the observation of relevant track geometry parameters (vertical alignment and lateral alignment). Understanding these deterioration rates, their relationship to other trackbed data and the effect of track maintenance, can facilitate modelling of trackbed performance. In general, earthwork movements appear to be characterised by excessive deterioration in both lateral alignment and difference in vertical alignment. Deterioration of both parameters was evident in known areas of earthwork failure.

The EIM study establishes an improved methodology to understand how earthwork movements will appear in the processed data. Failures often show movements for many years before becoming critical (i.e. before the earthwork movement starts to cause track geometry problems that cannot be rectified within a practicable planned maintenance cycle), although prediction of failure is difficult as an increase in rate of movement may simply be a symptom of inherent variability due to different reasons. The data can be indeed affected by both the seasonal variation of earthwork movements and the effects of earthwork problems on the track geometry recording process, but also by changing traffic patterns, and recording-car tolerance limits.

Deterioration of the track geometry related to earthwork movements can be captured by the track geometry data, provided that the recording frequency exceeds the recommended threshold, and the rate of deterioration is not substantially above 4 mm/yr. It is important to note that, if the actual rate of deterioration is substantially above 4 mm/yr and the track geometry recording frequency is not high enough, the track geometry data may appear as erratic and the reliability of estimating the rate of deterioration is reduced. The deterioration rate and recording frequencies at which the data may become erratic will vary with the line speed. The recommended thresholds in Figure 4.5. Indicative value of minimum required recording frequencyshould be used as a guide to understand the required recording frequencies to give reliable estimates of the actual rate of deterioration, and hence EIM values. Further work is required to determine the reliability of the metric when the deterioration rates and recording frequencies do not meet the recommended values.

As suggested in previous work [126] to establish this technique and explained in Section 4.2, thresholds of geometry deterioration have been suggested based on observations from failure sites. This study has examined a further 51 known failure sites; of these, there are 28 sites which have enough data in the reported time period of failure and examination of the raw data suggests clear signs of failure in 19 of these cases. For these 19 sites, the maximum EIM values range between 4.2 mm/yr. and 12.8 mm/yr., for all years examined over all 10-yard sections of the failure sites. For the remaining 9 known failure sites, the data didn't suggest clear signs of failure. There are cases in which the embankment may have an issue, but this would not be detected by TRV. For example, if the embankment is experiencing a very shallow slip, or animal burrowing, then the tracks would not move much if at all. Thus, high EIMs would not be seen. Yet, the embankment could still be listed as a problematic site by NR. This assessment of the rate of deterioration for these earthworks' failures, combined with an understanding of the likely rates of deterioration due to trackbed failure and the effect of maintenance, confirms the assertion of the suggested risk level thresholds in Table 4.3. These thresholds apply to the calculated EIM and are the same values as shown in Section 4.2.

It should be noted that more calibration of these values is needed, as future work, and a wider study to understand the variance of the EIM for embankment assets with no known history of instability would help, giving a context to the proportion of risk threshold breaches.

4.5 Conclusions

The project presented in this chapter demonstrates that track geometry data are a viable source by which to detect railway embankment instability.

Thanks to the development of an algorithm, a value of EIM was computed. In this way a measure of the embankment asset vulnerability to failure was assigned for each 10-yards of track. Analysing a sample of 51 known failure sites, the

EIM clearly showed evidence of high track geometry deterioration, consistent with instability.

The frequency of track geometry data recoding is an important consideration and data availability is a prerequisite for reliable analysis. Data coverage is one major limitation of this technique; typically, only a quarter of the network has sufficient data to analyse the past three sequential years of earthwork performance.

A sensitivity analysis was conducted to consider the effect of other variables on the metric and the metric was shown to be sensibly independent of those other variables.

As a logical extension to this project and the algorithm developed during the study, further works are suggested. The current alignment process (shifting and rubber-banding) of the track geometry data would be extremely improved with the introduction of an automatic process. This will reduce the time and cost required to process and trend data to calculate the EIM and visualise the data for interpretation. More generally, the infrastructure owner should implement improved geo-location techniques which are widely available in other fields.

User input is also required to identify clearly erroneous track geometry data (Section 4.3.1). Automated detection and purging erroneous data would be a logical extension to this project, although it should be solved within the Network Rail systems, possibly making use of machine learning to improve the efficiency of the task.

The study completed by AECOM has been focussed on the analysis of embankment assets identified as in need of remedial work. However, the detection process using the EIM has not been applied through the UK rail network so that, at present, it is not known how this analysis would perform for other embankment assets with no known history of instability. Such a study will assist with understanding what level of false positives may be generated through scaling-up this analysis and help to quantify other factors which may be influencing the metric.

CHAPTER 5. METHODOLOGY

5. Methodology

As stated previously, it is essential to detect failures at an early stage in order to target strengthening works in a cost-effective manner. Embankment instability is a significant cause of traffic disruption and/or train delays: it either makes track unusable or requires speed restrictions to be put in place for the safe passage of trains. Preventing trains from encountering failed or failing embankments that have lost their ability to perform is the current key geotechnical challenge for Network Rail. At present there is no reliable method for predicting when or where earthwork failures may occur.

This research attempted to link the causes of embankment instability (Chapter 3) to visible/detectable symptoms (track geometry parameters Chapter 3). Thus the aim of the research was to highlight whether correlations between track geometry metrics and embankment geotechnical features existed, and if they did, thereby to gain an early understanding of potential failure. This would allow proactive plan intervention and, therefore, improve budget allocation. Finding the possible correlations between factors, indeed, would identify issues that require closer monitoring than others, and therefore assign a priority of intervention. Another aim of the research was to help the development of a decision-making model for the improvement of geotechnical asset management processes. To achieve this, a thorough study of the factors playing a role in failures, the types of failure, the monitoring systems, the maintenance works, the current asset management strategies, and the decision-making processes was undertaken.

The track geometry datasets used for this study were available in AECOM's database, previously collected for the "Embankment Instability Modelling Research" project (Chapter 4). That AECOM project demonstrated that track geometry data is a viable data source to consider for detection of railway embankment instability. An algorithm was developed and tested to produce a new metric named Embankment Instability Metric (EIM) (Chapter 4) for the classification of the severity of earthwork movement. Based on the findings of that project, and assuming the validity of its hypothesis and limitations, the author listed the parameters likely to influence embankment instability. These parameters were then correlated with the EIM so as to:

- Set parameter criticality.
- Establish priority of monitoring and intervention.
- Establish a range of values showing the different levels of performance for each parameter.

The outcome of this research is a wider understanding of the interaction of the embankment with the complex rail structure system. The link between causes of instability and their symptoms leads to a better technical comprehension of the asset deterioration, it allows the improvement of its modelling and permits a more considered decision-making process.

As a last step, this research delivered a predictive tool based on the multi-criteria decision-making approach (presented in Chapter 7).

Steps of the research are as follows (Figure 5.1):

- Identification of instability drivers through literature review (Chapter 3)
- Selection of assets for analysis
- Screening of indicators based on criteria
- Data collection

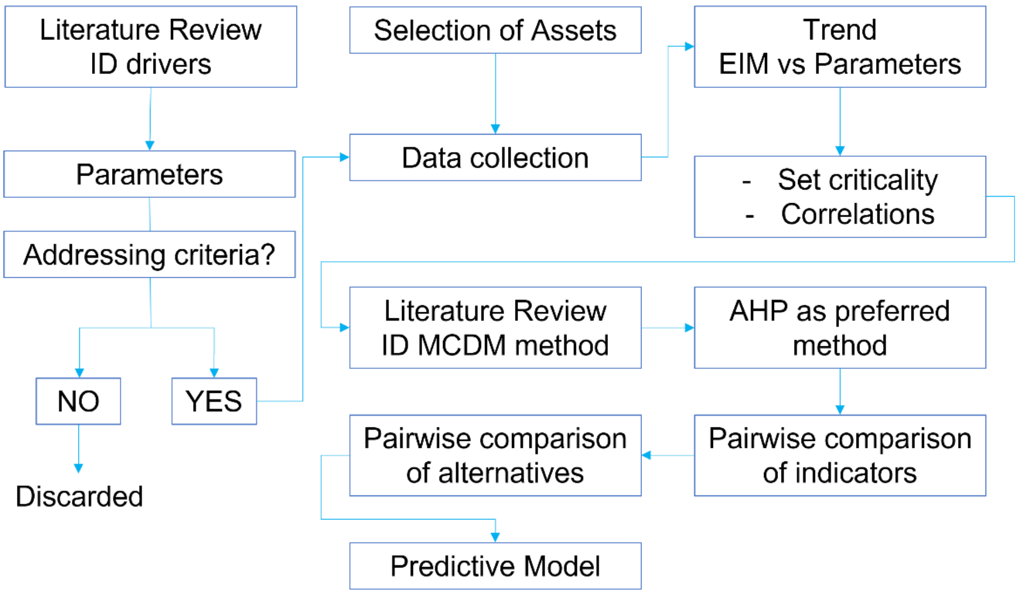


Figure 5.1 Research Steps

5.1 Sites for analysis

With the assistance of staff at Network Rail's York offices, the author identified the susceptible embankment sites for the study. An asset was defined as susceptible if it had failed at least one time between 2010 and 2018 (the time frame for the study). Indeed, in January 2018 Network Rail commissioned AECOM's Rail Asset Management team to undertake the study on the use of track geometry data to perform analysis of embankment assets, which were planned to be renewed, refurbished or maintained during a 5-year Control Period, as described in Chapter 4.

The study examined 51 known failures; of these, 26 failure events had reliable data for the analysis in the reported time period of failure. These 26 failures are distributed among 22 asset sites along 16 lines (Figure 5.2) and have been used for the analysis in this thesis. The assets, chosen from those mentioned in Chapter 4, are listed in the Table 5.1 below:

Table 5.1 Sites for analysis

Line Name	Engineer's Line References (ELR)	Asset Start Mileage	Asset End Mileage	Failure Start Mileage	Failure End Mileage	Date of Failure
Glasgow and South Western Line	GSW	59.1430	29.1507	59.1479	59.1483	2016/08/28
Guildford to Ash Junction	GTW1	34.1108	34.1210	34.1105	34.1207	2016/09/09
				34.1126	34.1211	2018/08/02
Hanslope Northampton and Rugby Line	HNR	70.0110	70.0220	70.0189	70.0210	217/07/27
Longlands, Eaglescliffe and Newcastle Line - East Low Junction to Newcastle East Junction via Hartlepool	LEN3	85.0440	85.0516	85.0442	85.0505	2012/12/29
Main Line (Paddington to Penzance via Bath)- St Germans to mileage change point east of Chacewater	MLN3	268.0770	268.0880	268.0790	268.0870	2017/03/09
				268.0780	268.0850	2017/11/07

Line Name	Engineer's Line References (ELR)	Asset Start Mileage	Asset End Mileage	Failure Start Mileage	Failure End Mileage	Date of Failure
Neasden South to Aynho Junction- Ashendon and Aynho Line	NAJ3	5.1316	5.1536	5.1410	5.1519	2016/08//31
Newcastle and Carlisle Line - Millage change to Carlisle South Junction	NEC2	20.0441	20.0548	20.0436	20.0475	2012/07/09
		25.1540	25.1650	25.1615	25.1629	2015/12/27
		38.1425	38.1535	38.1551	38.1565	2012/06/28
New Guildford Line (Hampton Court Junction to Guildford)	NGL	20.0671	20.0766	20.0682	20.0725	2016/08/19
						2018/04/04
		25.1105	25.1210	25.1101	25.1141	2014/09/26
Oxford Worcester and Wolverhampton Line	OWW	92.154	92.1722	92.1611	92.1672	2017/07/28
Settle and Carlisle Line	SAC	255.0440	255.0550	255.0450	255.0470	2016/07/30
South Croydon to Uckfield	SCU1	19.099	19.110	19.1034	19.1078	2014/02/07 (DOWN)

Line Name	Engineer's Line References (ELR)	Asset Start Mileage	Asset End Mileage	Failure Start Mileage	Failure End Mileage	Date of Failure
Branch - South Croydon to Uckfield						2014/02/10 (UP)
St Pancras to Chesterfield Line - St Pancras to Bedford	SPC1	40.1430	40.1540	40.1520	40.1560	2014/02/02
		44.1430	44.1540	44.1520	44.156	2014/02/02
		45.0770	45.0880	45.0814	45.0835	2015/15/02
Tapton Junction (Chesterfield) to Colne - Masborough to Colne	TJC3	162.0990	162.1100	162.1000	162.1050	2016/09/21
Tonbridge to Hastings Line	TTH	42.1650	42.1745	42.1652	42.1652	2012/06/17
West Coast Main Line (north of Carlisle) - Carlisle to Law Junction	WCM1	40.1339	40.1375	40.1346	40.1386	2013/12/30
		40.1339	40.1375	40.1365	40.1373	2015/12/05
West Coast Main Line (north of Carlisle) - Law Junction to Glasgow	WCM2	94.1320	94.1433	94.1293	94.1328	2016/01/03

Figure 5.2 Asset Locations

5.2 Screening of factors

Slope failure occurs when the downward active forces on material, due to gravity, induce shear stresses in the soil that exceed its shear strength. The shear strength is a consequence of the geotechnical properties of the rock or soil mass and its state of effective stress. Therefore, factors that tend to increase the shear stresses or decrease the shear strength increase the chances of a slope failure. Different processes can lead to a reduction in the shear strength of the mass. Increased pore pressure, cracking, swelling, decomposition of clayey rock fills, creep under sustained loads, leaching, strain softening, weathering, erosion and cyclic loading are common factors that decrease the shear strength of a soil mass. In contrast to this the shear stress in rock mass may increase due to additional loads and increase in cracks' water pressure at the top, increase in soil weight due to water content, excavation at the bottom of the slope and seismic effects.

From the literature review, thirteen different parameters emerged as potential embankment instability drivers. It appeared relatively quickly that data for all the thirteen drivers were not easy to collect. Information on some of these parameters is not recorded for all the sites, while data records are not available at all for some parameters. For the purposes of the study – i.e to obtain a predictive, rapid assessment tool that does not require lengthy and expensive data collection at each embankment site – a screening of indicators was carried out considering the following criteria:

1. Scientific evidence: the parameter must be a proven driver for instability as indicated by scientific research published in recent decades. Peer-reviewed papers must show professional consensus on the importance of an issue (Figure 5.3).
2. Availability: Data collected for the study needs to be:
   - openly available in a public repository that issues datasets, and/or
   - generated at a central, large-scale facility, available upon request

   If data is subjected to third party restrictions (NR, AECOM) the availability needs to be previously agreed. Data storage and how data can be obtained and interrogated must be well-defined at an early stage.
3. Measurability: The way in which parameters are measured/evaluated must be clear, objective, and unbiased. The clarity is especially important when qualitative parameters are involved.
4. Data Coverage: Data collected and involved in the study must be available for all the sites evaluated. If one of the datasets is unavailable for one asset, then the parameter must be discarded from the analysis.
5. Updatability: Data considered must be routinely collected and so updatable to repeat the analysis in the future.

A score from 0 to 10 (with 0 meaning "totally addressed", 5 meaning "partially addressed" and 10 meaning "impossible to address" (see Table 5.2 for complete definitions) was assigned at each driver for each criteria based on expert opinions (AECOM employees who repeatedly deal with problematic railway sites), literature review presence and availability/reliability in as in

Table 5.3.

Table 5.2 Scores and definitions

Description	Level	Score
Totally addressed		0
Addressed	High	1
	Medium	2
	Low	3
Partially addressed	High	4
	Medium	5
	Low	6
Barely addressed	High	7
	Medium	8
	Low	9
Impossible to address		10

Table 5.3 Tot Score per Driver

Driver	Scientific evidence	Availability	Measurability	Data Coverage	Updatability
Soil material	1	4	5	6	7
Water content and rain	1.5	0	1	1	1
Geology	2	1	5	1	4
Density index	2.5	8	1	9	8
Permeability	2.5	9	1	9	8
Erosion	3	10	5	10	9
Soil specific gravity	3.5	4	1	7	4
Soil permeability	4	9	1	9	8
Compaction	5	9	5	10	9
Vegetation	6	2	5	2	3
Drainage	6	6	5	6	6
Soil compressibility	7	8	1	9	8
Railway traffic	8	7	5	3	3
Wildlife	9	7	5	8	7
Weather condition	9	2	1	2	2

The scientific evidence is based on the recurrence of key words in web-based scientific search engines for publications on slope instability between 1980 and 2019, in main journals specialised in subject category "Geotechnical Engineering and Engineering Geology" (i.e. Geotechnique, Engineering Geology, Rock Mechanics and Rock Engineering). The result of the information gathered is illustrated in Figure 5.3 where the keywords are arranged from the most recurrent (soil material) to the less recurrent (weather condition).

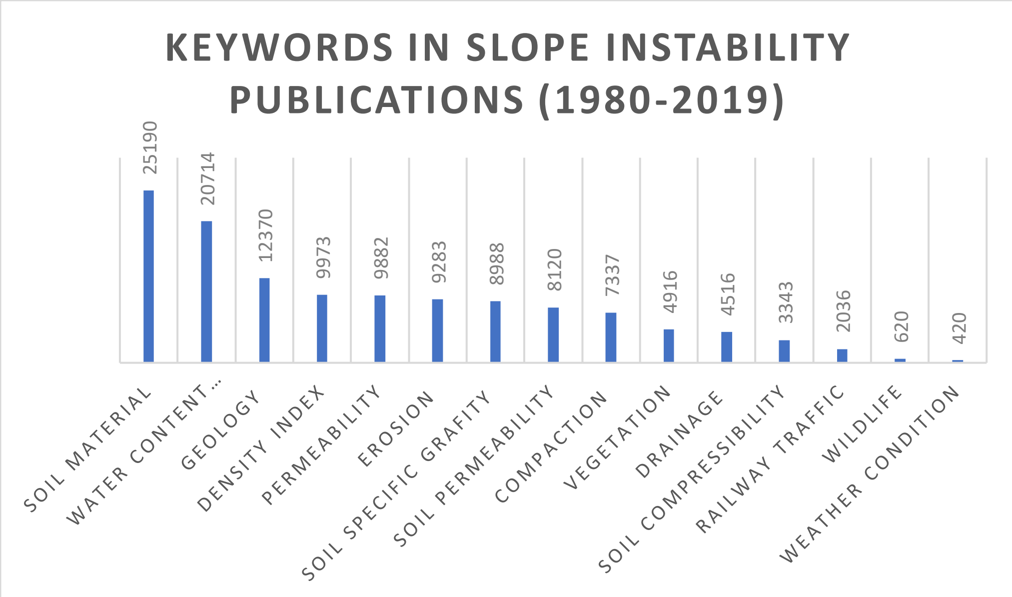


Figure 5.3 Key words in geotechnical engineering publications on slope instability between 1980 and 2019

To check availability of data and for eventual collection, software like LADS, JBA and Geo-Rinm Viewer were used. LADS - Linear Asset Decision Support is an ORBIS project implementation, a tool allowing linear asset information to be aligned and visually represented in an interactive form so as to aid decision-making. JBA is a Network Rail Earthworks Examination software; it is a location-aware data collection system, specifically designed for examination of earthwork condition and hazards along the railway system owned by Network Rail. This is linked to a centralised web server providing data synchronisation and a web-based mapping application for management and reporting. Geo-RINM Viewer captures imagery and detail of all 20,000 miles of track and surrounding infrastructure and gives information on the geology, compressibility and plasticity of the earthworks.

Availability is scored 0 “totally addressed” when information on the driver is easily obtained from database research with no restrictions (i.e. rainfall information can be gathered from open source web servers). Vice versa, it is scored 10 “impossible to address” when no record of its action is stored in any official database (i.e. there is no record found on erosive actions).

The data coverage score reflects the availability of information on drivers for the specific sites analysed (Table 5.1 Sites for analysis), at least for the time frame considered (2010 – 2018).

Indeed, the updatability of the data sets is scored as well to reflect the possibility to repeat the analysis for future assessment.

To finally define which drivers address the criteria described, a total score (Table 5.4) for each is calculated as the average of the scores assigned to the criteria. The resulting scores are not rounded, and all the drivers with a total final score of 5 or lower are included in the following steps. This is valid even if a driver scored 10 on one or more factors, but the final average still is 5 or lower. The author arbitrarily set the cut-off at 5 since ‘partially addressed’ is considered the minimum level required to appropriately analyse a parameter. The selected drivers are highlighted in bold in Table 5.4.

Table 5.4 Total Score per factor

Driver	Total Score
Soil material	**4.6**
Water content and rain	**0.9**
Geology	**2.6**
Density index	5.7
Permeability	5.9
Erosion	7.4
Soil specific gravity	**3.9**
Soil permeability	6.2
Compaction	7.6
Vegetation	**3.6**
Drainage	5.8
Soil compressibility	6.6
Railway traffic	5.2
Wildlife	7.2
Weather condition	**3.2**

Soil material, *Geology* and *Soil specific gravity* will be considered from this point onward as a unique driver called "Type of soil". This choice is due to the way NR provide information on these drivers: NR classify the soils into categories which group liquid limit, clay fraction and apparent cohesion (Section 3.2.5.6, Section 5.5.1).

Weather condition will be identified as "Seasonal Deformation". The combination of rainfall and temperature data was used to identify Dry and Wet Season to interpret shrink and swell processes affecting stability (Section 3.5.1).

5.3 Possibility of linking the EIM to geotechnical embankment failure

From the point of view of identification of embankment failures, several failure mechanisms have been reported and categorised in the literature for railway embankments. These include: (i) soil mechanics properties (ii) geotechnical issues (iii) external factors. They are inter-related and may act together in combination or sequentially (e.g. shrink-swell deformation may lead to progressive failure of the embankment, with the final rupture triggered by an increase in pore water pressure).

Starting from these 28 failures (Table 5.1), the correlations between EIM and the parameters affecting embankment behaviours are highlighted in order to identify possible synergies and interactions. Based on the discussion presented in the previous sections, the parameters considered are:

- Type of soil.
- Rainfall.
- Vegetation.
- Seasonal deformation.

Finding correlation between the EIM and the above parameters can allow for prioritisation of interventions. Looking backward at the evolution of the parameter values, starting from the point at which distress was first detected, helps in understanding the origin of the problem. Moreover, correlating these trends with the EIM allows identification of a "warning point" for each correlation and, so, can help to highlight critical value ranges for each parameter considered in the analysis. The development of knowledge of asset degradation comes from the understanding of the degradation mechanism itself and geometry.

5.4 Using correlation to improve the geotechnical asset management

Linking the evolution of the features affecting stability to the effect each has on visible symptoms (track geometry) leads to progress in the monitoring system efficiency and prioritisation of interventions. This would be translated in terms of "intervention before failure" and so in reduced risk of derailment and reduced cost of unforeseen failures.

In the scenario of an improved Geotechnical Risk Management, a more informed Decision-Making Process is allowed.

Multi-Criteria Decision-Making MCDM methods have been developed to support the decision-maker in their unique and personal decision process. MCDM methods provide stepping-stones and techniques for finding a compromise between the best solution and the available resources. They have the distinction of placing the decision-maker at the centre of the process. They are not automatable methods that lead to the same solution for every decision-maker, but they incorporate subjective information. Subjective information, also known as preference information, is provided by the decision-maker, which leads to the compromise solution.

Considering the number of MCDM methods available, the decision-maker is faced with the arduous task of selecting an appropriate decision support tool, and often the choice can be difficult to justify. None of the methods are perfect nor can they be applied to all problems. Each method has its own limitations, particularities, hypotheses, premises and perspectives. Up to now, there has been no possibility of deciding whether one method makes more sense than another in a specific problem situation.

A brief literature review of MCDM methods in the field of infrastructure management follow in Chapter 6.

5.5 Data Collection

5.5.1 Type of Soil

One of the geotechnical properties affecting the stability of a slope is the shear strength of the soils. This, in turn, may be related to the particle size distribution, density, clay fraction, liquid limit, plasticity.

To obtain the information about different soils Geo-RINM Viewer software was used. The software captures imagery and detail of all 20,000 miles of NR track and surrounding infrastructure and also gives information on geology. The information collected was related to the apparent cohesion, liquid limit and clay fraction of the asset considered. NR classify the embankment core geology within a large database from which information needed was extracted for the sites considered. The geology group and subgroup descriptors (Table 5.5) and the clay fraction vs liquid limit (Figure 5.4) are as follow:

Table 5.5 Geology group descriptors

Group	Subgroup	Geology Group
Cohesive	Low potential[1]	S0
Cohesive	Medium potential	S1
Cohesive	Medium potential	S2
Cohesive	Medium-High potential	S3
Cohesive	High potential	S4
Cohesive	Unknown potential	S5
Granular	All	S6
Organic	Peat	P
Weak Rock	Chalk	C1
Weak Rock	Other	R
Other Rock	All	X

[1]"Potential" refers to the degree of swelling or shrinkage that can be anticipated due to moisture content changes

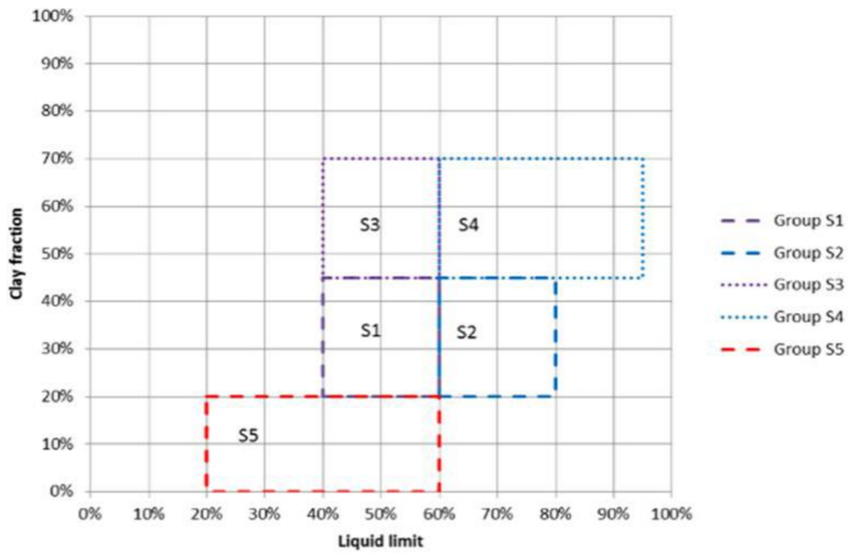


Figure 5.4 Clay fraction vs Liquid Limit

The term "potential" in Table 5.5 refers to the soil shrink-swell potential, i.e. the extent to which a soil shrinks or swells with changes in soil moisture content. This is largely controlled by the amount and type of clay in the soil (Section 3.5.1).

The geology of the sites for the analysis can be found in Table 5.6:

Table 5.6 Type of Soil per ELR

ELR	Group	Geology Group
GWS	Cohesive	S1
LEN3	Cohesive	S1
NEC2	Cohesive	S1
SAC	Cohesive	S1
TJC3	Cohesive	S1
WCM2	Cohesive	S1
HNR	Cohesive	S3
NAJ3	Cohesive	S3
OWW	Cohesive	S3
GTW1	Cohesive	S4
NGL	Cohesive	S4

ELR	Group	Geology Group
SCU1	Cohesive	S4
SPC1	Cohesive	S4
THH	Cohesive	S4
NEC2	Cohesive	S5
NEC2	Non cohesive	S6
SPC1	Non cohesive	S6
WCM1	Non cohesive	R
MLN3	Non cohesive	X

5.5.2 Rainfall

Shear strength of soils is highly affected by moisture conditions (i.e. water content), especially if the soil contains clay materials (frequent in the UK). The main driver for slope failure is often rainfall, and it is possible that a hotter future European climate will see rainfall arrive in more intense storm events.

The centre for Ecology and Hydrology CEH provides CEH-GEAR: 1 km gridded estimates of daily and monthly rainfall for Great-Britain and Northern Ireland. The rainfall estimates are derived from the Met Office national database of observed precipitation. To derive the estimates, monthly and daily (when a complete month is available) precipitation totals from the UK rain gauge network were used. The natural neighbour interpolation methodology, including a normalisation step based on average annual rainfall, was used to generate the daily and monthly estimates. The estimated rainfall on a given day refers to the rainfall amount precipitated in 24 hours between 9am on that day until 9am on the following day.

Monthly or daily resolution is available. Clicking on the map, a rainfall time series plot will be displayed. The time range of the plot includes the time displayed on the map and depends on whether monthly or daily resolution is selected the rainfall amount precipitated in 24 hours between 9am on that day until 9am on the following day (Figure 5.5).

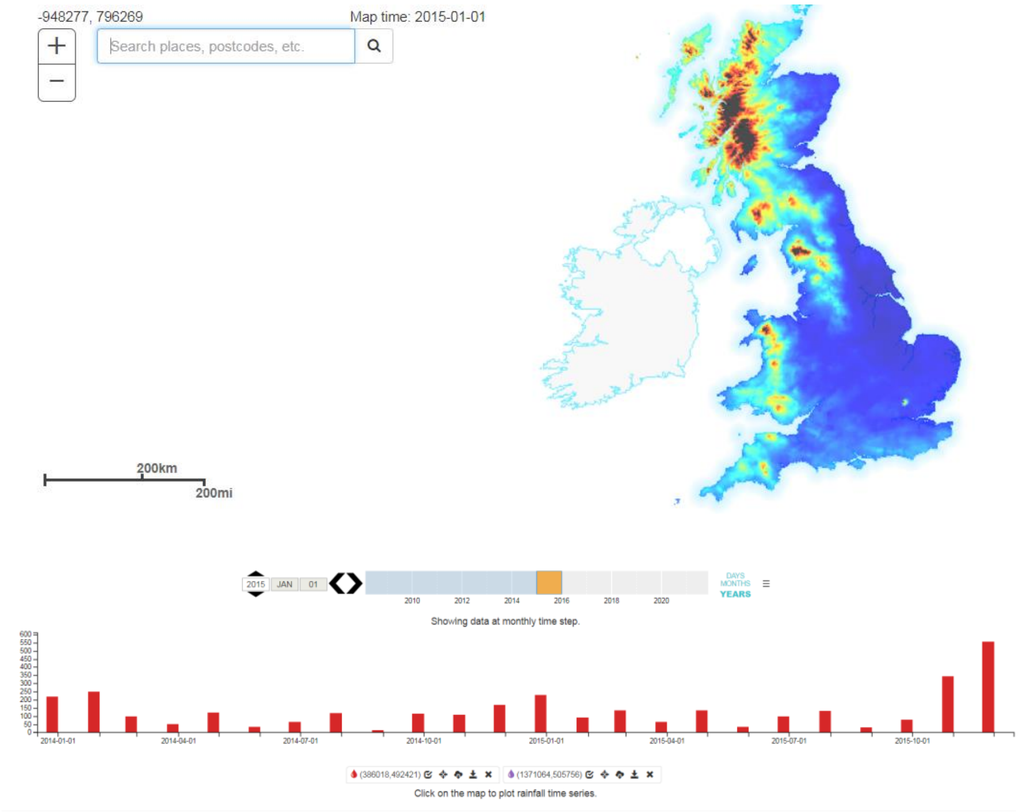


Figure 5.5 Rainfall time series plot in CEH-GEAR

Data was downloaded in 9 NC files (this format of file is used for climate data saved in multi-dimensions so that the user can view each element of dimensions such as Latitude, Longitude, Time), one per year (2010 – 2018). This type of file contains daily rainfall data per each location across the UK. Using the geographic coordinate system, the data was extracted from the NC file and downloaded in Excel with the use of a code written in Python.

5.5.3 Vegetation

Originally, engineering practice saw vegetation as a hazard to be removed. With improved knowledge, a number of positive key impacts were considered on engineering performance due to changes in slope hydrology and mechanical reinforcement of roots.

The Geo-RINM aerial survey (Figure 5.6) data allows a user to identify and to target specific trees that could cause problems for train passage. Geo-RINM Viewer allows discovery of the height of vegetation, the gradient of slopes and combining this with information from tree census, vegetation resilience modelling and BGS data for proactive vegetation management. Moreover, some running trains are equipped with cameras; they record videos and take pictures of the route. Through the software Routeview, selecting route and date of interest, it is possible to have a clear idea of the type of vegetation present along the slopes.

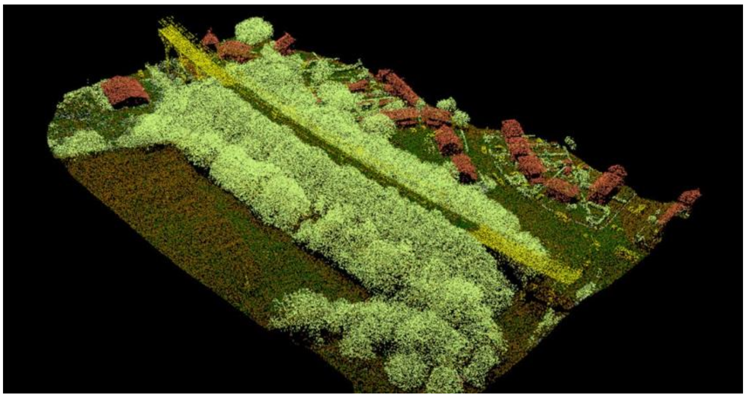


Figure 5.6 - Aerial survey in Geo-RINM

The pictures collected for the sites considered are downloaded from Routeview, and vegetation has been grouped into High, Medium, Low or Mixed vegetation.

The following, four pictures (Figure 5.7, Figure 5.8, Figure 5.9, Figure 5.10) give representative images of each group. The totality of the picture can be found in Appendix A.

The vegetation is distributed among the sites as per Table 5.14

Figure 5.7 High Vegetation (GTW 34m1210y)

Figure 5.8 Medium Vegetation (SPC1 44m1540y)

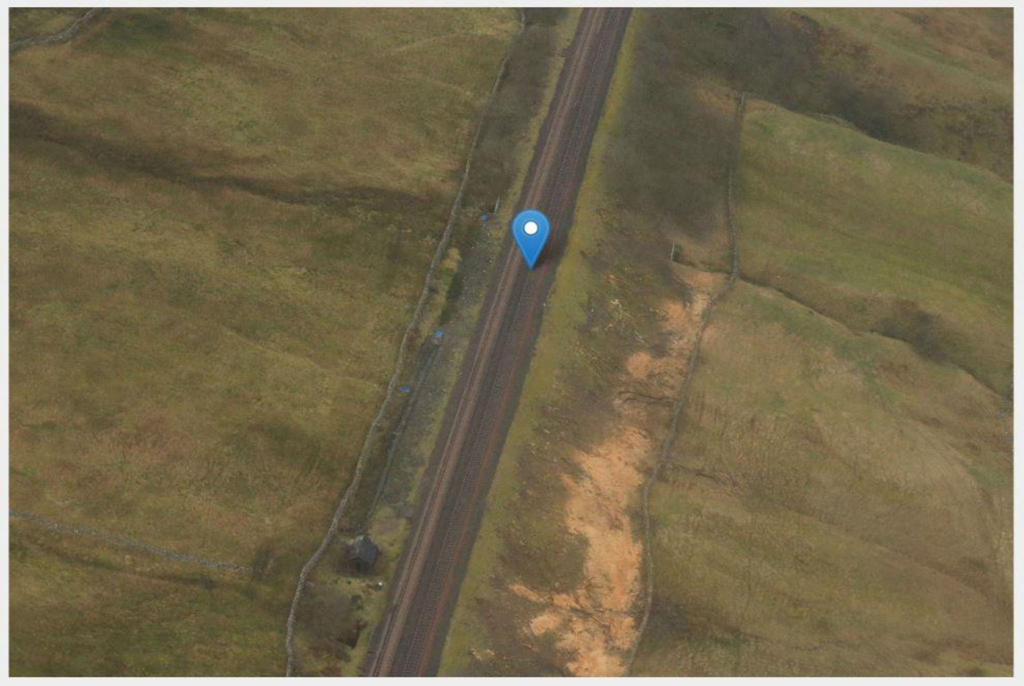


Figure 5.9 Low Vegetation (SAC 255m0440y)

Figure 5.10 Mixed Vegetation (WCM1 94m1320)

5.5.4 Temperature

To collect information on Temperature between January 2010 to December 2018, CustomWeather database was used. The database provides several daily climate information, for the purpose of this thesis daily maximum and minimum temperature per site were collected for the time window considered.

5.5.5 Embankment Instability Metric EIM

The EIM is collected as described in Chapter 4. As mentioned, through the analysis of track geometry data, it has been noted that embankment behaviour sometimes shows seasonal variability. Hence, to encompass one full Dry Season and one full Wet Season for each period analysed, it was decided to run the annual period of analysis from 01-May to 30-April named "deterioration year" (DetYr). Consequently, for the time frame considered, when the data set is completed, there are 7 EIM values per site. The data can be found in Appendix B.

5.6 Data Analysis

The data analysis of this thesis addresses the following research questions:

- Is it possible to link EIM to geotechnical features and external factors influencing embankment instability?
- If so, how can these correlations improve the geotechnical asset management?

5.6.1 Rainfall – simple analysis

As regards the Rainfall, the volume of data available per site is higher than the recorded EIM. Indeed, for the 8 years considered, a daily record of mm of rainfall is available while only one record of EIM is reported per deterioration year. This difference in data volume makes the direct comparison of these two datasets difficult and imprecise.

Therefore, a different approach was tried. The daily rainfall, per site, was summed across each deterioration year (May to April) obtaining a comparable volume of data between total mm of Rainfall data and EIM. On the other hand, 8 single points per site was not representative enough of potential correlation.

Hence, the rainfall data was finally distributed and evaluated against the EIM categories, described in Table 4.2.

Table 5.7 Amount of data per category

EIM category	Observations
EIM < 1	89
1 < EIM < 2	35
2 < EIM < 4	33
EIM > 4	40

Since the goal is to obtain the proportion of variance in the dependent variable (Y = EIM) explained by the independent variable (X = rainfall), a linear regression data analysis was adopted to visually show the strength or the weakness of the correlation between the two variables.

Generally, the less the variability in the data, the stronger the correlation and the tighter the fit to the regression line.

The following four graphs (Figure 5.11, Figure 5.12, Figure 5.13, Figure 5.14) show the results obtained from analysing EIM vs Total Rainfall (i.e. the sum of daily rainfall per deterioration year):

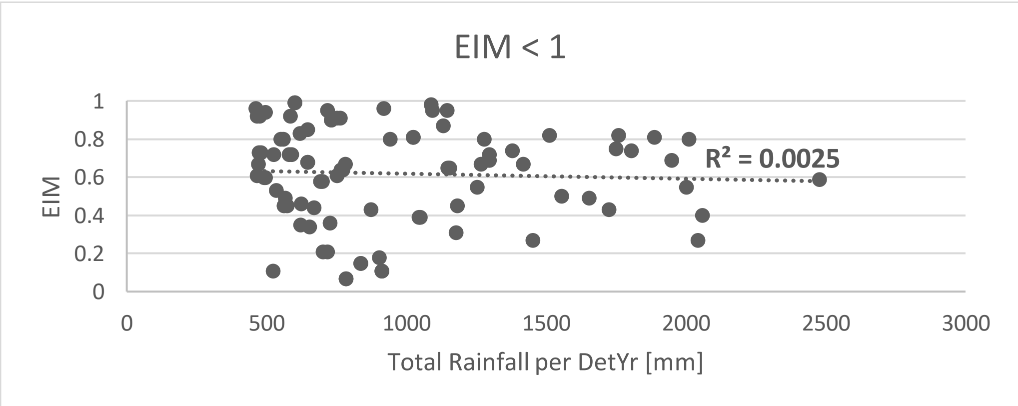


Figure 5.11 Rainfall vs EIM < 1

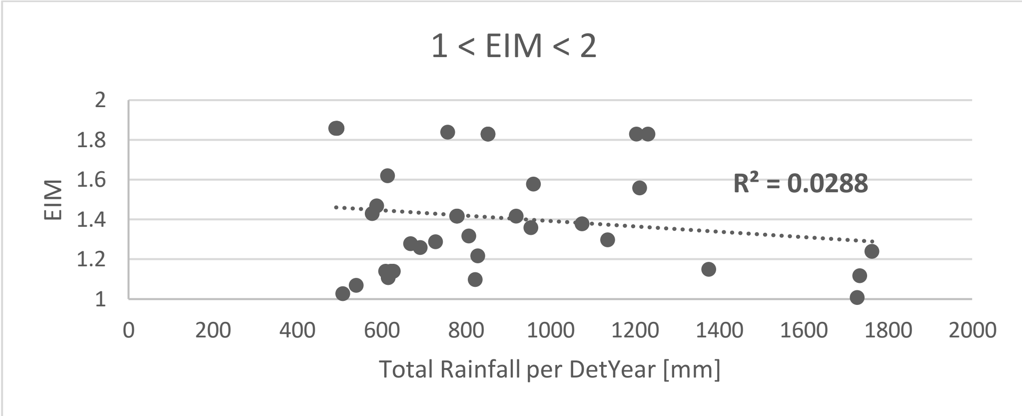


Figure 5.12 Rainfall vs 1 < EIM < 2

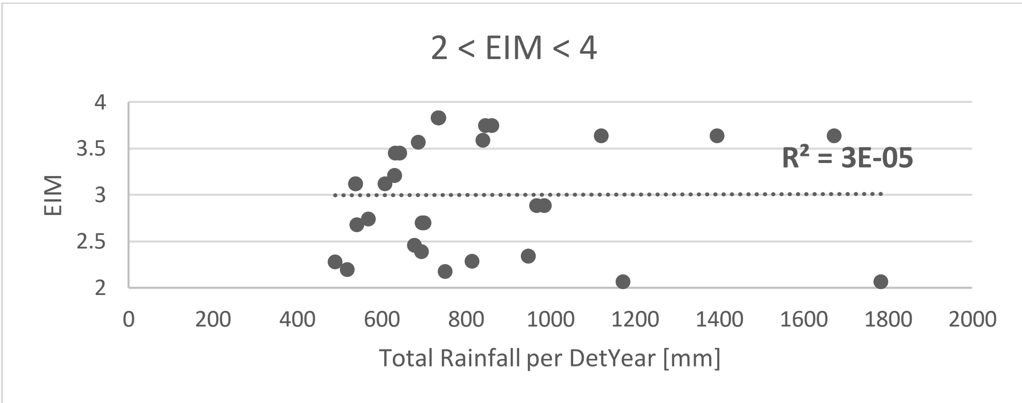


Figure 5.13 Rainfall vs 2 < EIM < 4

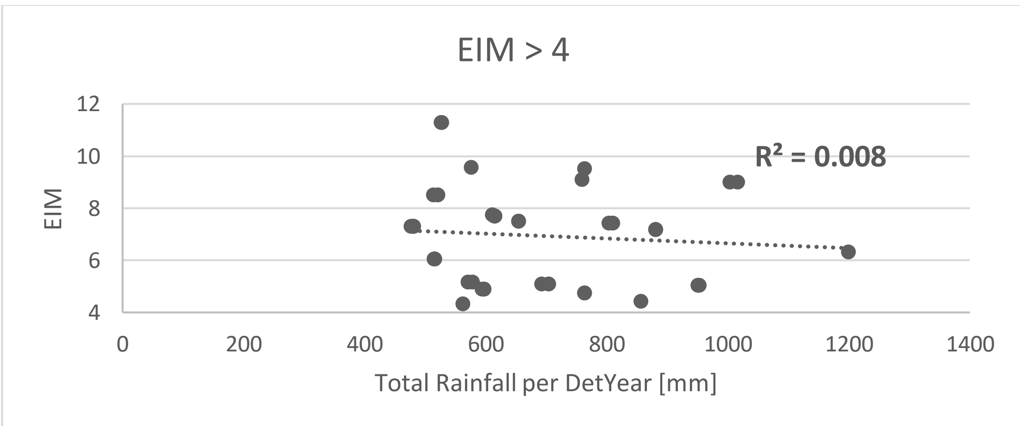


Figure 5.14 Rainfall vs EIM > 4

The summary output from the regression analysis is reported in Table 5.8.

In regression analysis, the objective is to figure out the relationship between the variables being analysed. In simple linear regression, the relationship is assumed as being linear or, in other words, a straight line:

$$Y = a + bX$$

A variable could be impacted by one or more factors, as the author is trying to assess. R^2 indicates the percentage of variation in the dependent variable (EIM) explained by the independent variable (Rainfall) or how well the regression model fits the data. The R^2 value ranges from 0 to 1, and a higher value indicates a better fit. The P-value, or probability value, also ranges from 0 to 1 and indicates if the test is significant. In contrast to the R^2 value, a smaller P-value is favourable as it indicates a correlation between the dependent and independent variables. Statistically speaking, the P-value is the probability of obtaining a result as extreme or more extreme than the one obtained in a random distribution. In other words, the P-value is the probability that the value of the independent variable, coefficient "b" in our regression model, is not reliable. These P-values are also given in Table 5.8.

Table 5.8 Summary of Regression Analysis

EIM category	R^2	P-value
EIM < 1	0.0025	0.64
1 < EIM < 2	0.029	0.33
2 < EIM < 4	3.2 E-05	0.9
EIM > 4	0.008	0.58

Results of this analysis are discussed in Section 5.7.

5.6.2 Rainfall – further analysis

The author carried out further investigation considering the average rainfall and the maximum rainfall over a shorter duration for the individual sites. In this case, a duration of 72 hours was selected, and the analysis was performed scanning 6-month periods, splitting therefore the deterioration year into a "warm season" (May to October) and a "cold season" (November to April). The 72-hour rainfall data was obtained by summing up daily rainfall (rain in day a, rain in day b, rain in day c, rain in day d, rain in day e …) for 3 consecutive days following the scheme (rain in a+b+c), (rain in b+c+d), (rain in c+d+e) and so on. Then the average and max 72h rainfall in each 6-month period were calculated.

Unlike using total rainfall over a year, as for the previous analysis, calculating the maximum 3-day reading and comparing it against the average 3-day reading within a 6-month period, highlights whether peaks of rainfall occurred, allowing an additional evaluation of whether a correlation exists between the rainfall and the EIM. In other words, are poorer values of EIM registered after a sudden storm or unexpected 3-day wet period? Tables showing the results of this analysis can be found in Appendix C. To assess whether a maximum value is indeed a peak, the ratio R between Maximum value and Mean value (R=Max 72hr/Mean 72hr) was calculated for the 6-month periods at all sites . The distribution of the R value is as follows (Table 5.9):

Table 5.9 Distribution of Peak:Mean ratio of 3-day rainfalls

% of occurrences	R
0	>12.28
15	>8
20	>7.5
25	>7.25
30	>7
50	>6
60	>5.5
75	>5
100	>3.24

By observing the percentages obtained, the author considers a "peak" value of Max. rainfall over a 3-day period for any particular site to be when the ratio R for that site is equal or greater than 7.25, which corresponds to the upper quartile of the available occurrences. Then the R value for these sites, for the relevant 6-month period, is compared to the corresponding EIM value.

Results of these analyses are discussed in Section 5.7.

5.6.3 Seasonal Deformation

As said before, the temperature data is used to represent seasonal deformation. A combination of this parameter and the rainfall will give indication of Dry and Wet Seasons. To find the thresholds for defining Dry and Wet Seasons, the months within the time frame analysed have been sorted according to both daily average temperature and daily average rainfall. Once again, the total months (96 occurrences) are split keeping the Deterioration year as reference; months from May to October (48 occurrences) are considered “Warm Season” while months between November and April (48 occurrences) are considered “Cold Season”. The 48 months belonging to each season are then sorted according to temperature and rainfall.

For the Warm Season, the months have been sorted from the warmest to the least warm based on “Average max Temperature” and on “Average daily Rainfall”. The warmest month appears to be July 2013 showing an average max temperature of 22.55 C° and September 2014 is the least rainy month in Warm Season recording 0.47 mm of rain.

For Cold Season, the months have been sorted following the same approach considering “Average minimum Temperature” and “Average daily Rainfall”. The coldest is December 2010 with -3.95 C° while December 2015 is the rainiest month with 5.8 mm.

To then highlight the driest months (warmest temperature and lowest level of rainfall) (Table 5.10), only the first 24 months per each ranking were considered. The months that were present in both the first 24 warmest and least 24 rainy were considered the driest months. For the same principle, the months with a place in the first 24 coldest and at the same time in the first 24 most rainy, were considered the wettest months (Table 5.11).

Table 5.10 Driest months

Dry		**Average Tmax [C°]**	**Average Rain [mm]**
2014	July	21.44	1.63
2013	August	20.06	1.73
2010	July	20.00	2.42
2016	August	19.70	2.13
2016	July	19.57	1.76
2011	July	18.98	2.11
2014	September	18.47	0.46
2016	September	18.33	2.42
2011	September	18.04	2.38
2015	June	17.83	1.24

Table 5.11 Wettest months

Wet		**Average Tmin [C°]**	**Average Rain [mm]**
2012	December	1.15	4.59
2016	January	2.20	4.59
2011	December	2.21	4.15
2015	January	1.09	4.13
2014	December	1.52	3.34
2010	November	1.48	3.33
2017	December	1.32	3.09
2016	November	1.63	3.03
2016	February	0.80	2.96
2011	January	0.29	2.92
2013	January	1.41	2.77
2010	February	-0.74	2.67
2012	January	1.70	2.56

According to the result summarised in the tables above, the threshold values for temperature and rainfall between Dry and Wet Seasons are set as follow (Table 5.122):

Table 5.12 Dry and Wet Seasons

Thresholds	**Monthly Rainfall R_{limit} [mm]**	**Max daily Temperature T_{limit}[C°]**
DRY	<2.25	>10
WET	>2.25	<10
MEDIUM	>2.25	>10
MEDIUM	<2.25	<10

Each Warm (May to October) and Cold (November to April) Season at each individual site was tested against the boundary in Table 5.12, to determine whether it was classified as Wet, Dry or Medium Season.

The formula applied presented 4 conditions as per Table 5.122

1) $R < R_{limit}$ & $T > T_{limit}$ => **Dry**
2) $R \geq R_{limit}$ & $T \leq T_{limit}$ => **Wet**
3) $R > R_{limit}$ & $T > T_{limit}$ => **Medium**
4) $R < R_{limit}$ & $T < T_{limit}$ => **Medium**

Tables with the analysis can be found in Appendix D. Results of this analysis are discussed in Section 5.7.

5.6.4 Type of Soil

Sites have been grouped first into Cohesive and Non-Cohesive soils. Within the two classes, sites have been reclassified according to the geology group (Table 5.6). EIM values against type of soil (Table 5.13) were observed site by site as, of course, this value doesn't change with time but only with position.

Table 5.13 Geology group vs EIM

ELR	Group	Geology group	EIM	Average EIM per Geology Group
GWS	Cohesive	S1	0.65	1.81
LEN3	Cohesive	S1	4.23	
LEN3 (2)	Cohesive	S1	4.23	
NEC2	Cohesive	S1	0.42	
SAC	Cohesive	S1	0.59	
TJC3	Cohesive	S1	1.06	
WCM2	Cohesive	S1	1.50	
NEC2 (2)	Cohesive	S#	0.98	0.98
HNR	Cohesive	S3	2.98	4.41
NAJ3	Cohesive	S3	5.54	
OWW	Cohesive	S3	3.90	
GTW1	Cohesive	S4	5.41	2.46
NGL	Cohesive	S4	2.89	
NGL (2)	Cohesive	S4	4.29	
SCU1	Cohesive	S4	2.57	
SPC1 (2)	Cohesive	S4	0.78	
SPC1 (3)	Cohesive	S4	2.14	
THH	Cohesive	S4	1.08	
THH (2)	Cohesive	S4	0.54	
NEC2 (3)	Non cohesive	S6	3.05	2.01
SPC1	Non cohesive	S6	0.98	
WCM1	Non cohesive	R	0.77	0.77
MLN3	Non cohesive	X	2.70	2.70

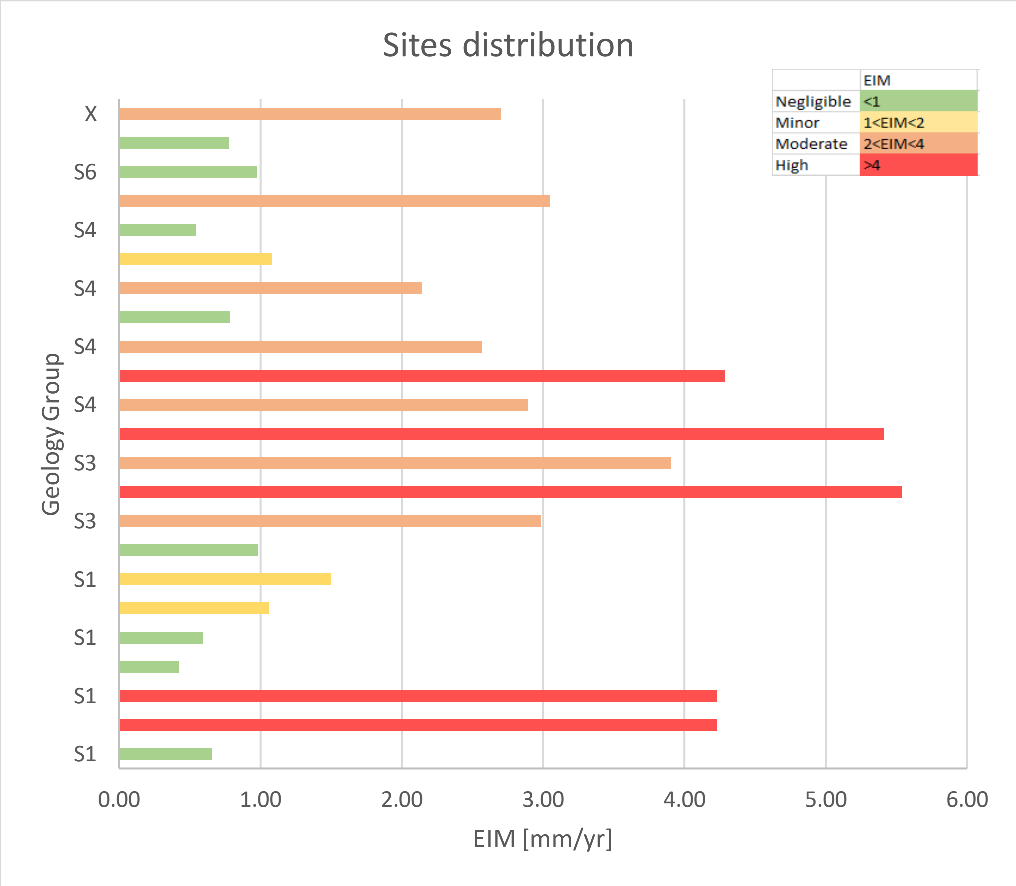


Figure 5.15 Site distribution

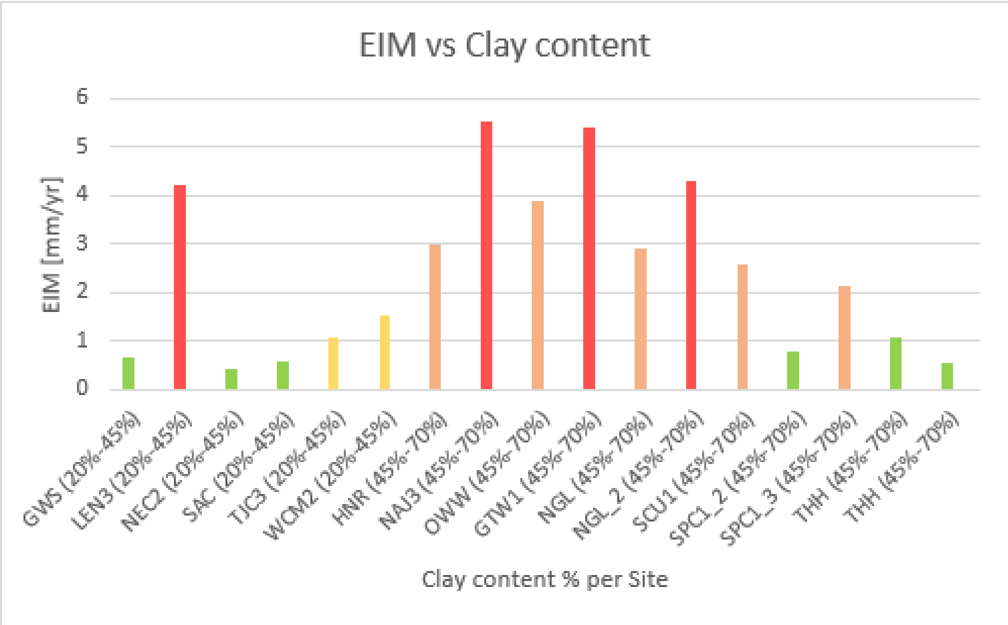


Figure 5.16 EIM per Clay content

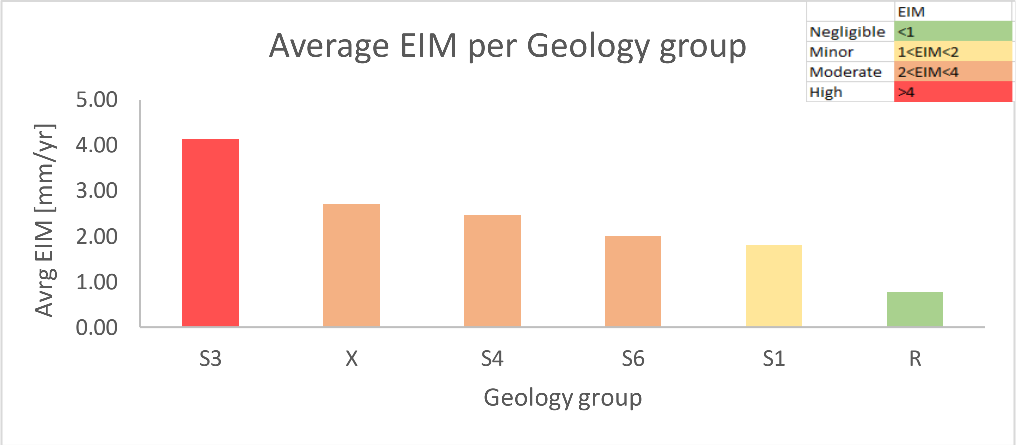


Figure 5.17 Average EIM per geology group

Results of this analysis are discussed in Section 5.7.

5.6.5 Vegetation

As per the type of soil, the vegetation doesn't change with time (or at least within the time frame considered). This has been verified checking pictures of vegetation for the past 10 years for each single site. The tool utilised to collect the information allows observations for different years and also for different times of the year.

Observation of vegetation, site by site, brought to the following classification (Table 5.14):

Table 5.14 Vegetation Groups distribution

ELR	Vegetation Group	ELR	Vegetation Group
GWS	High	OWW	High
LEN3 (start)	High	GTW1	High
LEN3 (end)	Low	NGL	Mix
SAC	Low	SCU1	Mix
TJC3	Low	SPC1 (start)	Medium
WCM2	Mix	SPC1 (end)	Low
NEC2	High	THH (start)	Low
NEC2_3	High	THH (end)	Medium
HNR	Mix	WCM1	High
NAJ3	High	MLN3	Mix

When plotting this, the geology group has been considered as well (Figure 5.18, Figure 5.19, Figure 5.20, Figure 5.21).

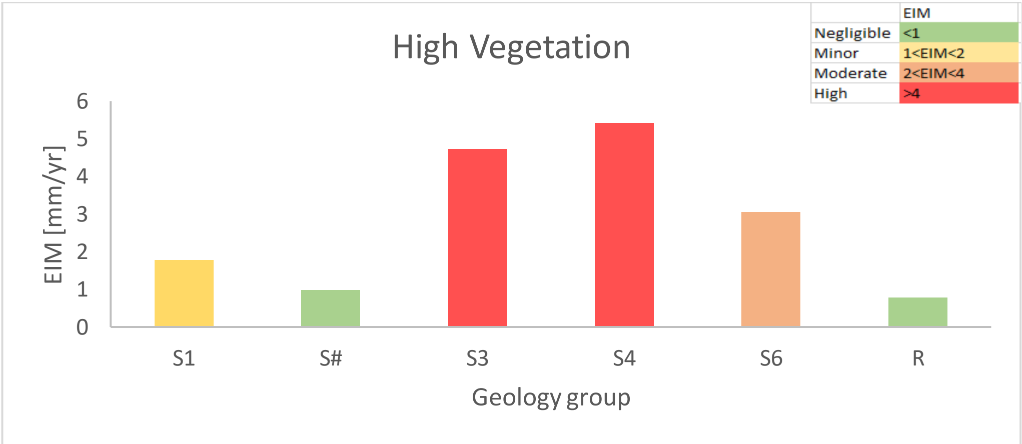


Figure 5.18 High Vegetation

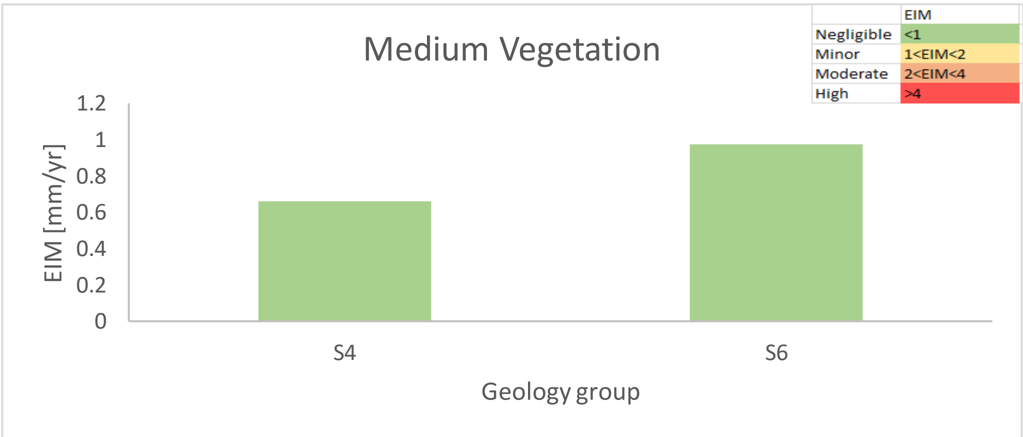


Figure 5.19 Medium Vegetation

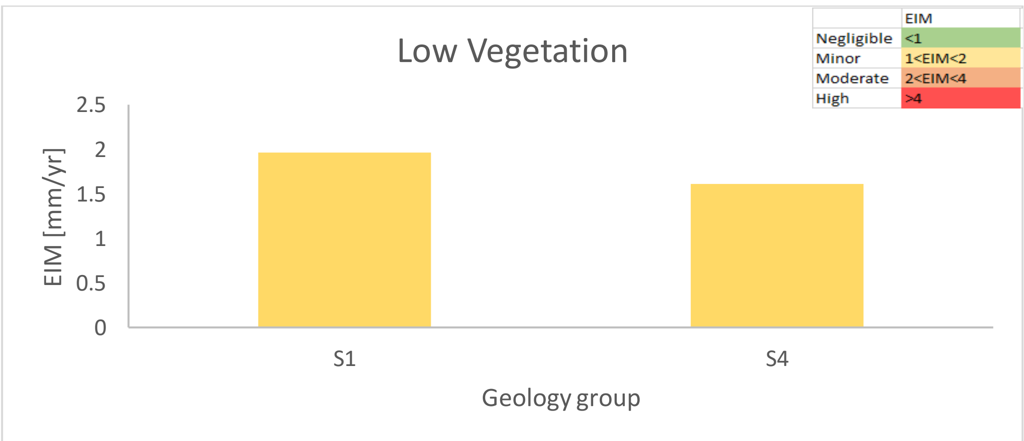


Figure 5.20 Low Vegetation

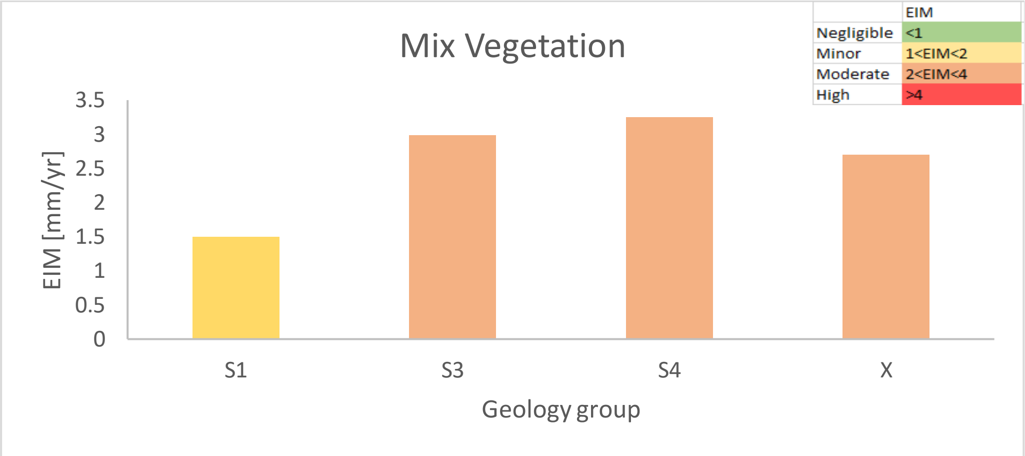


Figure 5.21 Mix Vegetation

Results of this analysis are discussed in Section 5.7.

5.7 Results

The data analysis undertaken aims to establish parameters criticality supposing EIM is influenced by them.

5.7.1 Rainfall

Regarding the analysis on total Rainfall, from Table 5.8 and the tables in Appendix C it can be assessed how EIM and Rainfall do not show correlation. There may be different reasons why the linear regression performs as bad as in this case. For example, the rainfall data could be noisy so to not explain the variation in the response. There may also be non-linear associations between Rainfall and EIM. The values in Figure 5.11, Figure 5.12, Figure 5.13, and Figure 5.14 indicate both the absence of a meaningful relationship and a lot of uncertainty.

Similarly, the analysis of heavy rainfall over a 72-hour period (characterized by the ratio R) against the EIM does not show any significant correlation between the two parameters. As can be seen from the tables in Appendix C, the peaks of 3-day max rainfall are scattered and correspond to both high and moderate values of EIM as well as minor and negligible EIM values. A direct correlation between rainfall and EIM is not shown by this analysis.

Nevertheless, it is well known, and strongly indicated by the literature review, that water content influences the stability of slopes (Section 3.2.5.2). Even though this is not visible by the two analysis techniques reported above, the author does not conclude that no correlation exists between rainfall and stability. We might expect sites to still be at risk when rainfall exceeds the normal content for that site.

As a result, in this thesis, it is not possible to define a precise critical value for the rainfall parameter or to establish ranges of values to reflect the parameter's influence.

5.7.2 Seasonal Deformation

The analysis of seasonal deformation has shown that, among 132 EIM values registered, of which 20 are "High EIM", 16 of these values (92.5%) have been registered as corresponding to, or immediately following, a Dry Season (Appendix D); the remaining 4 correspond to a Medium Season following another Medium Season. For other EIM categories, Dry Seasons have also been found along with Wet and Medium Season.

From the data analysis an inverse correlation can be observed: a Dry Season does not necessarily result in a High EIM value, but if a High EIM value is registered then this corresponds to, or follows after, a Dry Season. Furthermore, non-cohesive soils never deliver a High EIM value during or immediately after a Dry season. Summary tables are as follows (Table 5.15 and Table 5.16).

The heatmaps help to set critical values. A Dry Season is the most critical season at which High and Moderate values of EIM correspond. The Medium Season shows a mild influence on High values of EIM, while almost the 47% of Minor EIM values are in correspondence with this season. Wet Season shows no occurrence for High EIM and just 3 occurrences when a Moderate EIM is registered.

Table 5.15 Heatmap EIM per %Season

EIM	Occurrence	%Dry	%Medium	%Wet
Negligible	50	10.00	64.00	26.00
Minor	30	36.67	46.67	16.67
Moderate	25	68.00	20.00	12.00
High	27	74.07	25.93	0.00

Red = most frequent EIM, Green = least frequent, Other colours = intermediate frequency

Table 5.16 Heatmap Season per %EIM

Season	Occurrence	%EIM Negligible	%EIM Minor	%EIM Moderate	%EIM High
Dry	53	9.43	20.75	32.08	37.74
Medium	58	55.17	24.14	8.62	12.07
Wet	21	61.90	23.81	14.29	0.00

Red = most frequent EIM, Green = least frequent, Other colours = intermediate frequency

At first sight, this is unexpected as it is in contrast with previous study findings (Section 3.3) showing that wet winter months are the most critical. It is the author's opinion that the reason behind this result can be deduced from the presentation of the results as the graph in Figure 5.22. This plots all the assets and the parameters analysed: total average EIM per asset, type of soil, vegetation, seasonal deformation. A measure of the wetness of each site for every 6-month period is simplistically assigned by giving a numerical value to each season (Wet=3; Medium=2; Dry=1) and then averaging them for all the 6-month periods, obtaining what here is called Historical season.. All the seasons can be found in Appendix D. 22 assets are analysed in this thesis (Table 5.1) but only 20 points are plotted in Figure 5.22 since 3 of these assets are on the same track (NEC2) and so represent the same EIM, soil, vegetation, and season. Therefore, site NEC2 has only one point in the plot.

Figure 5.22 shows that the majority of the sites showing EIM values higher than 2 mm/y, and therefore spread towards drier periods of the years (see Table 5.15 and Table 5.16), are on cohesive soil with high shrink-swell potential. As found in the literature of this thesis (Section 3.5.1), cohesive soils are prone to shrinkage in warm dry weather. Among those critical sites with poor EIMs and cohesive soils with high swell/shrink potential, the highest EIMs are registered for sites with adjacent high vegetation (Section 3.6.3). It is striking to note that those sites with cohesive soils and low vegetation have low EIMs (~1.0). Heavy vegetation is indeed major source of drying and shrinkage cohesive soils in warm, summer, weather (Section 3.5.4.1).

For cohesive soils, when rain follows a shrinkage in dry weather, water enters the soil, particularly via cracks formed during shrinkage. This allows water to affect greater volumes of the soil than would be the case if water had to percolate through intact cohesive soil where permeability is low. Thus these soils may expand and soften rapidly (a consequence of the significantly different wetting and drying curves (Figure 3.13)) throughout a large volume of the soil, not just near-surface. Over a few years, repeated expansion on wetting and then contraction on drying, would likely result in the overlying track structure following this deformation. Due to the localisation of cracks, and due to the inherent soil heterogeneity, swelling is unlikely to be even along the length or across the width of the trackbed. If we determine to maintain the track alignment by tamping [133] in the late summer, so raising the line after a heavy shrinkage period, with the start of the rain season the soil swells again and the line would most probably be too high and in need of tamping again around January. The TRV recording the geometry will register these variations due to seasonal shrinking and swelling deformation, giving poor EIM values as a result.

For cohesive soils, the criticality is therefore assigned as follow (Table 5.17):

Table 5.17 Season Criticality for cohesive soils

Season	Criticality
Dry	Most critical
Medium	Critical
Wet	Less critical

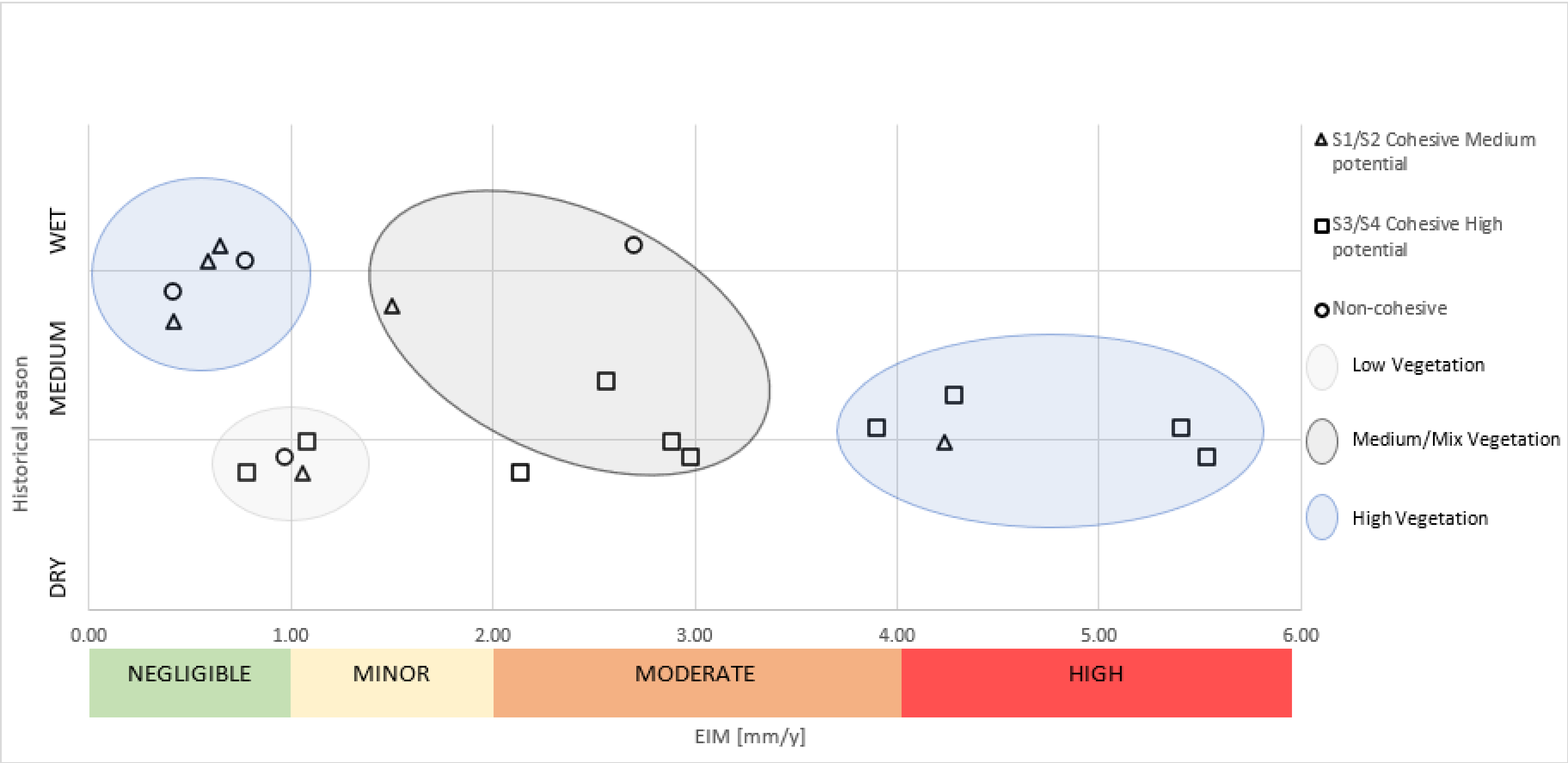


Figure 5.22 All sites distribution

5.7.3 Type of Soil

After the analysis the author decided that group R and the unknown S# should be discarded from the classification as only 2 and 7 records respectively are available. Hence, a classification of these groups would be highly imprecise.

From the graph in Figure 5.17 it can be observed how the geology group S3 shows the highest average EIM registered among all the groups. The heatmap also highlights this distribution (Table 5.18).

Table 5.18 Heatmap % of Type of soil per EIM

EIM	S1	S3	S4	S6	X
Negligible	62.5	13.2	43.5	53.8	42.9
Minor	12.5	15.8	17.7	30.8	14.3
Moderate	14.3	21.1	19.4	7.7	28.6
High	10.7	50.0	19.4	7.7	14.3

Figure 5.16 also points out the influence of clay fraction on EIM. The graph shows that soil containing a clay fraction between 45-70% most strongly influences the EIM value. This confirms S3's influence on the metric but also brings the attention to S4 being the other geology type with the same clay fraction.

The criticality is therefore assessed from 1 to 4 (Table 5.19), with 1 being the least critical and 4 being the most critical as follows:

Table 5.19 Type of soil Criticality

Geology Group	Criticality
S3	4
S4	3
S6	2
X	2
S1	1

5.7.4 Vegetation

Figure 5.18, Figure 5.1919,Figure 5.20, andFigure 5.21 show the distribution of the vegetation across the sites investigated. Once again, the heatmap (Table 5.20) helps with the identification of critical values.

Table 5.20 Heatmap EIM vs Vegetation

	% EIM Negligible	% EIM Minor	%EIM Moderate	% EIM High
Veg. High	40.4	42.9	39.4	67.5
Veg. Medium	16.9	5.7	3.0	0.0
Veg. Low	19.1	8.6	24.2	10.0
Veg. Mix	23.6	42.9	33.3	22.5

The criticality is assessed from 1 to 4 (Table 5.21); 1 the being the least critical and 4 being the most critical as follows:

Table 5.21 Vegetation Criticality

Vegetation	Criticality
Veg. High	4
Veg. Mix	3
Veg. Low	2
Veg. Medium	1

5.8 Conclusions

The data analysis highlighted how some of the parameters show a stronger link with the EIM than others. A range of critical values has been assigned for Seasonal Deformation, for which Dry Season is the most critical one. The geology group S3 is found to correspond to the highest percentage of EIM > 4 situations and therefore the highest critical value is assigned to it. Regarding the vegetation, High Vegetation is considered most critical; while regarding the Rainfall, the regression analysis didn't show an immediate correlation between the two variables. Nevertheless, the author doesn't suggest that stability is not influenced by the rain, but only that this correlation didn't stand out from the analysis undertaken. Rainfall must be considered alongside the other factors and not independently.

The plot in Figure 5.22 shows that, when the highest criticalities (cohesive soil S3 and S4, high vegetation, and traditionally dry season) are features for the analysed sites, then the poorest EIM values are recorded. Hence, attention is surely to be paid to the singular parameter criticalities, with alertness when all of them are combined.

The decision-makers could use the following decision tree (Figure 5.23) to assess the level of warning, based on the asset's properties and their combination.

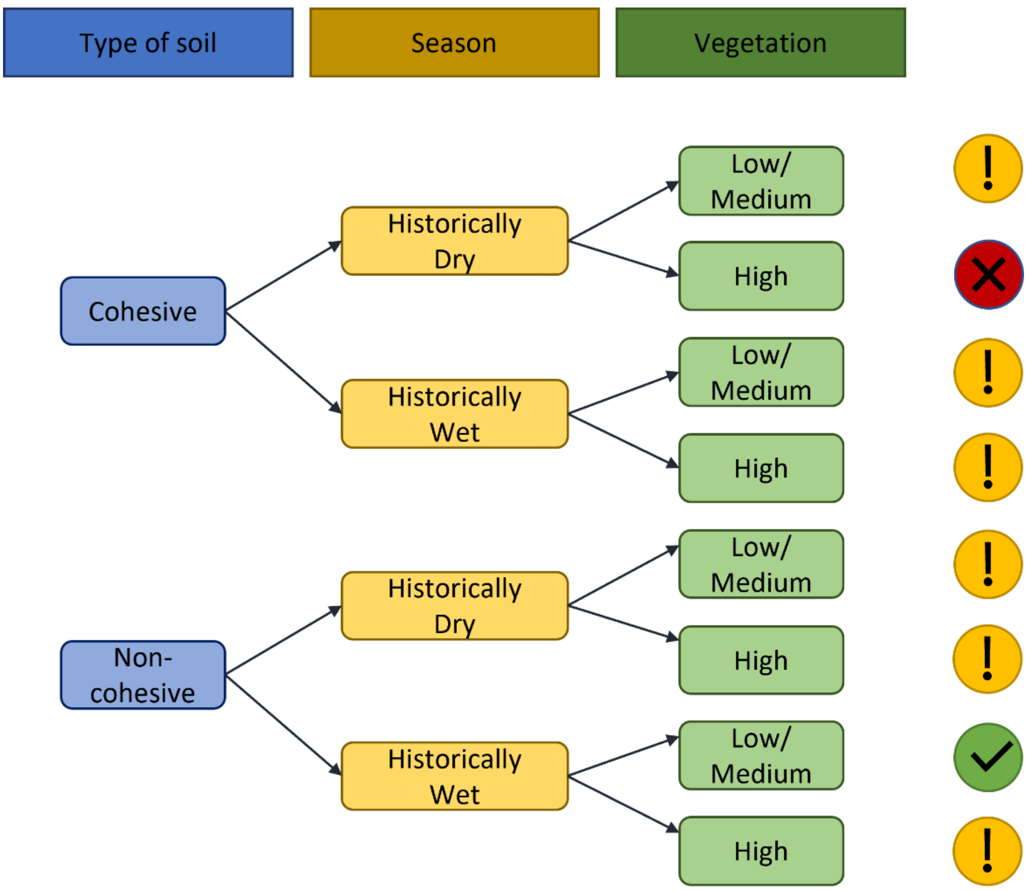


Figure 5.23 Decision tree

✓= Low criticality, × = High criticality, ! = intermediate criticality

The critical values have the final objective of assigning a weight to each parameter (to be used in Chapter 7), so to pairwise parameters and assess "importance" to each of them (see Chapter 6).

For the purpose of showing the steps of the MCDM tool developed, hypothetical values and critical values are assigned also to Rainfall starting from the value of rainfall <2.25 [mm/month] that has been found critical as it may be associated with a Dry Season. Also, by calculating a basic average Rainfall value for each EIM category the following table is obtain (Table 5.22).

Table 5.22 Average Rainfall values

EIM	Average Rainfall [mm/month]
Negligible	3.91
Minor	2.39
Moderate	2.15
High	1.70

Therefore, although imprecise and only for the purpose of developing the tool, the following critical values (Table 5.23) are adopted for Rainfall:

Table 5.23 Rainfall hypothetical Criticality

Average Rainfall [mm]	Criticality
$R \leq 1.7$	4
$1.7 < R \leq 2.25$	3
$2.25 < R \leq 3$	2
$R > 3$	1

5.8.1 Key developments

As stated at the beginning of this chapter, the aim of this research is to widen the understanding of the interaction between the embankment and the rail components. The link between causes and symptoms has led to a better technical comprehension of the asset criticalities which allowed the distribution of critical values per each parameter at each EIM level. Establishing priority of intervention brings to a more conscious decision-making process, this last also supported by the tool developed as outcome of this research work (Chapter 7).

The main limitation of the analysis is linked to the extension of the dataset available; data availability was indeed a critical point. Because of this, the number of parameters for the analysis decreased from the initial 13 identified from the literature to only 4 addressing the screening criteria. In the future the analysis would benefit from more parameters to be compared with EIM so to expand the evaluation of criticalities among the causes of potential embankment instability.

The results of this analysis lay down the basis of the calculations and steps for the tool developed.

CHAPTER 6. MULTI-CRITERIA DECISION-MAKING (MCDM) METHODS

LITERATURE REVIEW

6. Multi-Criteria Decision-making (MCDM) methods – Literature Review

In the previous Chapter 5, the methodology found correlations between parameters that can play a role in the embankment stability and in the EIM, a metric able to show track geometry displacement due to embankment instability.

The problem of identifying embankment failure at an early stage so to support the decision of intervention, presents several data, criteria, and objectives that need to be considered at the same time. Moreover, data come from technologies addressing different parts of the asset management strategy, looking at different parts of the issue, reading different measures. Therefore, the processing of these data may considerably vary, and the appropriate choice of the decision-making process becomes crucial [38]. To support this challenge Multi-Criteria Decision-Making (MCDM) methods are here evaluated. These methods provide a foundation for selecting, sorting, and prioritizing materials to arrive at a final solution [131]. Various mathematical techniques can be used for this process, all methods have their own pros and cons. The ones mainly used for problems in infrastructure asset management are reviewed in this chapter with the objective to find the most suitable for developing a proactive tool which is applicable to the scenario analysed in this thesis.

6.1 Introduction

Since the 1960s, Multi-Criteria Decision-Making (MCDM) has been an active research area. MCDM is an analysis that addresses structured planning and solving problems of mixed types. The purpose is to support decision-makers (DMs) facing such problems to solve them by establishing the best feasible solutions. Using MCDM can be said to be a way of dealing with complex problems by breaking the problems into smaller pieces. After some considerations and making judgements about smaller components, the pieces are reassembled to present an overall picture to the decision makers DMs.

MCDMs don’t lead to the same solution for every DM, on the contrary they are able to incorporate subjective information provided by the DMs themselves, which could lead to compromise solutions [132]. Often none of the alternatives that could be made perfectly achieve the pre-set goals. Yet, the alternative that best suits the goals can be selected by evaluating the different alternatives against a set of criteria [133]. The criteria help to differentiate among alternatives and select the most relevant one based on the DM's preferences. The compromise solution is, then, a feasible solution, which is the closest to the ideal [134].

Multi-criteria decision analysis presumes a trade-off between different criteria as in real-life design. This is required to improve different objectives simultaneously. A trade-off between the objectives is usually unavoidable. As a result, the optimal solution is not unique and belongs to a set of finite number of reasonable solutions. Eventually, the DM must choose only one of them, leading to the problem of ranking as the formal definition of the solution does not presume any preferences [135]. In the multi-criteria decision analysis, the ranking problem has been developed for the last 20 years, however, there are no universal approaches, and each method has its own background and principles.

Considering the number of MCDM methods available, the DM is faced with the arduous task of selecting an appropriate decision support tool, and often the choice can be difficult to justify. None of the methods are perfect nor can they be applied to all problems. Up to now, there has been no possibility of strictly deciding whether one method makes more sense than another in a specific problem situation [136]. Also, there is no unique and well-defined methodology that could be followed step-by-step from the beginning to the end of a decision aiding process.

In the following paragraphs the main methods used in the Architecture, Engineering and Construction (AEC) field are briefly reviewed. The reader is provided with reference to seminal papers at each paragraph for further background information.

6.2 MCDM general techniques

Generally, decision-making processes follow eight steps [137]–[140], which are presented in Figure 6.1:

(1) Choosing the appropriate decision-making method which fits the problem type is the first step in the decision-making process so to achieve the goal and the objectives. Methods must be compared based on different types of problems, highlighting their pros and cons.
(2) The requirements of a decision should be defined, based on expert's judgments or on any other technical restraints.
(3) Goals must be clarified and considered positively (i.e. in this case study the goal is "Find the right solution" and not "Find the non-correct options").
(4) Alternatives are what changes the preliminary condition into a preferred condition. Often none of the alternatives perfectly achieves the goals, but the alternative that best suits the goals can be found by differentiating them against a set of criteria.
(5) Defining and assessing the criteria according to specifications described by Baker et al.,:
    - able to distinguish among alternatives
    - complete enough to cover goal(s)
    - non-redundant
    - few in number
    - operational and meaningful.
(6) The decision method should be selected among the various available (relevant described in the following sections).
(7) The tool is used to choose the most suitable alternative.
(8) The answer provided by the MCDM tool must be checked.

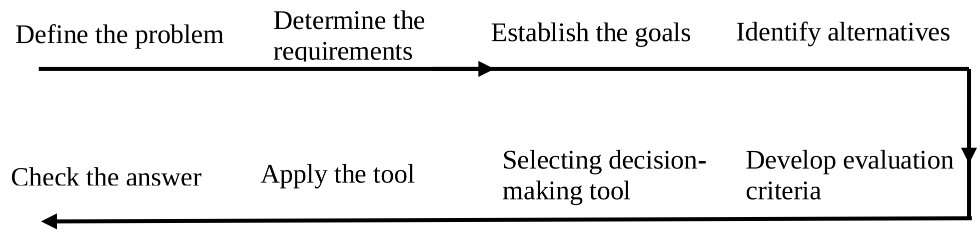


Figure 6.1 Common decision-making processes eight steps (after [8])

6.3 Glossary

The MCDM field adopts specific vocabulary and definitions. To make terminology clear and consistent, in this paragraph there is a non-exhaustive list of the most common terms in MCDM (more complete definitions can be found in [132], [133], [141],[142]–[144]).

Advantage	A benefit, gain, improvement, or betterment. Specifically, a beneficial difference between the attributes of two alternatives.
Alternatives	Two or more items from which one is to be chosen.
Attribute	A characteristic, quality, of one alternative.
Criterion	A decision rule or a guideline. Usually, a criterion represents conditions that eliminate an alternative from consideration if that alternative does not meet them
Design Team	Architects, engineers (structural, mechanical, electric, mechanical, etc.), designers, managers, and contractors arranged to provide design services in a specific project.
External Stakeholders	Community, regulatory agencies.
Factor	An element, part or component of a decision.
Internal Stakeholders	The design team plus the owner and users.
Lean Philosophy	Lean philosophy is about maximising customer 'value' while minimising waste.
Negative Iteration	"Iteration is essential for generating 'value' in design processes. However, not all iteration generates 'value'. Iteration that can be eliminated without 'value' loss is waste (negative iteration)". In other words, negative iterations do not add 'value'.

6.4 MCDM in the field of infrastructure management

Multi-Criteria Decision-making (MCDM) has been one of the fastest growing problem areas in many disciplines; there are few methods available, and their quality is hard to determine. Thus, the question "Which is the best method for a given problem?" has become one of the most important and challenging ones.

The literature on MCDM methods offers a large variety of techniques. To focus the attention on the field of interest (Architecture and Engineering Construction

AEC), a search on common MCDM methods was undertaken in title, abstract, and keywords utilizing the following databases: Elsevier, Springer, ScienceDirect, ResearchGate, Scopus and IEEExplore. The research included journal articles, thesis and conference proceedings concentrating mainly on the areas of infrastructure asset management. Figure 6.2 and Figure 6.3 graphically summarise the result of this search.

The following four MCDM methods were identified throughout the review:

1) Goal-programming and multi-objective optimisation methods,
2) Value-based method,
3) Outranking methods,
4) Choosing by Advantage.

As mentioned before, the following sections briefly review the methods giving reference for more detailed information, to then discuss the advantages and disadvantages of each method.

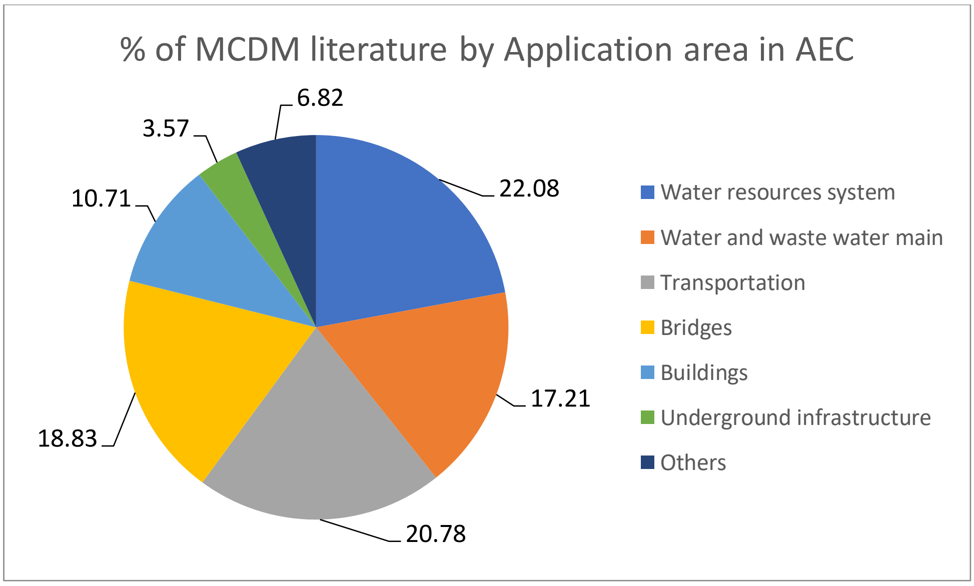


Figure 6.2 Percentage of MCDM papers in literature by application area

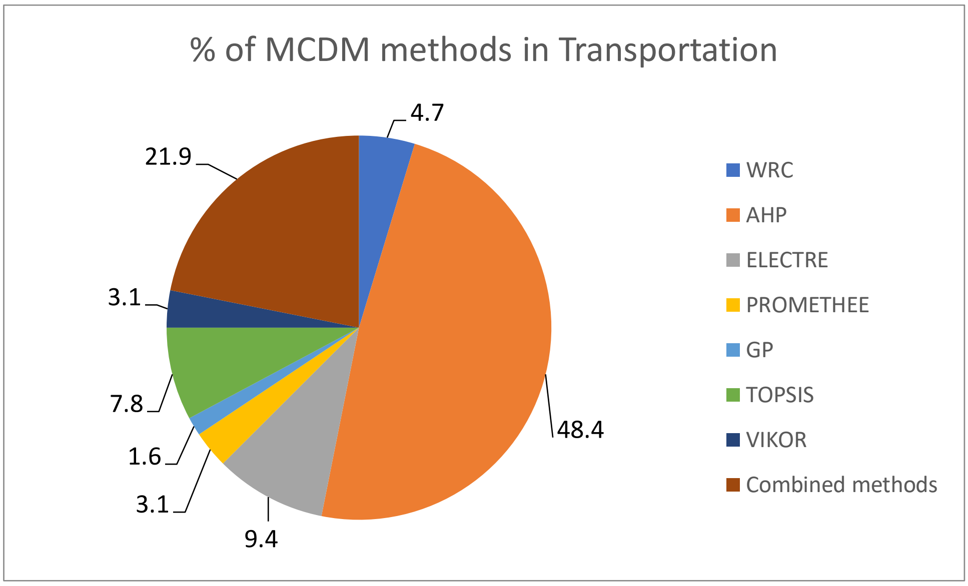


Figure 6.3 . Percentage of MCDM methods applied in Transportation field

6.4.1 Goal-programming and Multi-Objective optimisation methods

Goal programming (GP) can be considered as a branch of multi-objective optimization and it is one of the oldest multi-criteria decision-making techniques [19]. Whether goals are attainable or not, for each objective will be stated the level of optimisation that reaches a result as close as possible to the desired goal (the satisfying solution) [145],[146].

According to Belton and Stewart [132] these methods carry out Simon's satisficing concept [147]: improving the most important goal, until some satisfactory level of performance is achieved, and then shifting attention to the second most important goal, and so on. In this way, desirable or satisfactory levels are achieved for each of the factors. These methods are generally used when DMs find it difficult to express trade-offs or assigning weights to factors, but they are able to identify the criteria that would make the alternatives satisfying.

The application of a goal-programming methods can be summarised in the following steps [148]:

- Define factors and criteria for evaluation.
- Prioritise factors.
- Solve the optimisation problem.
- Arrive to a conclusion based on the results of this process.

After a decision is made, it is easy to explain why an alternative was chosen if the design team followed a multi-objective optimisation method.

6.4.2 Value-Based Methods

Value-based methods are focused on producing a value to represent the preference of the DMs. Value-based methods use explicit statements of acceptable trade-offs between different factors as a way of facilitating the construction of preferences. The most used value-based methods for AEC are two: the Analytic Hierarchy Process AHP and the Weighting-rating-calculating WRC [149].

6.4.2.1 Analytic Hierarchy Process AHP

The AHP method, proposed by Thomas Saaty (1980) [150], is a comparison-based method effective for dealing with complex decision-making. By reducing complex decisions to a series of pair-wise comparisons, and then synthesising the results, the AHP helps to capture both subjective and objective aspects of a decision. In addition, the AHP incorporates a useful technique for checking the consistency of the DM’s evaluations, thus reducing the bias in the decision-making process. The problem is first formulated as a hierarchy including several levels [148]. The first level represents the goal, the second level shows the main decision criteria, the next levels show the sub-criteria, and the last level indicates the alternatives. The elements of each level are compared by pair-wise forming a pair-wise comparison matrix. The relative weights found from each level are aggregated to identify the best alternative [151].

The AHP method is also used in cases where DMs are not comfortable with numerical scores but prefer qualitative or semantic scales (e.g., moderately important, highly important). The method can be summarised in the following steps [150]:

(1) Transform the problem in a hierarchy containing the goal, the alternatives for reaching it, and the criteria for evaluating the alternatives.

(2) Establish priorities among the criteria by making a series of judgments based on pair-wise comparisons of them.

(3) Establish priorities among the alternatives for each criterion based on pair-wise comparisons of attributes.

(4) Check the consistency of the judgments.

(5) Come to a final decision based on the results of the process.

In step (2), DMs are asked to indicate the strength of their preferences for one factor over another on the following scale (Table 6.1):

Table 6.1 AHP preference scale (after [150]**)**

Strength	Preference
1	Equally Preferred
3	Weak Preference
5	Strong Preference
7	Demonstrated preference
9	Absolute preference

After these judgments are done the pair-wise comparison matrix provides the weight of factors.

6.4.2.2 Weighting-rating-calculating WRC

The WRC can be described as a simplification of the AHP method. The WRC method can be summarized in the following steps ([141], [152], [153]):

(1) Identify alternatives.

(2) Identify factors and criteria for evaluation.

(3) Weigh factors.

(4) Rate alternatives for each factor.

(5) Calculate the value of each alternative and come to a final decision.

6.4.3 Outranking Methods

The outranking methods differ from value-based methods the result of the outranking methods is not a score for each alternative, but a determination that one alternative outranks the others. Alternative A is said to outrank another alternative B if, considering all the available information regarding the problem and the DM's preferences, there is a strong enough argument to support a conclusion that A is at least as good as B and that there is no strong argument to the contrary [154].

Outranking methods focus on pair-wise comparisons of alternatives to assess preferences, indifferences, and incomparability between alternatives. For example, if alternatives A and B are compared for a factor with a criterion i, several outcomes are possible: A can be preferred to B in regard to criterion i, B can be preferred to A, A and B can be indifferent, or A and B can be incomparable due to lack of information.

Outranking methods can be complex and less intuitive; hence, they are more suitable for expert analysis and for delivering detailed system insights [155].

The most prominent outranking approach is the ELECTRE (ELimination Et Choix Traduisant la REalité - ELimination and Choice Expressing the REality) family of methods, developed by Roy and associates in the 1960s. Less common are PROMETHEE, proposed by Brans from the Free University of Brussels [156], and the VIKOR method [157].

6.4.3.1 ELECTRE

ELECTRE methods are based on the evaluation of the "concordance" and "discordance" index to compare, for each pair of options A and B, whether option A is at least as good as B (concordance) or the strength of the evidence against this (discordance).

Roy created and first used ELECTRE in 1965, which is one of the best-known outranking methods. The original ELECTRE method (ELimination Et Choix Traduisant la REalite) is generally labelled as 'ELECTRE I', because several different versions of the ELECTRE method were subsequently given. All the versions of ELECTRE are based on the same fundamental concept of outranking relations between alternatives, taken two at a time, but are operationally somewhat different [139], [140], [155]. The main characteristics are as follows:

- Applicable even when there is missing information and incomparable alternatives.
- May or may not reach the preferred alternative and
- Applicable for both quantitative and qualitative attributes.

Roy [155] describes ELECTRE in the following steps:

(1) Define factors and criteria for evaluation.

(2) Weigh factors.

(3) Define scales for attributes.

(4) Calculate concordance and discordance indices.

(5) Construct outranking relations.

(6) Arrive at a final decision.

Due to complex computational procedure ELECTRE is time-consuming without using specific software.

6.4.4 Choosing By Advantages CBA

An emergent MCDM method in practice today is Choosing by Advantages (CBA) which has been successfully applied to many AEC projects [143], [149], [158]–[161].

CBA is a lean decision-making method developed by Suhr [162] aiming to guide people towards better decisions based on the advantages of alternatives through a reasoned and clear process.

According to this system, it is important to identify which factors will reveal significant differences between alternatives, not what factor will be important for the decision. CBA's fundamental rule is to initially identify only advantages of alternatives, so as to avoid double-counting and omissions. Advantages and disadvantages are the same except for their perspective. The second rule is to separate cost from value as cost is seen as a constraint, not a factor, and thereby should be given special attention when deciding. CBA bases decisions on the importance of the differences between the advantages of alternatives, rather than the importance of factors themselves, as is the case in other MCDA methods [133]. This distinction helps DMs to limit personal judgment by providing a point of reference. For moderately complex decisions, the method can be summarized in 7 steps:

(1) Stakeholders identify alternatives likely to yield important advantages over other alternatives.
(2) Stakeholders define factors to evaluate attributes of alternatives.
(3) Stakeholders need to agree on the criteria for each factor. Criteria can be either a desirable or a mandatory decision.
(4) Stakeholders summarise the attributes of each alternative.
(5) Stakeholders decide the advantages of each alternative.
(6) Stakeholders decide the importance of each advantage by stating their preferences for the advantages.
(7) Stakeholders evaluate cost data.

6.4.5 MCDM preference in the Architecture and Engineering Construction (AEC) industry according to literature

In the MCDM application for Architecture and Engineering Construction (AEC) industry, a decision-making process is more likely to be embedded in a wider process of problem structuring and resolution, rather than be found as a stand-alone problem; usually the problem of defining alternatives, factors, and criteria is as hard as deciding which alternative to select [131].

A clear preference for using value-based methods in the AEC industry exists in the literature, especially the AHP method, which is often used and documented in the literature for choosing a sustainable alternative. Goal-programming and outranking methods are found less in the literature compared with AHP. Applications of CBA are found only within the lean construction community and very few have the environmental perspective included in the analysis. Lean practices/processes are focused on increasing value to the costumer while reducing waste, which helps to achieve better design and construction solutions using fewer resources [136].

6.5 Relevant differences between MCDM methods

MCDM methods are potentially capable of improving the transparency, analytic rigour, and auditability of the decisions. MCDM methods have common aspects that differentiate them and their decision process: creating transparency, building consensus, continuous learning. These aspects and how the four main MCDM categories address them are presented in the following Table 6.2. For further background information, referral should be made to the seminal review papers [149], [162]–[167]

Table 6.2 Relevant differences between MCDM models

Creating Transparency	**Multi-objective optimisation**	**Value-based method**	**Outranking Method**	**Choosing By Advantage**
Various decision-making methods have different degrees of transparency, the lack of it prevents any meaningful link between the output and the assessment. To create transparency for trade-offs between attributes, characteristics of the alternatives should be identified and qualitative attributes should be described. Problems with the weighting approach occur when the chosen criteria are redundant and when key criteria are not included, resulting in the phenomena called double counting and undercounting respectively. The decision team usually wants to be cost conscious and at the same time achieve project objectives. Taking cost into account does not imply selecting the least costly alternative, but rather the alternative that yields the best project outcomes within financial constraints.	It is easy to explain why an alternative was chosen. However, these methods avoid making explicit trade-offs and it is not clear what the trade-offs are among attributes of the alternatives. The differences between alternatives are not highlighted. This is risky when multiple issues need to be considered at the same time for a design. Cost can be a factor, and could be ranked first, guiding the selection of an alternative merely by cost. This method only results in a solution (the alternative chosen) without providing a ranking of all the alternatives; this makes it impossible to analyse value vs cost.	The score of the alternatives provides a rationale for choosing an alternative. Objective or subjective, quantitative or qualitative information is considered during the decision process. Any level of detail about the goal can be listed or structured so that the overview of the focus is easily represented. However, the differences between alternatives may not be highlighted if high weights are given to factors that do not differentiate between alternatives, which can mask the true difference between alternatives. Moreover, cost can be a factor, which allows for mixing cost and the value of the alternatives.	DMs can only provide an outranking relation among the alternatives. Therefore, it is not clear what the trade-offs were among attributes of the alternatives. This is especially true when more factors and alternatives are incorporated in the decision. This method provides an outranking relation, but not an overall value of the alternatives making it impossible to analyse value vs cost.	DMs can provide a rationale for the decision. Using the differences between alternatives, it highlights the trade-offs among attributes of the alternatives. Cost cannot be a factor, so it doesn’t drive the decision. In contrast to other methods, it is possible to make an analysis of value vs. cost.

Building Consensus	Multi-objective optimisation	Value-based method	Outranking Method	Choosing By Advantage
Consensus results in the best solution that the group can achieve choosing the best available design alternative at that time with the available knowledge. Building consensus to move forward and implement the chosen alternative requires co-operation among members; reaching the consensus then avoiding unnecessary iterations. The consensus on the best alternative is based on the available information; information can change over time, therefore, decisions may also change over time.	The decision team needs to reach consensus on the weight of the factors and the ranking of attributes. Agreeing on which factors and criteria have more weight may be a source of conflict. Factors can be weighted by basing the weights on previous experiences potentially missing the specific context of the decision. The decision team would need to assign weights to factors and criteria first, which is a subjective task. Although the following calculation is objective, it is based on subjective scales.	The decision team needs to reach consensus on the weight of the factors and the ranking of attributes. Agreeing on which factors and criteria have more weight may be a source of conflict. The weight of the factors can be based on previous experiences. The decision team would need to assign weights to factors and criteria first, which is a subjective task. Then, they would need to rate the alternatives for each factor, according to their attributes, which is also a subjective task. Although the calculation that follows is objective, it is based on subjective scales.	The decision team needs to reach consensus on the weighting of factors and the rating of attributes. Even when the factors are not used in the same way as for the value-based methods, they are required in order to construct the outranking relationship. However, reaching an agreement on which factor and criterion is most important is, indeed, subjective. In terms of managing subjectivity in this case, the decision team would need to assign weights to factors and criteria, and rank attributes, which may be a subjective task, and then compare the alternatives, which is a more objective task.	The decision team needs to reach consensus on the weighting of advantages and criteria for evaluation. The method may help in building consensus when the decision team agrees on the advantages of the alternatives, based on the difference between their attributes. However, it may be challenging for the decision team to agree on the importance of the advantages. In terms of managing subjectivity in CBA the decision team compares known attributes, which is an objective task, and then weighs the advantages, which is a subjective task.

Continuous Learning	Multi-objective optimisation	Value-based method	Outranking Method	Choosing By Advantage
Providing a rationale for the final decision and documenting the decision-making process will allow stakeholders to learn and improve future decisions. In infrastructure management, due to the iterative characteristic of the decision-making process, learning is desirable in order to improve the design, reduce negative iteration, and learn for future projects.	this method does not provide an overall ranking of the alternatives. Therefore, the decision team may not have a clear and shared understanding of what the value of the discarded alternatives is. This may result in missing valuable information for improving the design. If a new alternative is added, the decision team can compare it just with the selected alternative using the previous ranking of factors. This may be convenient, but the team may miss the opportunity to look at all the differences between alternatives.	this method provides an overall ranking of the alternatives. Therefore, the decision team can have a score representing the value of the discarded alternatives, however, if a new alternative is added, its attributes need to be rated. If the team applies the 'swing weight' method, the weights may change to better represent the decision context. Then a calculation is required to evaluate all alternatives in accordance with the new weights of the factors.	this method does not provide an overall ranking of the alternatives. Therefore, the advantages of the discarded alternatives may not be visible. If a new alternative is added, all outranking relationships need to be calculated for that alternative. If a new factor were to be added, the decision team would need to assign a weight to it and recalculate the outranking relationship of all alternatives. Multiple decisions may not be compared using this method.	this method provides an overall ranking of the alternatives and provides advantages of the discarded alternatives. If a new alternative is added, the decision team needs to describe and assign a weight to its advantages. If a new factor and criterion is added, the decision team needs to assess which alternatives have advantages for that factor.

6.6 Selecting the decision-making method

After reviewing different sources, the author concludes that in literature is missing a general overview to support DMs in the selection of one decision-making method over another in the monitoring and maintenance management field. The author therefore proceeded to select the preferred method basing the choice on the information gathered and on how the specific methods could apply to the case study analysed in the thesis.

6.6.1 Why are methods discarded?

- Goal Programming is a pragmatic programming method that can choose from an infinite number of alternatives. One of its advantages is that it has the capacity to handle large-scale problems. Its ability to produce infinite alternatives provides a significant advantage over some methods, depending on the situation. A major disadvantage is its inability to give weight coefficients. The weighting part is an important step for the study in this thesis. Indeed, to reflect the understanding of parameters criticalities, ranking the instability drivers helps in setting priority of interventions. Goal programming has seen applications in different fields such as production planning, scheduling, health care, portfolio selection. Many of these applications have been used in combination with other methods to accommodate proper weighting. By doing so, it eliminates one of its weaknesses while still being able to choose from infinite alternatives.
- Outranking methods (ELECTRE family methods) can be a good option for DMs who want to consider all alternatives and prefer to outrank the alternatives instead of eliminating them. This method is not suitable for cases where the alternatives are widely different so that expressing preferences becomes almost impossible as they are not comparable in those cases.

  ELECTRE methods are relevant when facing decision situations in which the DM wants to include in the model at least three criteria. However, aggregation procedures are adapted more frequently in situations where decision models include more than five criteria (up to twelve or thirteen).

Moreover, when a strong heterogeneity related with the nature of evaluations exists among criteria (as per the case study of this research: Type of Soil, Vegetation, Rainfall, Seasonal Deformation), this makes it difficult to aggregate all the criteria in a unique and common scale. Outranking methods can be complex and less intuitive; hence, they are more suitable for expert analysis and for delivering detailed system insights. Due to complex computational procedure ELECTRE is time consuming without using specific software.

- Choosing by advantage (CBA), compared with other methods, is relatively newly introduced. CBA enables DMs to concentrate on what is important: the advantages (beneficial difference) that each alternative could deliver to stakeholders. CBA creates an open, transparent, and auditable decision process for design and construction work accounting for the complexity of most. CBA is well able to handle both objective and subjective data within a single decision process. According to this system, it is important to identify which factors will reveal significant differences between alternatives, not what factor will be important for the decision itself [4], [37], [38]. All the steps of this methods are indeed about identifying benefits and characteristics of the alternatives and so, in the case of this research work, the characteristics of the four site interventions linked to the EIM value (Table 4.3). The benefit of these interventions is mainly assessed against cost of the intervention itself. Indeed, the alternative "routine track maintenance" (EIM < 1 [mm/yr]), is much more beneficial from an operational and financial point of view than the alternative "remedial solution" (EIM > 4 [mm/yr]), which needs to account for cost to workforce, closure of the line, speed restriction, possession of the area etc. But an important rule of this method is to separate cost from value as cost is seen as a constraint, and thereby considered only as last step of the analysis. In the case of embankment maintenance, when the analysis starts with the evaluation of the alternative options, as in CBA, considering cost at the end of the analysis wouldn't give the right importance to this crucial aspect and decision would be made based on other less important characteristics of alternatives (i.e. aesthetic, administrative etc.).

As a conclusion, the method chosen is the Analytical Hierarchy Process (AHP).

6.6.2 Why is AHP selected?

As outlined in previous sections, AHP can provide DMs with a robust solution. The biggest advantage of this method is to simplify the problem and to always consider the DMs' preference and experience. Basically, AHP is a method of: breaking down a complex, unstructured situation into its components; arranging these components into a hierarchic order (levels); synthesise the judgments to determine which components have the highest priority and should then be acted upon to influence the outcome of the situation. The complex problem of identifying the most suitable intervention on an examined site can be de-structured adopting this method into more manageable levels, using a pair-wise approach to set components priorities. This gives the appropriate relevance to all the alternatives as well as to all the criteria involved in the process. In transport infrastructure indeed, any decision issue consists of various criteria and frequently these criteria have sub-criteria as well. For these criteria and sub-criteria either objective or subjective considerations or either quantitative or qualitative information can be evaluated with the AHP technique. Any level of detail can be listed or structured within this method.

The AHP generates a weight for each evaluation criterion according to the DM's pair-wise comparisons of the criteria. The higher the weight, the more important the corresponding criterion. Next, the AHP assigns a score to each alternative according to the DM's pair-wise comparisons of the alternatives based on that criterion. The higher the score, the better the performance of the alternative with respect to the considered criterion. As a last step, the AHP combines the weights of the criteria and the alternative scores to determine a global value for each alternative, and a consequent ranking. The global score for a given alternative is a weighted sum of the scores obtained with respect to all the criteria.

The decision is made among a set of evaluation criteria and a set of alternative options. Some of the criteria could be contrasting; for this reason, it is not true, in general, that the best option is the one which optimises each single criterion. Instead, the decision is the one which achieves the most suitable trade-off among the different criteria.

On the other hand, although every single evaluation is very simple, since it only requires the DM to express how two options or criteria compare to each other, the load of the evaluation task may become unreasonable. The AHP may require many evaluations by the user, especially for problems with many criteria and options. In fact, the number of pair-wise comparisons grows exponentially with the number of criteria and options and many comparisons may be needed to build the score matrix. However, to reduce the DM's workload, the AHP can be completely or partially automated by specifying suitable thresholds so as to automatically decide some pair-wise comparisons.

Regarding double-counting, none of the MCDM approaches studied by the author in this review has a clear way of managing double-counting, nevertheless the interdependency between the factors involved in this research study helps avoiding this phenomenon. Moreover, in AHP factors with similar characteristic could be grouped before weighing them avoiding counting twice those aspects common to different factors, if any.

6.7 Analytic Hierarchy Process (AHP)

6.7.1 AHP Structure

AHP theory has four axioms. It is important to satisfy these axioms in order to successfully apply the AHP technique to a decision-making problem.

Axiom 1- Reciprocal Comparison: The intensity of the preferences of the DM must satisfy the reciprocal condition: If A is x times more preferred than B, then B is 1/x times more preferred than A.

Axiom 2 - Homogeneity: The preferences are represented by means of a bounded scale.

Axiom 3 - Independence: In expressing preferences, criteria are assumed independent of the properties of the alternatives.

Axiom 4 - Expectations: To making a decision, the hierarchic structure is assumed to be complete.

The primary goal of the AHP is to select an alternative that best satisfies a given set of criteria out of a set of alternatives or to determine the weights of criteria in any application. AHP scales the weights of attributes at each level of the hierarchy with respect to a goal using DMs' experience and knowledge in a matrix of pair-wise comparison of attributes. The usual application of AHP is to select the best alternative from a discrete set of alternatives.

6.7.2 Process of AHP – Brief Statistic Steps

AHP provides a way to rank the alternatives of a problem by deriving priorities and can assist with identifying and weighting selection criteria, analysing the data collected for the criteria and expediting the decision-making process.

The AHP is based on a matrix of pair-wise comparisons between criteria, and it can be used to evaluate the relative performance of decision alternatives with respect to the relevant criteria.

AHP has three main steps:

1) structuring the hierarchy.
2) pair-wise comparisons (determining the weights).
3) decision phase (selection of the best alternative among the others).

The AHP is a methodology to rank alternative courses of action based on the DM's judgments concerning the importance of the criteria and the extent to which they are met by each alternative. To solve a decision problem with AHP, there are some steps that need to be followed. An interesting and thorough review of these steps can be found in [166].

6.8 Conclusion

The literature review undertaken helped identify the most used MCDM methods used in the transport infrastructure field. Among them, AHP is identified as the most suitable approach to follow for the development of the tool. Based on this approach the problem faced in this thesis will be simplified into 4 different levels:

1) Objective: "Choose the best action for the examined site"

2) Criteria: Seasonal Deformation, Type of Soil, Vegetation, Rainfall
3) Sub-criteria: Dry Medium Wet; S3 S4 S6-X S1; High Mixed Low Medium; $R \leq 1.7$ - $1.7 \leq R \leq 2.25$ - $2.25 \leq R \leq 3$ - $R > 3$
4) Alternatives: Clear, Green, Amber, Red.

A detailed description of these levels and of the tool developed can be found in the next Chapter 7.

CHAPTER 7. THE TOOL

7. The tool

After a literature review on the most common MCDM approaches to AEC problems, Chapter 6 concluded that the Analytic Hierarchy Process (AHP) method is the best fit for the challenge of the topic of this thesis. Thus, the objective of this chapter is to implement this method in a tool that will support decision-makers in identifying the most suitable intervention among a list of alternatives based on the actual condition of the asset examined.

In this chapter the MCDM tool created in Excel will be used to simulate a real case study and the steps of the calculation will be displayed by the use of commented screenshots.

In principle, the tool should and could be used to assess new embankment sites for which the EIM is not necessarily yet known. In this chapter though, for the purpose of validating the tool, a case study for which the EIM is already known will be used. This to try and demonstrate that the tool delivers the same action suggested by the EIM, but only with the use of the 4 parameters analysed.

7.1 Reciprocal Comparison

To apply this method and to comply with the axioms of its theory (Section 6.7) a "reciprocal comparison" of the 4 parameters (Seasonal Deformation, Type of Soil, Vegetation, Rainfall) needs to be undertaken so to assess preferences. The parameters in the tool are named "Criteria" while their groups (i.e. Dry, Medium, Wet for Seasonal Deformation) are referred to as "Sub-criteria" as per usual AHP terminology (see Glossary Section 6.3).

In the case of this thesis, a parameter, or criterion, is more important when it is more likely to create instability. The parameters that have shown a better correlation with the EIM are considered more important than others. The importance is assigned following the scale in Table 7.1 according to the findings of the analysis undertaken and were described in Chapter 5. Considering that, having shown the strongest correlation with the EIM, Seasonal Deformation is the most important criterion, followed by Type of Soil, Vegetation and Rainfall for last, importance and intensity are assigned as follow (Table 7.2).

The scale of comparison (Table 7.1) used in the tool is the following:

Table 7.1 Scale of comparison

Scale	Importance
1	Equal importance
2	Equal to moderate
3	Moderate importance of one factor over another
4	Moderate to strong
5	Strong or essential importance
6	Strong to very strong
7	Very strong importance
8	Very strong to extreme
9	Extreme importance

As explained in Chapter 5, the Rainfall didn't give statistically significant results as the data are inconclusive. Nevertheless, to better show the tool potential, the author decided to still include this parameter in the following steps. Comparison between the 4 criteria implies 6 combinations.

In Table 7.2 and Table 7.3 comparison of the criteria and sub-criteria are provided. Each time, the criterion defined as "more important" of the two being compared is shown in the third column and an intensity (Table 7.1) of this importance is also assigned.

Table 7.2 Comparison of the 4 criteria

Criterion A	Criterion B	More important	Intensity
Rainfall	Seasonal Deformation	Criterion B	9
Rainfall	Vegetation	Criterion B	3
Rainfall	Type of Soil	Criterion B	6
Seasonal Deformation	Vegetation	Criterion A	6
Seasonal Deformation	Type of Soil	Criterion A	3
Vegetation	Type of Soil	Criterion B	3

As said, these comparisons are based on the data analysis undertaken in Chapter 5, therefore this assessment is generic and always valid, it remains constant for each site analysed and for each case.

Regarding the Sub-criteria, the importance and intensity (as per Table 7.1) of each sub-criterion is assign based on the data collected for the specific site under examination: a sub-criterion is more important than another one when it shows the actual condition of the asset from the data collection. The intensity will be assigned considering the criticality ranking assigned as a result of the data analysis (Section 5.7). For instance, if the Type of Soil is S6 then the sub-criterion S6 will be the most important among the type of soil groups, while the rest will have equal importance and the intensity will be distributed following the criticality ranking on Section 5.7.3, as follows:

Table 7.3 Comparison of sub-criteria

Sub-criterion A	Sub-criterion B	More important	Intensity
S3	S4	Sub-criterion A	1
S4	S6	Sub-criterion B	3
S6 and X	S1	Sub-criterion A	3
S3	S6	Sub-criterion B	6
S3	S1	Sub-criterion A	1
S4	S1	Sub-criterion A	1

7.2 Alternatives

The alternatives represent the interventions/actions linked to each EIM. In Chapter 4, Table 4.3 describes the actions to be undertaken case by case according to the metric value obtained. If, as it is envisaged to demonstrate, there is a link between criteria and the EIM, it is then appropriate that the alternatives introduced into the tool are the same as suggested by the previous AECOM study. Also, AECOM have used a Red / Amber / Green (RAG) rationale for determining trigger values for the monitoring phase. Alternative, Trigger conditions and Actions are described in Table 7.4 below:

Table 7.4 Alternatives of the tool

Alternative	Trigger Condition	Action
Alternative 1	Clear Condition represents movements within the values expected.	Any track roughness would be addressed during the course of routine track maintenance and would not therefore be identified as an earthwork problem
Alternative 2	Green Condition represents conditions in excess of the values expected	May or may not be identified as an earthworks issue, could be dealt with through track maintenance assuming rates of deterioration do not increase.
Alternative 3	Amber Condition represents movements in excess of the values expected, that present a risk of structural damage	Moderate movement which is more like to be identified and related to a potential earthwork issue.
Alternative 4	Red Condition represents a possibility of structural damage and a risk of structural instability	High risk is judged to be the point at which it is obvious that there is a serious earthwork issue that requires regular track maintenance (very regular for high line speed) to maintain track geometry and will require a long-term earthwork remedial solution.

7.3 Use-Case

For the simulation, a site on the Bethnal Green and King's Lynn Line (BGK) is chosen. This site has been randomly picked from a list of embankment sites that AECOM analysed and for which the EIM has been calculated. As mentioned before, knowing the EIM is essential to validate the quality of the alternative proposed as a result of the calculation within the tool.

The time frame considered is between May 2016 and April 2017 and data sets were collected for this deterioration year (DetYr). A summary of the site data collection is as follows (Table 7.5 and Table 7.6):

Table 7.5 Data collection for BGK embankment site

Datum	Measure
Temperature (Average over DetYr)	9.9 [C°]
Type of Soil	S4
Vegetation	Low
Rainfall (Average per DetYr)	1.3 [mm/day]
EIM	1.14 [mm/yr]

Table 7.6 Criteria for site BGK

Criterion	Sub-criterion	Criticality
Seasonal Deformation	Medium	Critical
Type of Soil	S4	3
Vegetation	Low	2
Rainfall (Avg per DetYr)	$R \leq 1.7$	4

The hierarchy of the problem could be diagrammed as shown below (Figure 7.1):

Figure 7.1 Hierarchy of the problem

7.3.1 Pairwise comparing the criteria and sub-criteria with respect to the Objective

As described in Section 7.1 the comparison between criteria is assigned as per Table 7.2 Comparison of the 4 criteria

The comparison between all the sub-criteria has the following results (Table 7.7,Table 7.8,Table 7.9,Table 7.10):

Table 7.7 Seasonal Deformation Sub-criteria Reciprocal Comparison

Sub-criterion A	Sub-criterion B	More important	Intensity
Dry	Medium	Sub-criterion B	6
Dry	Wet	Sub-criterion A	1
Medium	Wet	Sub-criterion A	3

Table 7.8 Type of Soil Sub-criteria Reciprocal Comparison

Sub-criterion A	Sub-criterion B	More important	Intensity
S3	S4	Sub-criterion B	3
S3	S6 and X	Sub-criterion A	1
S4	S6 and X	Sub-criterion A	3
S3	S1	Sub-criterion A	1
S4	S1	Sub-criterion A	6
S6 and X	S1	Sub-criterion A	1

Table 7.9 Vegetation Sub-criteria Reciprocal Comparison

Sub-criterion A	Sub-criterion B	More important	Intensity
High	Mix	Sub-criterion B	1
High	Low	Sub-criterion B	6
Mix	Low	Sub-criterion B	3
High	Medium	Sub-criterion A	1
Mix	Medium	Sub-criterion A	1
Low	Medium	Sub-criterion A	3

Table 7.10 Rainfall Sub-criteria Reciprocal Comparison

Sub-criterion A	Sub-criterion B	More important	Intensity
$R \leq 1.7$	$1.7 < R \leq 2.25$	Sub-criterion A	6
$R \leq 1.7$	$2.25 < R \leq 3$	Sub-criterion A	7
$1.7 < R \leq 2.25$	$2.25 < R \leq 3$	Sub-criterion A	1
$R \leq 1.7$	$R > 3$	Sub-criterion A	8
$1.7 < R \leq 2.25$	$R > 3$	Sub-criterion A	1
$2.25 < R \leq 3$	$R > 3$	Sub-criterion A	1

7.3.2 Pairwise comparing the Alternatives with respect to the Criteria

The alternatives are evaluated between each other against the sub-criteria resulting from data collection and, thus, representing the condition of the site. To the comparison against the rest of the sub-criteria an intensity of 1 "equal importance" will be assign. It is mandatory to assign even this last score as all the sub-criteria have their own weight and contribute to the final result, but the proper pairwise is done only for the sub-criteria characterizing the site under assessment. The alternatives are named Clear, Green, Amber, Red following the RAG classification in Section 7.2.

The comparison of the alternatives for the use-case is as follow:

Table 7.11 Pairwise comparing the Alternatives with respect to criterion "Seasonal Deformation"

C1: *Seasonal Deformation*							
Alternatives		C1.1: *Dry*		C1.2: *Medium*		C1.3: *Wet*	
X	Y	More important	Intensity	More important	Intensity	More important	Intensity
CLEAR	GREEN	X	1	Y	3	X	1
CLEAR	AMBER	X	1	X	1	X	1
GREEN	AMBER	X	1	X	3	X	1
CLEAR	RED	X	1	X	1	X	1
GREEN	RED	X	1	X	6	X	1
AMBER	RED	X	1	X	1	X	1

Table 7.12 Pairwise comparing the Alternatives with respect to criterion "Type of Soil"

C2: *Type of Soil*									
Alternatives		C2.1: *S3*		C2.2: *S4*		C2.3: *S6 - X*		C2.4: *S1*	
X	Y	More important	Intensity	More important	Intensity	More important	Intensity	More important	Intensity
CLEAR	GREEN	X	1	X	1	X	1	X	1
CLEAR	AMBER	X	1	Y	6	X	1	X	1
GREEN	AMBER	X	1	Y	3	X	1	X	1
CLEAR	RED	X	1	Y	1	X	1	X	1
GREEN	RED	X	1	Y	1	X	1	X	1
AMBER	RED	X	1	X	3	X	1	X	1

Table 7.13 Pairwise comparing the Alternatives with respect to criterion "Vegetation"

C3: *Vegetation*									
Alternatives		C3.1: *High*		C3.2: *Mix*		C3.3: *Low*		C3.4: *Medium*	
X	Y	More important	Intensity	More important	Intensity	More important	Intensity	More important	Intensity
CLEAR	GREEN	X	1	X	1	Y	3	X	1
CLEAR	AMBER	X	1	X	1	X	1	X	1
GREEN	AMBER	X	1	X	1	X	3	X	1
CLEAR	RED	X	1	X	1	X	1	X	1
GREEN	RED	X	1	X	1	X	6	X	1
AMBER	RED	X	1	X	1	X	1	X	1

Table 7.14 Pairwise comparing the Alternatives with respect to criterion "Rainfall"

		C4: *Rainfall*							
Alternatives		C4.1: $R \leq 1.7$		C4.2: $1.7 < R \leq 2.25$		C4.3: $2.25 < R \leq 3$		C4.4: $R > 3$	
X	Y	More important	Intensity	More important	Intensity	More important	Intensity	More important	Intensity
CLEAR	GREEN	X	1	X	1	X	1	X	1
CLEAR	AMBER	X	1	X	1	X	1	X	1
GREEN	AMBER	X	1	X	1	X	1	X	1
CLEAR	RED	Y	9	X	1	X	1	X	1
GREEN	RED	Y	6	X	1	X	1	X	1
AMBER	RED	Y	3	X	1	X	1	X	1

7.4 Steps of the tool

The tool is implemented as an Excel file with 7 tabs. In the first tab "Objective" (Figure 7.2) the user can input the basic information:

- Name and date
- Objective of the problem
- Number of criteria

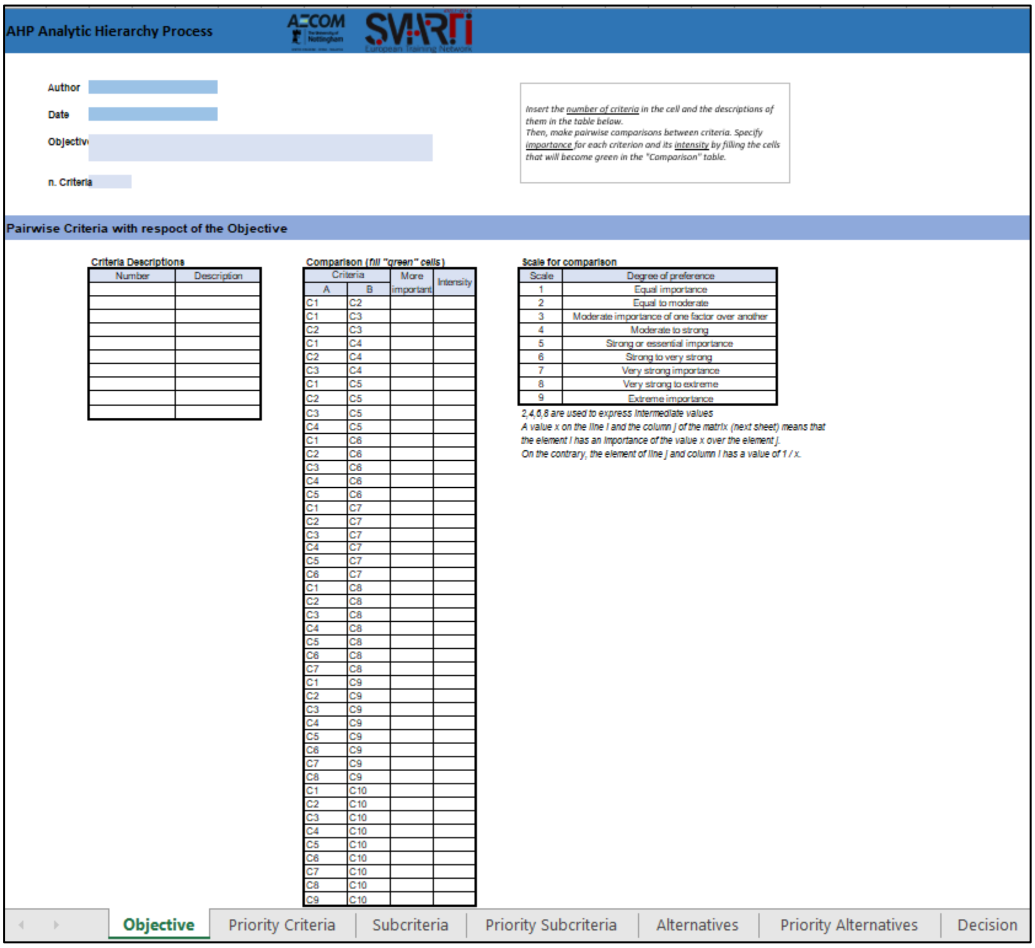


Figure 7.2 Objective tab

The tool and its tables and matrices are built in order to be able to accommodate a maximum number of 10 criteria. When the number of criteria is inserted, the cells that need to be completed in the "Comparison table" become green (Figure 7.3).

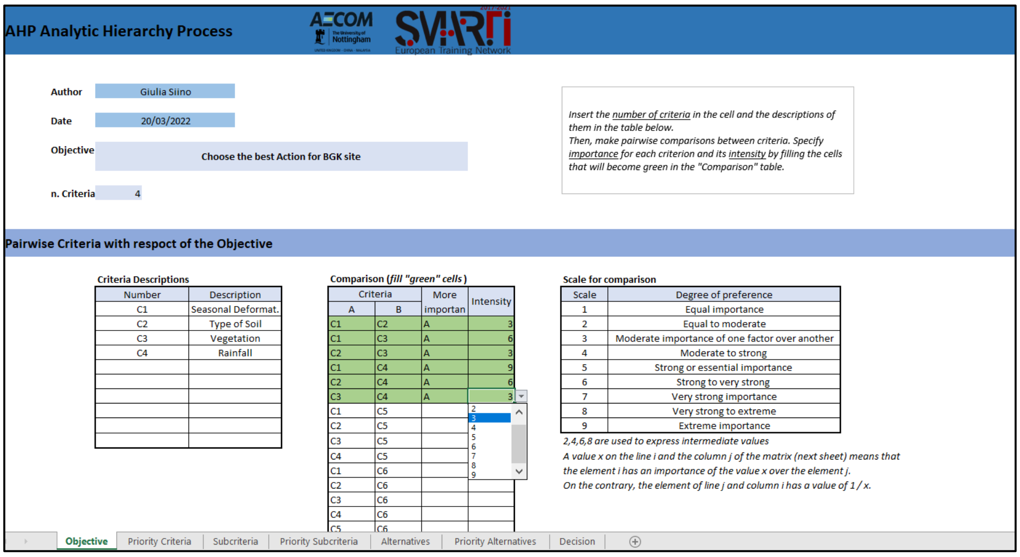


Figure 7.3 Objective tab - info input

From two dropdown lists, both the importance and the intensity can be selected and, in this way, each can be assigned pairwise.

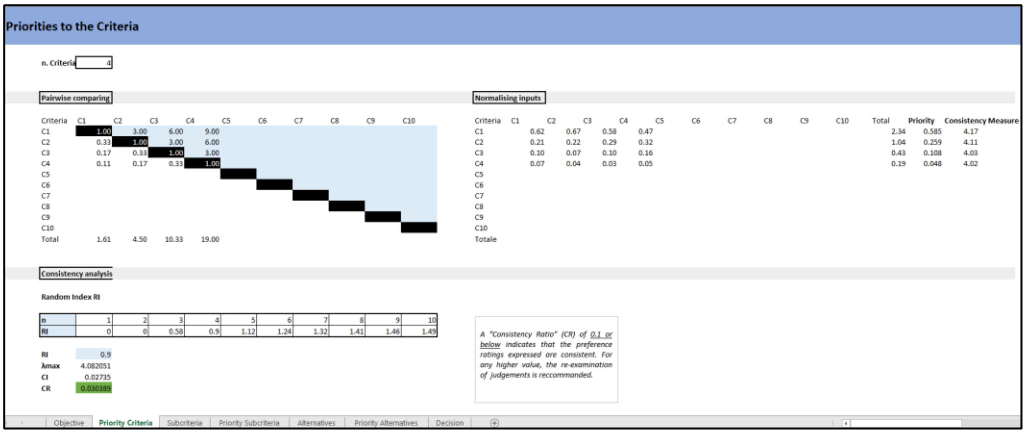


Figure 7.4 Priority Criteria tab

In the "Priority Criteria" tab (Figure 7.4) the consistency analysis is undertaken with the calculation of the "Consistency Ratio" (CR) for which a value of 0.1 or less indicates that the preference ratings expressed in the Objective tab are consistent. When the CR is within the range recommended, the cell becomes green and the analysis can move forward.

The "Sub-criteria" tab (Figure 7.5) allows insertion of the number of sub-criteria for each criterion and pairwise comparison of them with a series of tables in which importance and intensity can be chosen by means of dropdown lists.

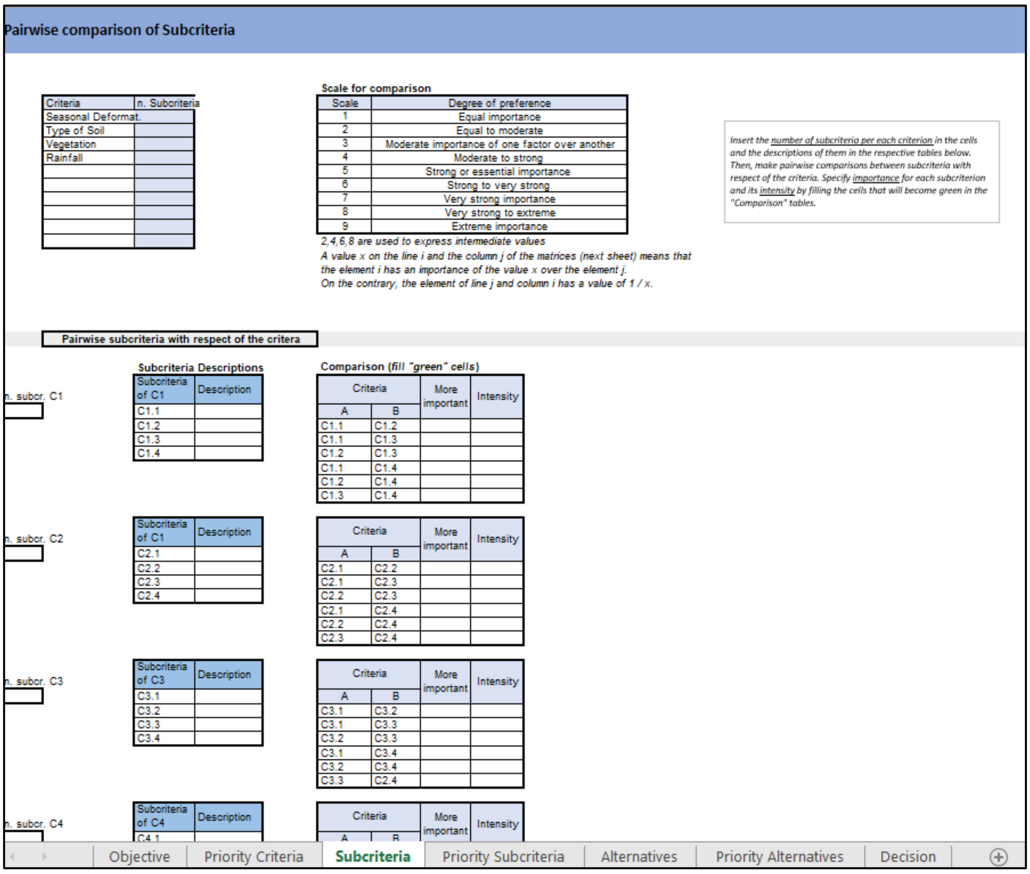


Figure 7.5 Sub-criteria tab

In the same way as for the Comparison table of Criteria, when the number of sub-criteria is input then the corresponding cells that require filling become green. In this case the tool is built to allow a max of 4 sub-criteria per criterion (Figure 7.6).

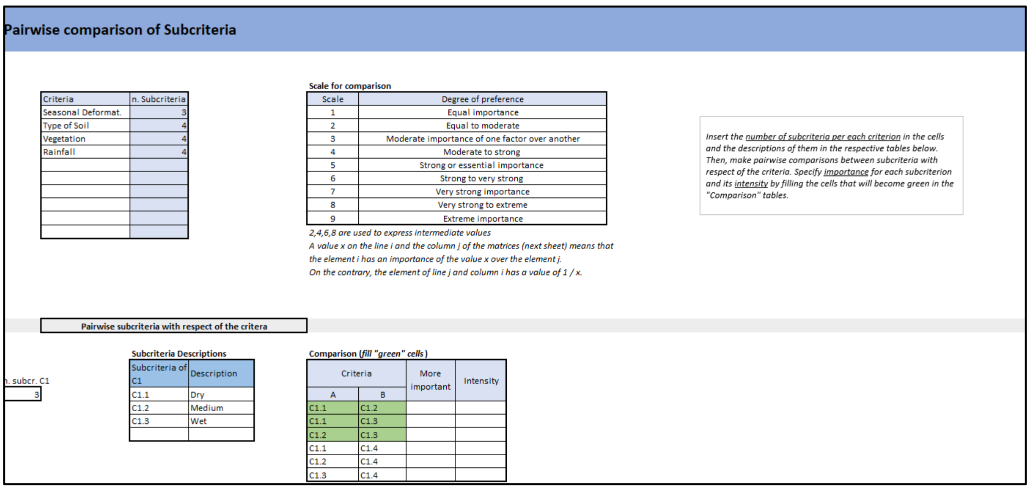


Figure 7.6 Sub-criteria info input

The tables completed after pairwise comparison of Sub-criteria are as follows (Figure 7.7):

Pairwise subcriteria with respect of the critera

n. subcr. C1: 3

Subcriteria Descriptions

Subcriteria of C1	Description
C1.1	Dry
C1.2	Medium
C1.3	Wet

Comparison (*fill "green" cells*)

Criteria A	Criteria B	More important	Intensity
C1.1	C1.2	B	6
C1.1	C1.3	A	1
C1.2	C1.3	A	3
C1.1	C1.4		
C1.2	C1.4		
C1.3	C1.4		

n. subcr. C2: 4

Subcriteria of C1	Description
C2.1	S3
C2.2	S4
C2.3	S6 - X
C2.4	S1

Criteria A	Criteria B	More important	Intensity
C2.1	C2.2	B	3
C2.1	C2.3	A	1
C2.2	C2.3	A	3
C2.1	C2.4	A	1
C2.2	C2.4	A	6
C2.3	C2.4	A	1

n. subcr. C3: 4

Subcriteria of C3	Description
C3.1	High
C3.2	Mix
C3.3	Low
C3.4	Medium

Criteria A	Criteria B	More important	Intensity
C3.1	C3.2	B	1
C3.1	C3.3	B	6
C3.2	C3.3	B	3
C3.1	C3.4	A	1
C3.2	C3.4	A	1
C3.3	C2.4	A	3

n. subcr. C4: 4

Subcriteria of C4	Description
C4.1	R ≤ 1.7
C4.2	1.7 < R ≤ 2.25
C4.3	2.25 < R ≤ 3
C4.4	R > 3

Criteria A	Criteria B	More important	Intensity
C4.1	C4.2	A	6
C4.1	C4.3	A	7
C4.2	C4.3	A	1
C4.1	C4.4	A	8
C4.2	C4.4	A	1
C4.3	C4.4	A	1

Figure 7.7 Sub-criteria comparison completed

The calculation of the local and global priorities for each sub-criteria is undertaken in the "Priority Sub-criteria" tab (Figure 7.8), where a set of matrices allows pairwise comparison of the input.

Priorities to the Subcriteria

Pairwise comparing

C1.n	C1.1	C1.2	C1.3	C1.4
C1.1	1.00	0.17	1.00	
C1.2	6.00	1.00	3.00	
C1.3	1.00	0.33	1.00	
C1.4				
Total	8.00	1.50	5.00	

Normalising inputs

C1.n	C1.1	C1.2	C1.3	C1.4	Total	Local priority	Global priority
C1.1	0.13	0.11	0.20		0.44	0.15	0.0851
C1.2	0.75	0.67	0.60		2.02	0.67	0.39353
C1.3	0.13	0.22	0.20		0.55	0.18	0.10679
C1.4							

C2.n	C2.1	C2.2	C2.3	C2.4
C2.1	1.00	0.33	1.00	1.00
C2.2	3.00	1.00	3.00	6.00
C2.3	1.00	0.33	1.00	1.00
C2.4	1.00	0.17	1.00	1.00
Total	6.00	1.83	6.00	9.00

C2.n	C2.1	C2.2	C2.3	C2.4	Total	Local priority	Global priority
C2.1	0.17	0.18	0.17	0.11	0.63	0.16	0.04052
C2.2	0.50	0.55	0.50	0.67	2.21	0.55	0.14313
C2.3	0.17	0.18	0.17	0.11	0.63	0.16	0.04052
C2.4	0.17	0.09	0.17	0.11	0.54	0.13	0.03464

C3.n	C3.1	C3.2	C3.3	C3.4
C3.1	1.00	1.00	0.17	1.00
C3.2	1.00	1.00	0.33	1.00
C3.3	6.00	3.00	1.00	3.00
C3.4	1.00	1.00	0.33	1.00
Total	9.00	6.00	1.83	6.00

C3.n	C3.1	C3.2	C3.3	C3.4	Total	Local priority	Global priority
C3.1	0.11	0.17	0.09	0.17	0.54	0.13	0.01446
C3.2	0.11	0.17	0.18	0.17	0.63	0.16	0.01692
C3.3	0.67	0.50	0.55	0.50	2.21	0.55	0.05975
C3.4	0.11	0.17	0.18	0.17	0.63	0.16	0.01692

C4.n	C4.1	C4.2	C4.3	C4.4
C4.1	1.00	6.00	7.00	8.00
C4.2	0.17	1.00	1.00	1.00
C4.3	0.14	1.00	1.00	1.00
C4.4	0.13	1.00	1.00	1.00
Total	1.43	9.00	10.00	11.00

C4.n	C4.1	C4.2	C4.3	C4.4	Total	Local priority	Global priority
C4.1	0.70	0.67	0.70	0.73	2.79	0.70	0.0333
C4.2	0.12	0.11	0.10	0.09	0.42	0.10	0.00499
C4.3	0.10	0.11	0.10	0.09	0.40	0.10	0.00479
C4.4	0.09	0.11	0.10	0.09	0.39	0.10	0.00464

Objective | Priority Criteria | Subcriteria | **Priority Subcriteria** | Alternatives | Priority Alternatives | Decision

Figure 7.8 Priority Sub-criteria tab

Finally, the last step for the user is to input pairwise comparison of the alternatives with respect to the criteria and Sub-criteria (Figure 7.9).

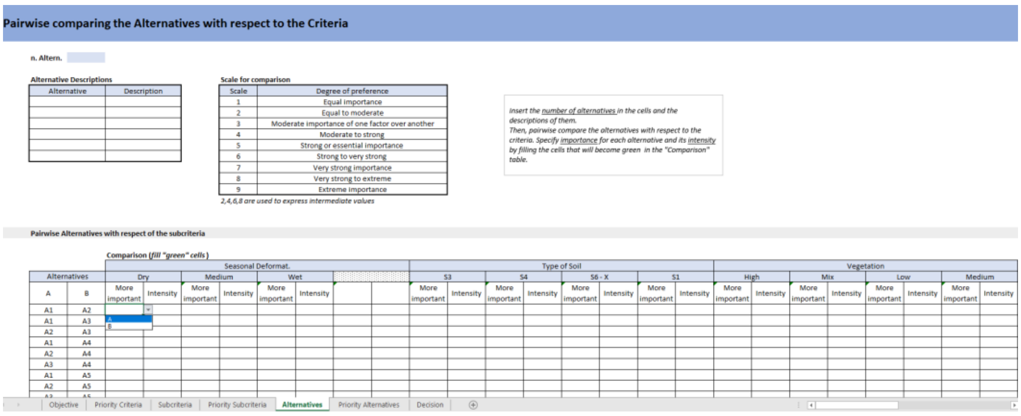


Figure 7.9 Alternatives tab

The table ones again has dropdown lists and the cells in the lower part of the Alternatives tab sheet get coloured green when the number of Alternatives is input (Figure 7.10) in the upper part of this sheet and the alternatives are displayed as soon as the description column is completed.

The table with the completed alternative comparison is displayed in Figure 7.11.

wise comparing the Alternatives with respect to the Criteria

n. Altern. 4

Alternative Descriptions

Alternative	Description
A1	Clear
A2	Green
A3	Amber
A4	Red

Scale for comparison

Scale	Degree of preference
1	Equal importance
2	Equal to moderate
3	Moderate importance of one factor over another
4	Moderate to strong
5	Strong or essential importance
6	Strong to very strong
7	Very strong importance
8	Very strong to extreme
9	Extreme importance

2,4,6,8 are used to express intermediate values

Insert the number of alternatives in the cells and the descriptions of them. Then, pairwise compare the alternatives with respect to the criteria. Specify importance for each alternative and its intensity by filling the cells that will become green in the "Comparison" table.

Figure 7.10 Alternatives info input

Pairwise Alternatives with respect of the subcriteria

Comparison (*fill "green" cells*)

		Seasonal Deformat.								Type of Soil							
Alternatives		Dry		Medium		Wet				S3		S4		S6 - X		S1	
A	B	More importan	Intensity	More importan	Intensity	More importan	Intensity			More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity
Clear	Green	A	1	B	3	A	1			A	1	A	1	A	1	A	1
Clear	Amber	A	1	A	1	A	1			A	1	B	6	A	1	A	1
Green	Amber	A	1	A	3	A	1			A	1	B	3	A	1	A	1
Clear	Red	A	1	A	1	A	1			A	1	B	1	A	1	A	1
Green	Red	A	1	A	6	A	1			A	1	B	1	A	1	A	1
Amber	Red	A	1	A	1	A	1			A	1	A	3	A	1	A	1

Vegetation								Rainfall							
High		Mix		Low		Medium		R ≤ 1.7		1.7 < R ≤ 2.25		2.25 < R ≤ 3		R > 3	
More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity	More importan	Intensity
A	1	A	1	B	3	A	1	A	1	A	1	A	1	A	1
A	1	A	1	A	1	A	1	A	1	A	1	A	1	A	1
A	1	A	1	A	3	A	1	A	1	A	1	A	1	A	1
A	1	A	1	A	1	A	1	B	9	A	1	A	1	A	1
A	1	A	1	A	6	A	1	B	6	A	1	A	1	A	1
A	1	A	1	A	1	A	1	B	3	A	1	A	1	A	1

Figure 7.11 Completed alternatives comparison

In a similar manner as was done for prioritizing the sub-criteria (Figure 7.8) the local and global priority calculation is undertaken in the "Priority Alternatives" tab (Figure 7.12). The matrices are as follows:

Priorities to the Alternatives

n. Altern.	4

Pairwise comparing

C1.1	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C1.2	A1	A2	A3	A4	A5	A6
A1	1.00	0.33	1.00	1.00		
A2	3.00	1.00	3.00	6.00		
A3	1.00	0.33	1.00	1.00		
A4	1.00	0.17	1.00	1.00		
A5						
A6						
Total	6.00	1.83	6.00	9.00		

C1.3	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C2.1	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C2.2	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	0.17	1.00		
A2	1.00	1.00	0.33	1.00		
A3	6.00	3.00	1.00	3.00		
A4	1.00	1.00	0.33	1.00		
A5						
A6						
Total	9.00	6.00	1.83	6.00		

C2.3	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C2.4	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C3.1	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C3.2	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

Normalising inputs

C1.1	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.02128
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.02128
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.02128
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.02128
A5									
A6									

C1.2	A1	A2	A3	A4			Total	Local priority	Global priority
A1	0.17	0.18	0.17	0.11			0.63	0.16	0.06161
A2	0.50	0.55	0.50	0.67			2.21	0.55	0.21764
A3	0.17	0.18	0.17	0.11			0.63	0.16	0.06161
A4	0.17	0.09	0.17	0.11			0.54	0.13	0.05267
A5									
A6									

C1.3	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.0267
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.0267
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.0267
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.0267
A5									
A6									

C2.1	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A5									
A6									

C2.2	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.11	0.17	0.09	0.17			0.54	0.13	0.01916
A2	0.11	0.17	0.18	0.17			0.63	0.16	0.02241
A3	0.67	0.50	0.55	0.50			2.21	0.55	0.07915
A4	0.11	0.17	0.18	0.17			0.63	0.16	0.02241
A5									
A6									

C2.3	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.01013
A5									
A6									

C2.4	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00866
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00866
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00866
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00866
A5									
A6									

C3.1	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00362
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00362
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00362
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00362
A5									
A6									

C3.2	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A5									
A6									

C3.3	A1	A2	A3	A4	A5	A6
A1	1.00	0.33	1.00	1.00		
A2	3.00	1.00	3.00	6.00		
A3	1.00	0.33	1.00	1.00		
A4	1.00	0.17	1.00	1.00		
A5						
A6						
Total	6.00	1.83	6.00	9.00		

C3.3	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.17	0.18	0.17	0.11			0.63	0.16	0.00936
A2	0.50	0.55	0.50	0.67			2.21	0.55	0.03305
A3	0.17	0.18	0.17	0.11			0.63	0.16	0.00936
A4	0.17	0.09	0.17	0.11			0.54	0.13	0.008
A5									
A6									

C3.4	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C3.4	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00423
A5									
A6									

C4.1	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	0.11		
A2	1.00	1.00	1.00	0.17		
A3	1.00	1.00	1.00	0.33		
A4	9.00	6.00	3.00	1.00		
A5						
A6						
Total	12.00	9.00	6.00	1.61		

C4.1	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.08	0.11	0.17	0.07			0.43	0.11	0.00358
A2	0.08	0.11	0.17	0.10			0.46	0.12	0.00387
A3	0.08	0.11	0.17	0.21			0.57	0.14	0.00473
A4	0.75	0.67	0.50	0.62			2.54	0.63	0.02112
A5									
A6									

C4.2	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C4.2	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00125
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00125
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00125
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00125
A5									
A6									

C4.3	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C4.3	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.0012
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.0012
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.0012
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.0012
A5									
A6									

C4.4	A1	A2	A3	A4	A5	A6
A1	1.00	1.00	1.00	1.00		
A2	1.00	1.00	1.00	1.00		
A3	1.00	1.00	1.00	1.00		
A4	1.00	1.00	1.00	1.00		
A5						
A6						
Total	4.00	4.00	4.00	4.00		

C4.4	A1	A2	A3	A4	A5	A6	Total	Local priority	Global priority
A1	0.25	0.25	0.25	0.25			1.00	0.25	0.00116
A2	0.25	0.25	0.25	0.25			1.00	0.25	0.00116
A3	0.25	0.25	0.25	0.25			1.00	0.25	0.00116
A4	0.25	0.25	0.25	0.25			1.00	0.25	0.00116
A5									
A6									

Figure 7.12 Priority Alternatives tab

This tab, like the "Priority Criteria" and the "Priority Subcriteria" tabs, is blocked to the user as the matrices are pre-set and calculation is automatically developed. This minimises human error when the tool is used and also simplifies the job of the user who only has access to the cells that need to be completed (which are also highlighted in green).

The decision is then calculated and shown automatically in the last "Decision" tab where the most suitable alternative is highlighted in green (Figure 7.13).

Making the Decision

Global priority of Alternatives

Alternatives	Seasonal Deformat.				Type of Soil				Vegetation				Rainfall			
	Dry	Medium	Wet		S3	S4	S6 - X	S1	High	Mix	Low	Medium	R ≤ 1.7	1.7 < R ≤ 2.25	2.25 < R ≤ 3	R > 3
Clear	0.021276	0.061614	0.026696		0.010130	0.019156	0.010130	0.008660	0.003615	0.004229	0.009355	0.004229	0.003580	0.001247	0.001198	0.001161
Green	0.021276	0.217636	0.026696		0.010130	0.022409	0.010130	0.008660	0.003615	0.004229	0.033046	0.004229	0.003867	0.001247	0.001198	0.001161
Amber	0.021276	0.061614	0.026696		0.010130	0.079154	0.010130	0.008660	0.003615	0.004229	0.009355	0.004229	0.004729	0.001247	0.001198	0.001161
Red	0.021276	0.052670	0.026696		0.010130	0.022409	0.010130	0.008660	0.003615				0.021123	0.001247		

Alternatives	Global Priority
Clear	0.186277
Green	0.369529
Amber	0.247423
Red	0.177956

The Alternative with the highest global priority (in green) contributes the most to the achievement of the "Objective".

Objective | Priority Criteria | Subcriteria | Priority Subcriteria | Alternatives | Priority Alternatives | Decision

Figure 7.13 Decision tab

7.5 Result of the Use-Case

The simulation delivers the user to a Decision that corresponds to Alternative 2 - Green which is described (Table 7.4) as the alternative adopted when the embankment instability metric EIM is in the "Minor" range (Chapter 4). As stated before, the EIM of this embankment site is known and it is, indeed, belonging to the EIM Minor range ($1 < EIM < 2$) which helps to demonstrate the validity of the model and the tool built.

CHAPTER 8. CONCLUSIONS

8. Conclusions

8.1 Overview of the research

Detecting embankment instability at an early stage is a challenge to any railway operator and geotechnical infrastructure asset manager.

The focus of the work described in this thesis was, firstly, to detect factors leading to embankment instability by performing a deep literature study. After setting common criteria (*Scientific evidence, Availability, Measurability, Data coverage, Updatability*) the factors addressing the criteria were analysed to assess if a link existed between them and the signs of instability detected by track recording vehicles. An Embankment Instability Metric (EIM) was used as a measurement of track geometry deterioration that might be related to embankment instability. The objective of this analysis was to spot correlations between parameters (geotechnical features) and track geometry (EIM) so as to link the causes of instability to any early indication that might be provided by the EIM.

To achieve that, an 8-year backward analysis (2010 - 2018) was performed involving the data collection of four factors addressing the aforementioned criteria and known from literature as playing a role in the embankment instability: type of soil, rainfall, vegetation and 'seasonal deformation' (an indirect indicator of swelling and shrinkage caused by a combination of rainfall and temperature). These parameters were individually assessed against the EIMs (calculated and provided by AECOM) for 28 embankment sites, identified by the author with the help of Network Rail team, as showing instability during the time frame considered for this study. This was done so as to spot whether the four parameters were individually, or in combination, leading to a worsening condition of the track. If so, critical values for each of them was established.

The findings of this analysis laid the basis for the development of a proactive tool for railway maintenance based on a multi-criteria decision-making approach. The most suitable method among the available ones was determined, after an extensive literature review, to be the Analytic Hierarchy Process (AHP) method. The goal of detecting the maintenance action to be undertaken on a specific embankment asset, given information on the type of soil, rainfall, vegetation, and seasonal deformation, is achieved by the use of the tool which thereby supports decision makers during their desktop studies.

8.2 Results

The literature review highlighted how geotechnical features influence, at different levels, the stability or instability of the embankment asset. Some of these features addressed the common criteria set for the screening to be assessed against track geometry (EIM). In this section, the results of the research are presented by addressing the research questions asked in the Introduction.

8.2.1 Does the EIM and the parameters leading to embankment instability show correlations?

The specific analysis reported in this thesis, when treating the parameters one at the time, highlighted a strong correlation between the Seasonal deformation and the EIM, correlation between type of soil and EIM, correlation between vegetation and EIM, but no correlation visible for the rainfall. It is established from the literature that all these parameters do lead to embankment instability; therefore, the author acknowledges that even when the reported analysis doesn't show the influence of a specific parameter over the track deformation, the parameter is proven to be critical for embankment instability from the literature; so its influence shouldn't be overlooked during operations. Where this is the case, it is concluded that either the parameter is related to embankment instability but not to the EIM, or its influence on the EIM cannot be detected in isolation, but only in combination with other parameters.

The correlations resulting from this analysis led to criticalities for each parameter from minor to critical (Table 8.1):

Table 8.1 Criticality assigned to each parameter

Parameter	Minor Criticality	Medium Criticality	High Criticality
Seasonal deformation	Wet season	Medium season	Dry season
Type of soil	Non-cohesive soil	Cohesive soil low shrink-swell potential	Cohesive soil high shrink-swell potential
Vegetation	Low vegetation	Medium vegetation	High vegetation

However, when the parameters are treated in combination and plotted together with the EIM (Figure 5.22) a better understanding of the big picture is obtained. It was observed that embankments made of cohesive soils with high shrink-swell potential, on historically dry sites, with adjacent high vegetation are the ones showing the poorest track condition, otherwise the highest value of EIM.

Therefore, even if rainfall shows no clear influence on stability when analysed independently, when treated in combination with temperature, soil type and vegetation there are strong indications that post-summer cracking of cohesive soils, aided by vegetative evapo-transpiration, can provide high sensitivity to rainfall in autumn leading, in turn, to track quality degradation.

8.2.2 How can these correlations improve the Geotechnical Asset Management?

The final purpose of establishing correlations and criticalities was the development of a tool for proactive maintenance, delivered in Excel format.

The tool developed supports the asset managers with the decision on the most suitable action to undertake for an asset when examined. The Geotechnical Asset Management does therefore benefit from:

- improved use of existing gathered data from various disciplines and monitoring technologies.
- better understanding of the asset characteristics to feed into geotechnical assessments.
- possibility of preliminarily assessing the criticality of the asset without the need to access the site.
- more informed decision-making and prioritisation of intervention using consistent data.

8.3 Implications of the results

The research work addresses some of the challenges faced by current GAM on early embankment failure detection by improving the understanding of the link between cause of instability and the early, visible symptoms of distress. Furthermore, the tool developed supports the decision-making process at a preliminary stage.

The implications of the results are several and various, covering from financial aspects to managerial ones:

- Financial savings: proactive planning reduces the need to have recourse to expensive mitigation measures, therefore the tool developed in this thesis supports cost-effective interventions.
- Manage involuntarily safety risks: the criticality of an asset can be identified at an early stage.
- Enhance data-driven decision: the decision-making tool is evidence-based and developed on logical trade-off of data.
- Support stakeholders in writing, implementing, and managing policies: the critical bands set for the geotechnical features help the implementation of embankments management policy and potential identification of warning scenarios.
- Knowledge of the asset: merging embankment information collected from different monitoring systems, and coming from multiple disciplines, into the same whole picture brings to a comprehensive understanding of the asset.

- Proactive tool: the tool can evaluate the asset risk before the need to go on site arises, hence it improves the asset-management and risk-assessment.

8.4 Limitations & Future work

The findings of this study need to be considered in light of some limitations. First, the work of this thesis uses the Embankment Instability Metric; this is the result of a previous AECOM's project to which the author of this thesis did not give their contribution. Consequently, the limitations of that project automatically become the limitations of this research. Another limitation of the analysis is the availability of data which narrowed down the factors considered for correlations from 15 to only 4.

Lastly, just testing the tool with one site gives a demonstration of the reasonableness of the approach but it does not assess the validity of the tool. At the time of the analysis just seen, AECOM was gathering data for the calculation of EIM values for a list of new assets. The author was provided with one available asset from that list to proceed with the testing of the tool. To validate the tool, it would be important for future research work to test more embankment sites where EIM is available and prove that the tool delivers, or does not deliver, the same intervention suggested by the metric.

This last step was not achieved for the research within this thesis due to the contractual agreements with AECOM coming to their natural end. For the future, obtaining more EIMs could be constrained by the necessary involvement of AECOM being the intellectual owner of the algorithm. In an ideal scenario, where the EIM is opensource and easy to collect for several years for all locations, the investigation of the relationship between symptoms and causes could progress evaluating additional conditions, extending the set of parameters in the tool for a more accurate evaluation.

Another way of validating the tool would be to test and try it with data coming from a different railway culture. From conversation with railway engineers at the SNCF (Société Nationale des Chemins de fer Français) offices in Paris, the author gathered information on how future work could be extended to other industries. There are some similarities between how inspection and track geometry recording is undertaken in the UK and in France. SNCF indeed collect the same track geometry parameters (vertical and horizontal alignment, and twist mainly) to set hazard categories and plan track inspections if a disorder is found. There are also some differences that would primarily be linked to policy, prioritisation, and budget allocation. The advantages of validating the tool using sets of data collected in a different railway environment, and so considering different construction history and maintenance activities, would give an independent indication of whether the idea presented in this thesis has generic applicability.

Finally, considering the thesis finding when treating all parameters in combination, it would be interesting, as future study, to investigate the benefit of vegetation management on more cohesive soils as a track quality defence action.

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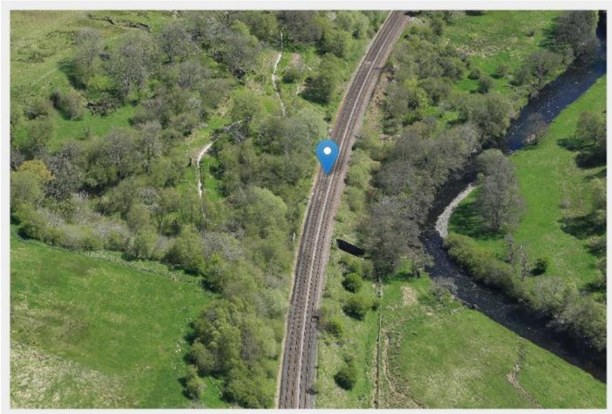
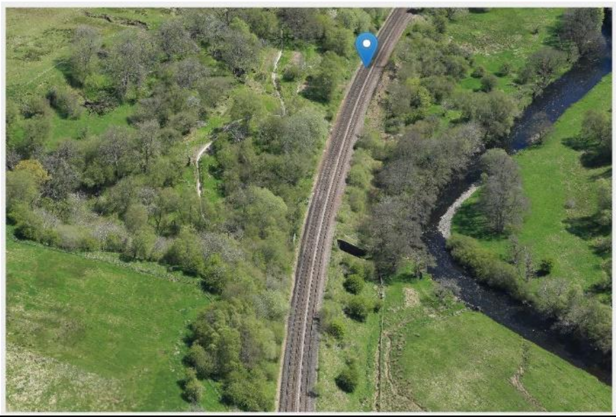
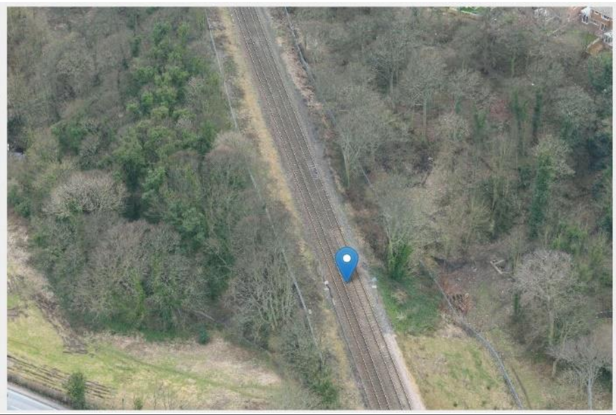
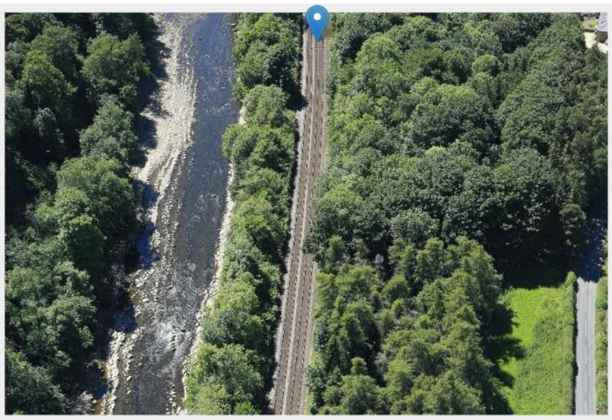
Appendix A

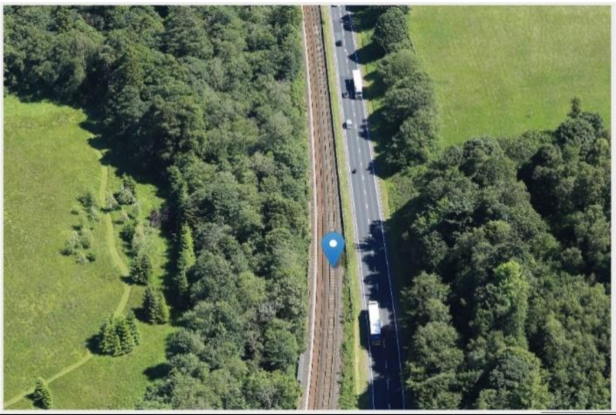
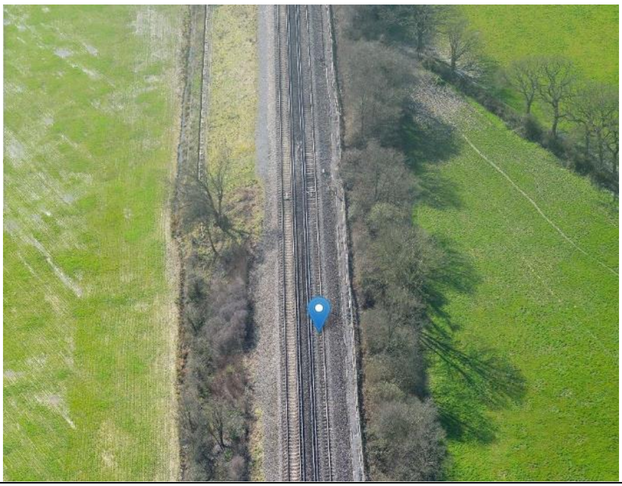
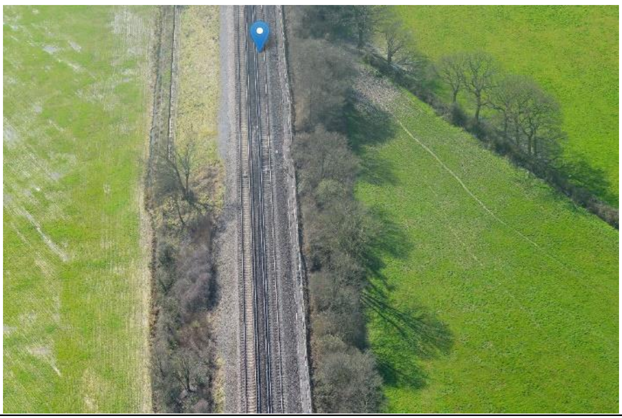
Pictures of vegetation at each site

Site	Vegetation Pictures from Routeview
GWS	
	
GTW1	
	

Site	Vegetation Pictures from Routeview
HNR	
	
LEN3	
	

Site	Vegetation Pictures from Routeview
MLN3	 
NAJ3	 

Site	Vegetation Pictures from Routeview
NEC2	
	
$NEC2_{(2)}$	
	

<table>
<tr><th>Site</th><th>Vegetation Pictures from Routeview</th></tr>
<tr><td rowspan="2">NEC2$_{(3)}$</td><td></td></tr>
<tr><td></td></tr>
<tr><td rowspan="2">NGL</td><td></td></tr>
<tr><td></td></tr>
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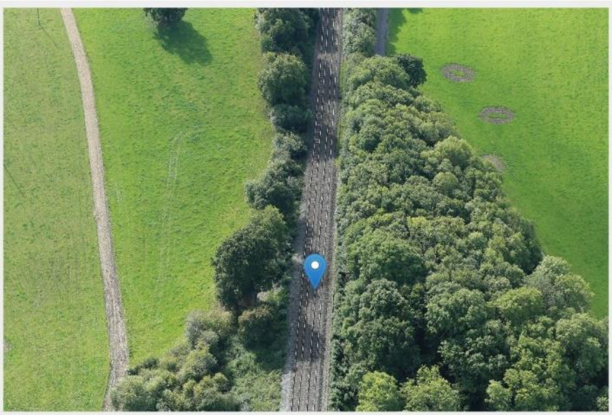
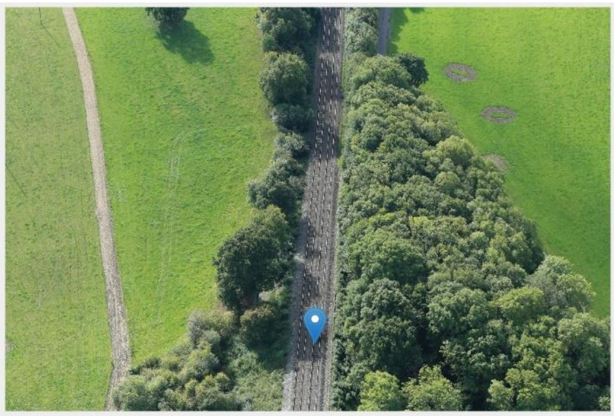
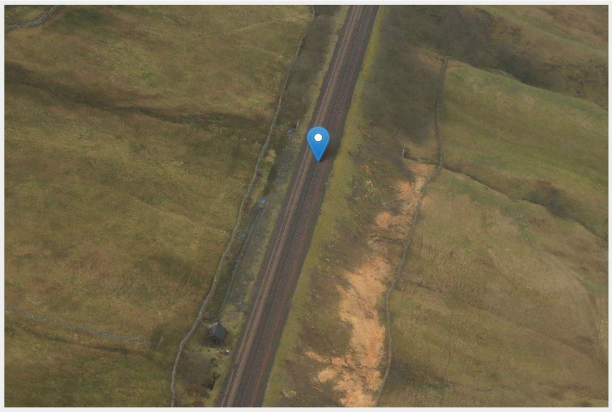
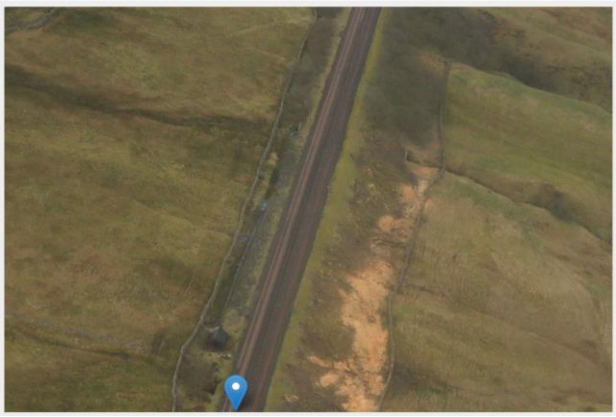
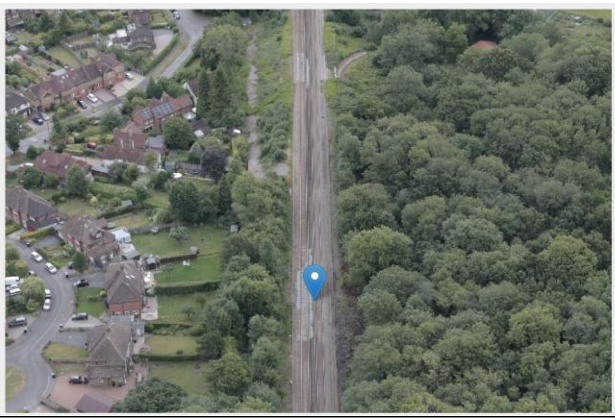
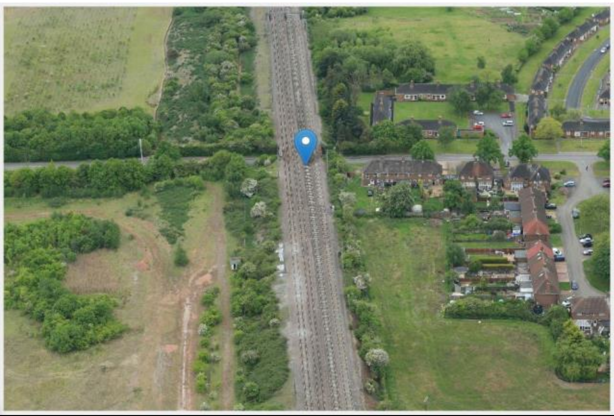
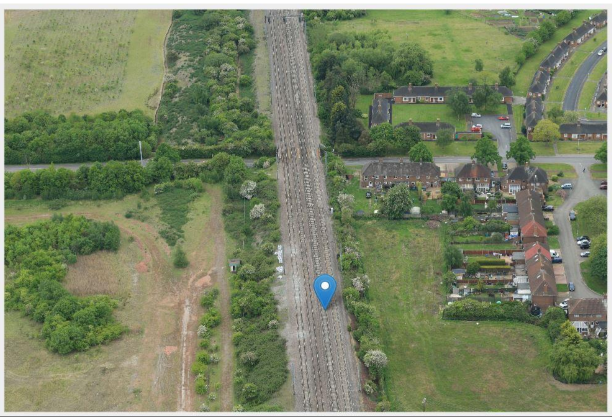
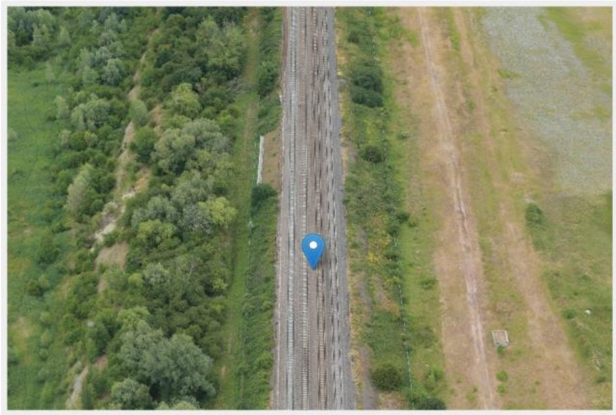
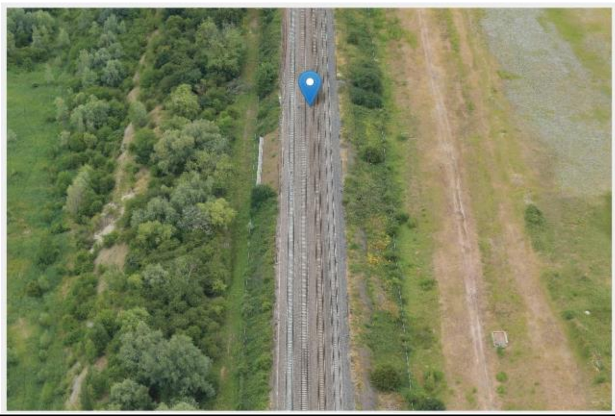
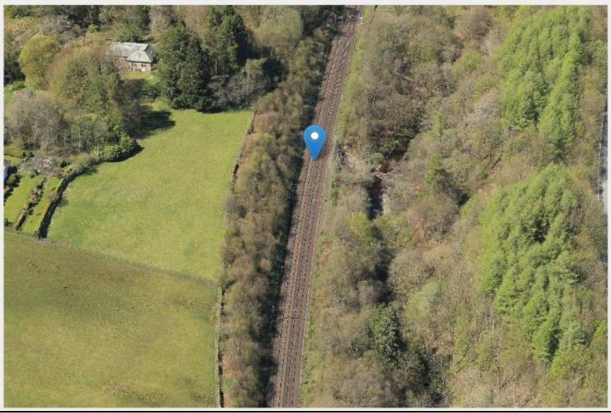
Site	Vegetation Pictures from Routeview
$NGL_{(2)}$	
	
OWW	
	

Site	Vegetation Pictures from Routeview
SAC	
	
SCU1	
	

Site	Vegetation Pictures from Routeview
SPC1	
	
$SPC1_{(2)}$	
	

Site	Vegetation Pictures from Routeview
SCP1$_{(3)}$	
	
TJC3	
	

Site	Vegetation Pictures from Routeview
THH	
	
WCM1	
	

Site	Vegetation Pictures from Routeview
WCM2	 

Appendix B

EIMs per each site and year

Site	Year	EIM
GWS	2011	0.31
	2012	0.43
	2014	0.75
	2015	0.82
	2016	1.01
	2017	0.98
GTW1	2011	2.68
	2012	0.85
	2013	4.43
	2014	
	2015	
	2016	9.51
	2017	9.57
HNR	2011	0.94
	2012	1.47
	2013	0.8
	2014	0.61
	2016	0.95
	2017	2.2
LEN3	2011	3.45
	2012	8.49
	2013	2.89
	2014	5.08
	2015	5.16
	2016	3.75
	2017	0.8
MLN3	2011	0.95
	2012	0.67
	2013	2.07
	2014	0.74
	2015	1.83
	2016	3.64
	2017	9

Site	Year	EIM
NAJ3	2011	7.29
	2012	6.05
	2013	7.17
	2014	3.83
	2015	4.88
	2016	7.68
	2017	1.86
NEC2	2011	0.35
	2012	0.44
	2013	1.22
	2014	1.84
	2015	1.14
	2016	0.96
	2017	0.92
NEC2_2	2011	0.21
	2012	0.07
	2013	0.11
	2014	0.15
	2015	0.9
	2016	0.81
	2017	0.68
NEC2_3	2011	0.18
	2012	6.31
	2013	0.8
	2014	
	2015	0.87
	2016	1.15
	2017	0.43

Site	Year	EIM
NGL	2011	0.72
	2012	1.62
	2013	1.1
	2014	1.42
	2015	9.08
	2016	3.57
	2017	2.74
NGL_2	2011	11.28
	2012	3.21
	2013	7.43
	2014	5.04
	2015	1.42
	2016	0.64
	2017	0.99
OWW	2011	
	2012	
	2013	2.34
	2014	1.83
	2015	7.49
	2016	4.73
	2017	3.12
SAC	2011	0.49
	2012	0.4
	2013	0.55
	2014	0.8
	2015	0.81
	2016	0.59
	2017	0.5

Site	Year	EIM
SCU1	2011	1.28
	2012	1.26
	2013	1.38
	2014	0.45
	2015	2.29
	2016	3.59
	2017	7.73
SPC1	2011	1.03
	2012	0.53
	2013	1.32
	2014	2.39
	2015	1.11
	2016	0.34
	2017	0.11
SPC1_2	2011	0.73
	2012	0.6
	2013	0.91
	2014	1.14
	2015	0.45
	2016	0.72
	2017	0.92
SCP1_3	2011	0.67
	2012	2.28
	2013	2.18
	2014	3.12
	2015	4.33
	2016	1.43
	2017	0.96

Site	Year	EIM
TJC3	2011	0.61
	2012	0.49
	2013	0.67
	2014	2.46
	2015	0.83
	2016	1.29
	2017	1.07
THH	2011	2.7
	2012	0.58
	2013	0.39
	2014	0.65
	2015	
	2016	
	2017	
WCM1	2011	0.72
	2012	1.12
	2013	0.82
	2014	0.69
	2015	1.24
	2016	0.27
	2017	0.55
WCM2	2011	1.58
	2012	1.56
	2013	1.3
	2014	3.64
	2015	1.36
	2016	0.69
	2017	0.36

Appendix C

72-hour rainfall calculation

		GSW				GTW1				HNR				LEN3				MLN3			
		Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM
2010.00	winter																				
2010.00	summer	8.975147	43.64	4.86		4.92	51.00	10.36		5.49	44.07	8.03		5.38	35.38	6.57		9.62	73.70	7.66	
2011.00	winter	10.29	52.28	5.08	0.31	4.17	32.74	7.86	2.68	2.78	20.40	7.34	0.94	5.90	35.78	6.07	3.45	8.72	38.22	4.38	0.95
2011.00	summer	13.95	47.82	3.43		5.06	32.47	6.41		3.66	22.26	6.08		4.78	28.34	5.93		11.73	56.18	4.79	
2012.00	winter	16.75	89.35	5.33	0.43	5.49	41.95	7.64	0.85	5.55	36.78	6.63	1.47	3.43	27.42	7.99	8.49	13.48	56.50	4.19	0.67
2012.00	summer	12.39	53.66	4.33		7.27	46.68	6.42		8.61	49.10	5.70		9.24	87.80	9.50		9.80	41.72	4.26	
2013.00	winter	13.02	76.11	5.84	0.27	7.93	34.77	4.39	4.43	8.32	48.32	5.81	0.80	8.13	75.36	9.27	2.89	10.27	36.27	3.53	2.07
2013.00	summer	10.01	47.20	4.72		4.75	37.87	7.97		5.63	43.93	7.81		6.28	65.46	10.42		9.28	54.07	5.82	
2014.00	winter	18.23	70.58	3.87	0.75	10.00	53.50	5.35		6.36	31.89	5.01	0.61	4.93	30.84	6.26	5.08	13.38	45.78	3.42	0.74
2014.00	summer	9.06	72.57	8.01		6.16	39.31	6.38		5.88	38.62	6.57		6.00	43.54	7.26		8.30	29.83	3.59	
2015.00	winter	16.64	68.16	4.10	0.82	6.20	30.32	4.89		4.90	19.87	4.06	0.46	3.78	24.49	6.49	5.16	11.91	41.69	3.50	1.83
2015.00	summer	8.72	36.78	4.22		5.64	40.00	7.09		5.32	34.35	6.46		6.39	58.13	9.10		8.92	39.95	4.48	
2016.00	winter	21.82	86.79	3.98	1.01	6.47	33.62	5.19	9.51	6.32	34.66	5.48	0.95	8.04	47.00	5.85	3.75	15.04	48.74	3.24	3.64
2016.00	summer	8.38	43.61	5.20		4.97	39.86	8.02		4.27	20.83	4.88		4.43	33.59	7.58		7.29	31.81	4.37	
2017.00	winter	9.71	51.20	5.27	0.98	4.67	47.10	10.09	9.57	4.36	37.95	8.70	2.20	5.14	34.54	6.72	0.80	9.51	49.24	5.18	9.00
2017.00	summer	11.60	49.46			5.75	40.16			5.38	38.76			5.73	46.70			10.07	37.11		

		NAJ3				NAJ3 (2)				NEC2				NEC2 (2)				NEC2 (3)			
		Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM
2010.00	winter																				
2010.00	summer	4.71	42.97	9.11		4.71	42.97	9.11		6.56	27.81	4.24		5.35	34.57	6.46		5.35	34.57	6.46	
2011.00	winter	3.17	17.23	5.43	7.29	3.17	17.23	5.43	7.29	8.66	73.75	8.51	0.35	7.08	50.44	7.13	0.21	7.08	50.44	7.13	0.21
2011.00	summer	3.86	20.15	5.21		3.86	20.15	5.21		11.06	66.58	6.02		7.29	48.79	6.70		7.29	48.79	6.70	
2012.00	winter	4.47	30.50	6.82	6.05	4.47	30.50	6.82	6.05	9.40	34.79	3.70	0.44	6.21	27.65	4.46	0.07	6.21	27.65	4.46	0.07
2012.00	summer	8.25	36.99	4.48		8.25	36.99	4.48		12.89	59.73	4.64		9.59	49.26	5.14		9.59	49.26	5.14	
2013.00	winter	7.24	33.30	4.60	7.17	7.24	33.30	4.60	7.17	9.17	40.62	4.43	1.22	6.41	33.59	5.24	0.11	6.41	33.59	5.24	0.11
2013.00	summer	4.77	28.51	5.98		4.77	28.51	5.98		8.71	63.80	7.32		6.70	43.86	6.54		6.70	43.86	6.54	
2014.00	winter	7.23	39.14	5.41	3.83	7.23	39.14	5.41	3.83	11.13	48.88	4.39	1.84	7.32	29.21	3.99	0.15	7.32	29.21	3.99	0.15
2014.00	summer	5.58	35.38	6.34		5.58	35.38	6.34		8.36	44.13	5.28		5.86	32.30	5.52		5.86	32.30	5.52	
2015.00	winter	4.71	16.80	3.56	4.88	4.71	16.80	3.56	4.88	10.85	67.12	6.19	1.14	7.06	37.30	5.28	0.90	7.06	37.30	5.28	0.90
2015.00	summer	4.28	36.18	8.46		4.28	36.18	8.46		7.10	40.64	5.73		5.75	39.16	6.82		5.75	39.16	6.82	
2016.00	winter	5.76	29.34	5.09	7.68	5.76	29.34	5.09	7.68	18.64	131.99	7.08	0.96	13.76	94.79	6.89	0.81	13.76	94.79	6.89	0.81
2016.00	summer	4.23	26.84	6.35		4.23	26.84	6.35		7.76	30.87	3.98		5.87	27.70	4.72		5.87	27.70	4.72	
2017.00	winter	4.20	29.06	6.92	1.86	4.20	29.06	6.92	1.86	6.63	37.44	5.65	0.92	5.20	35.40	6.81	0.68	5.20	35.40	6.81	0.68
2017.00	summer	4.91	32.55			4.91	32.55			9.82	46.50			6.67	40.61			6.67	40.61		

		NGL				NGL (2)				OWW				OWW (2)				SAC			
		Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM
2010.00	winter																				
2010.00	summer	4.83	58.51	12.11		4.80	53.10	11.07		5.81	47.97	8.26		5.81	47.97	8.26		11.19	83.82	7.49	
2011.00	winter	3.97	32.14	8.10	0.72	3.91	33.14	8.48	11.28	3.36	29.80	8.87		3.36	29.80	8.87		15.98	123.79	7.75	0.49
2011.00	summer	5.10	35.39	6.94		4.96	33.95	6.85		3.66	27.32	7.47		3.66	27.32	7.47		17.22	116.10	6.74	
2012.00	winter	5.10	37.59	7.37	1.62	4.95	38.11	7.69	3.21	4.97	40.37	8.13		4.97	40.37	8.13		17.61	110.83	6.29	0.4
2012.00	summer	7.05	52.26	7.42		7.00	52.71	7.53		8.60	48.00	5.58		8.60	48.00	5.58		18.06	100.04	5.54	
2013.00	winter	7.44	30.61	4.11	1.10	7.65	30.35	3.97	7.43	8.39	47.42	5.65	2.34	8.39	47.42	5.65	2.34	16.65	88.88	5.34	0.55
2013.00	summer	4.96	36.91	7.44		4.92	35.88	7.29		5.65	38.03	6.73		5.65	38.03	6.73		12.47	70.12	5.62	
2014.00	winter	10.34	61.84	5.98	1.42	9.94	61.35	6.17	5.04	8.02	40.64	5.07	1.83	8.02	40.64	5.07	1.83	20.94	72.98	3.49	0.8
2014.00	summer	7.26	54.64	7.53		7.21	55.84	7.75		5.99	33.34	5.57		5.99	33.34	5.57		10.23	50.17	4.91	
2015.00	winter	6.40	34.20	5.35	9.08	6.09	32.25	5.30	1.42	5.20	27.48	5.29	7.49	5.20	27.48	5.29	7.49	21.46	119.61	5.57	0.81
2015.00	summer	5.61	43.82	7.81		5.14	39.60	7.70		4.95	26.32	5.32		4.95	26.32	5.32		10.33	48.74	4.72	
2016.00	winter	6.46	35.42	5.48	3.57	5.66	32.22	5.70	0.64	7.46	46.57	6.24	4.73	7.46	46.57	6.24	4.73	35.86	216.65	6.04	0.59
2016.00	summer	5.51	44.69	8.11		5.21	43.71	8.39		4.27	35.88	8.41		4.27	35.88	8.41		11.41	82.45	7.23	
2017.00	winter	4.74	55.16	11.64	2.74	4.61	56.61	12.28	0.99	4.95	50.40	10.19	3.12	4.95	50.40	10.19	3.12	14.46	98.48	6.81	0.5
2017.00	summer	5.98	38.30			5.83	38.28			5.41	29.30			5.41	29.30			17.79	89.12		

		SCU1				SCP1				SCP1 (2)				SCP1 (3)				TJC3			
		Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM
2010.00	winter																				
2010.00	summer	5.78	39.29	6.80		5.09	52.06	10.23		5.17	54.94	10.63		5.06	52.20	10.31		4.05	36.00	8.88	
2011.00	winter	5.51	36.00	6.53	1.28	2.71	15.93	5.87	1.03	3.24	18.25	5.64	0.73	2.71	15.99	5.89	0.67	3.94	28.32	7.19	0.61
2011.00	summer	4.84	36.60	7.57		3.74	24.01	6.42		3.97	23.09	5.82		3.71	23.64	6.38		3.41	27.72	8.12	
2012.00	winter	6.68	39.75	5.95	1.26	4.03	40.98	10.18	0.53	4.51	48.20	10.68	0.60	4.02	40.86	10.17	2.28	5.63	47.21	8.38	0.49
2012.00	summer	9.70	55.85	5.76		7.19	38.17	5.31		7.59	43.20	5.69		7.16	38.07	5.31		7.31	46.33	6.34	
2013.00	winter	9.19	35.89	3.90	1.38	6.33	27.30	4.31	1.32	6.86	29.50	4.30	0.91	6.32	26.70	4.23	2.18	6.78	50.76	7.49	0.67
2013.00	summer	6.85	59.77	8.72		4.41	27.10	6.15		4.90	27.23	5.56		4.40	26.96	6.13		5.25	31.79	6.05	
2014.00	winter	12.59	78.94	6.27	0.45	5.43	30.48	5.62	2.39	6.30	35.43	5.62	1.14	5.41	30.74	5.68	3.12	5.41	20.33	3.76	2.46
2014.00	summer	6.60	56.33	8.53		5.40	37.70	6.99		5.74	39.76	6.92		5.39	37.74	7.00		5.84	44.15	7.56	
2015.00	winter	7.54	39.56	5.24	2.29	4.23	15.72	3.71	1.11	4.75	17.50	3.68	0.45	4.23	15.90	3.76	4.33	5.22	24.96	4.78	0.83
2015.00	summer	5.45	46.83	8.59		4.54	38.25	8.43		5.12	43.74	8.54		4.53	38.61	8.52		5.52	34.77	6.30	
2016.00	winter	8.17	37.88	4.63	3.59	4.96	26.17	5.27	0.34	5.58	23.03	4.13	0.72	4.94	26.40	5.35	1.43	6.44	29.08	4.52	1.29
2016.00	summer	5.14	36.02	7.00		4.27	27.47	6.43		4.82	33.83	7.02		4.22	26.32	6.23		4.95	33.59	6.79	
2017.00	winter	5.15	55.76	10.84	7.73	3.55	26.60	7.49	0.11	3.94	26.40	6.70	0.92	3.54	26.60	7.50	0.96	4.33	47.78	11.02	1.07
2017.00	summer	6.81	53.38			5.10	39.00			5.58	43.06			5.11	39.10			5.41	43.49		

		THH				WCM1				WCM2			
		Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM	Avrg	Rmax	Ratio	EIM
2010.00	winter												
2010.00	summer	5.43	52.64	9.70		9.13	45.58	4.99		7.23	42.01	5.81	
2011.00	winter	6.38	43.13	6.76	2.7	11.97	65.76	5.49	0.72	8.41	56.63	6.74	1.58
2011.00	summer	4.66	31.40	6.74		14.75	64.51	4.37		10.52	61.61	5.86	
2012.00	winter	6.88	42.80	6.22	0.58	15.97	85.84	5.37	1.12	10.94	58.10	5.31	1.56
2012.00	summer	8.70	61.91	7.11		15.35	73.73	4.80		10.40	47.40	4.56	
2013.00	winter	9.80	42.48	4.33	0.39	15.32	85.46	5.58	0.82	9.31	45.99	4.94	1.30
2013.00	summer	5.98	53.76	8.99		10.99	73.28	6.67		7.11	43.50	6.11	
2014.00	winter	12.81	74.74	5.83	0.65	20.59	70.45	3.42	0.69	11.35	55.03	4.85	3.64
2014.00	summer	5.99	66.91			11.69	97.76	8.36		5.94	35.04	5.90	
2015.00	winter	8.29	51.27			18.12	75.90	4.19	1.24	9.71	54.66	5.63	1.36
2015.00	summer	6.71	45.00			10.65	46.63	4.38		7.09	48.49	6.84	
2016.00	winter	9.33	53.87			25.76	113.95	4.42	0.27	15.87	61.22	3.86	0.69
2016.00	summer	4.39	33.67			10.54	59.36	5.63		6.12	27.37	4.47	
2017.00	winter	5.69	63.20			10.26	55.28	5.39	0.55	6.06	37.63	6.21	0.36
2017.00	summer	6.46	38.65			12.92	54.62			7.98	33.94		

Appendix D

Seasonal deformation calculation

		GSW		GTW1		HNR	
		Avrg	EIM	Avrg	EIM	Avrg	EIM
2010	warm season	Medium		Medium		Medium	
	cold season	Medium		Dry		Dry	
2011	warm season	Wet	0.31	Medium	2.68	Medium	0.94
	cold season	Medium		Dry		Dry	
2012	warm season	Wet	0.43	Medium	0.85	Medium	1.47
	cold season	Medium		Dry		Medium	
2013	warm season	Wet	0.27	Medium	4.43	Medium	0.80
	cold season	Medium		Dry		Dry	
2014	warm season	Wet	0.75	Wet		Medium	0.61
	cold season	Medium		Dry		Dry	
2015	warm season	Wet	0.82	Medium		Medium	0.46
	cold season	Medium		Dry		Dry	
2016	warm season	Wet	1.01	Medium	9.51	Medium	0.95
	cold season	Medium		Dry		Dry	
2017	warm season	Wet	0.98	Medium	9.57	Medium	2.20
	cold season	Medium		Dry		Dry	

		NAJ3		NAJ3 (2)		NEC2	
		Avrg	EIM	Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Dry		Medium	
	cold season	Medium		Medium		Medium	
2011	warm season	Dry	7.29	Dry	7.29	Medium	0.35
	cold season	Medium		Medium		Medium	
2012	warm season	Medium	6.05	Medium	6.05	Wet	0.44
	cold season	Medium		Medium		Medium	
2013	warm season	Dry	7.17	Dry	7.17	Medium	1.22
	cold season	Medium		Medium		Medium	
2014	warm season	Dry	3.83	Dry	3.83	Medium	1.84
	cold season	Medium		Medium		Medium	
2015	warm season	Dry	4.88	Dry	4.88	Medium	1.14
	cold season	Medium		Medium		Wet	
2016	warm season	Dry	7.68	Dry	7.68	Dry	0.96
	cold season	Medium		Medium		Medium	
2017	warm season	Dry	1.86	Dry	1.86	Medium	0.92
	cold season	Medium		Medium		Medium	

		NGL		NGL (2)		OWW	
		Avrg	EIM	Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Dry		Dry	
	cold season	Medium		Medium		Medium	
2011	warm season	Dry	0.72	Dry	11.28	Dry	
	cold season	Medium		Medium		Medium	
2012	warm season	Dry	1.62	Dry	3.21	Medium	
	cold season	Medium		Medium		Medium	
2013	warm season	Dry	1.10	Dry	7.43	Dry	2.34
	cold season	Wet		Wet		Wet	
2014	warm season	Dry	1.42	Dry	5.04	Dry	1.83
	cold season	Medium		Medium		Medium	
2015	warm season	Dry	9.08	Dry	1.42	Dry	7.49
	cold season	Medium		Medium		Wet	
2016	warm season	Dry	3.57	Dry	0.64	Dry	4.73
	cold season	Medium		Medium		Medium	
2017	warm season	Dry	2.74	Dry	0.99	Dry	3.12
	cold season	Medium		Medium		Medium	

		SCU1		SPC1		SPC1 (2)	
		Avrg	EIM	Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Dry		Dry	
	cold season	Medium		Medium		Medium	
2011	warm season	Dry	1.28	Dry	1.03	Dry	0.73
	cold season	Medium		Medium		Medium	
2012	warm season	Medium	1.26	Wet	0.53	Dry	0.60
	cold season	Wet		Medium		Medium	
2013	warm season	Dry	1.38	Dry	1.32	Dry	0.91
	cold season	Wet		Medium		Medium	
2014	warm season	Dry	0.45	Dry	2.39	Dry	1.14
	cold season	Medium		Medium		Medium	
2015	warm season	Dry	2.29	Dry	1.11	Dry	0.45
	cold season	Wet		Medium		Medium	
2016	warm season	Dry	3.59	Dry	0.34	Dry	0.72
	cold season	Medium		Medium		Medium	
2017	warm season	Dry	7.73	Dry	0.11	Dry	0.92
	cold season	Medium		Medium		Medium	

		THH		WCM1		WCM2	
		Avrg	EIM	Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Medium		Dry	
	cold season	Medium		Wet		Wet	
2011	warm season	Dry	2.7	Medium	0.72	Medium	1.58
	cold season	Medium		Wet		Wet	
2012	warm season	Medium	0.58	Medium	1.12	Medium	1.56
	cold season	Wet		Wet		Wet	
2013	warm season	Dry	0.39	Medium	0.82	Dry	1.30
	cold season	Wet		Wet		Wet	
2014	warm season	Dry	0.65	Medium	0.69	Dry	3.64
	cold season	Wet		Wet		Wet	
2015	warm season	Dry		Medium	1.24	Dry	1.36
	cold season	Wet		Wet		Wet	
2016	warm season	Dry		Medium	0.27	Dry	0.69
	cold season	Medium		Wet		Medium	
2017	warm season	Dry		Medium	0.55	Medium	0.36
	cold season	Medium		Medium		Medium	

		LEN3		MLN3	
		Avrg	EIM	Avrg	EIM
2010	warm season	Medium		Wet	
	cold season	Dry		Medium	
2011	warm season	Medium	3.45	Wet	0.95
	cold season	Dry		Medium	
2012	warm season	Medium	8.49	Wet	0.67
	cold season	Medium		Medium	
2013	warm season	Medium	2.89	Wet	2.07
	cold season	Dry		Medium	
2014	warm season	Medium	5.08	Wet	0.74
	cold season	Dry		Medium	
2015	warm season	Medium	5.16	Wet	1.83
	cold season	Dry		Medium	
2016	warm season	Wet	3.75	Wet	3.64
	cold season	Dry		Dry	
2017	warm season	Medium	0.80	Wet	9.00
	cold season	Dry		Medium	

		NEC2 (2)		NEC2 (3)	
		Avrg	EIM	Avrg	EIM
2010	warm season	Medium		Dry	
	cold season	Medium		Wet	
2011	warm season	Medium	0.21	Medium	0.21
	cold season	Medium		Wet	
2012	warm season	Wet	0.07	Medium	0.07
	cold season	Medium		Wet	
2013	warm season	Medium	0.11	Medium	0.11
	cold season	Medium		Wet	
2014	warm season	Medium	0.15	Medium	0.15
	cold season	Medium		Wet	
2015	warm season	Medium	0.90	Dry	0.90
	cold season	Wet		Wet	
2016	warm season	Dry	0.81	Medium	0.81
	cold season	Medium		Medium	
2017	warm season	Medium	0.68	Medium	0.68
	cold season	Medium		Medium	

		OWW (2)		SAC	
		Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Medium	
	cold season	Medium		Wet	
2011	warm season	Dry		Medium	0.49
	cold season	Medium		Wet	
2012	warm season	Medium		Medium	0.4
	cold season	Medium		Wet	
2013	warm season	Dry	2.34	Medium	0.55
	cold season	Wet		Wet	
2014	warm season	Dry	1.83	Medium	0.8
	cold season	Medium		Wet	
2015	warm season	Dry	7.49	Medium	0.81
	cold season	Wet		Wet	
2016	warm season	Dry	4.73	Medium	0.59
	cold season	Medium		Wet	
2017	warm season	Dry	3.12	Medium	0.5
	cold season	Medium		Medium	

		SCU1 (3)		TJC3	
		Avrg	EIM	Avrg	EIM
2010	warm season	Dry		Dry	
	cold season	Medium		Medium	
2011	warm season	Dry	0.67	Dry	0.61
	cold season	Medium		Medium	
2012	warm season	Dry	2.28	Dry	0.49
	cold season	Medium		Medium	
2013	warm season	Dry	2.18	Dry	0.67
	cold season	Medium		Medium	
2014	warm season	Dry	3.12	Dry	2.46
	cold season	Medium		Medium	
2015	warm season	Dry	4.33	Dry	0.83
	cold season	Medium		Medium	
2016	warm season	Dry	1.43	Dry	1.29
	cold season	Medium		Medium	
2017	warm season	Dry	0.96	Dry	1.07
	cold season	Medium		Medium	