Ground Improvement for Khulna Soft Clay Soil

Final Report

Mott MacDonald

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Abstract

There are major concerns for the resilience of rural road embankments exposed to an aggressive coastal environment, in areas of high flood risk and where embankments are often constructed on soft soil deposits with high compressible organic content. This study collates the relevant findings from existing research, field observations and ground investigation to understand the effectiveness and limitations of existing ground improvement techniques implemented in Khulna region, and to develop appropriate recommendations to overcome the typical construction challenges for road embankments and structures in Khulna region. Using the results gained observational ground models have been developed for to help better understand the deformation mechanisms and assess the likely contributory causes. Ground improvement techniques are presented that are considered either (a) technically feasible and (b) within the likely budget for rural road construction together with guidance for implementation to deal with specific construction issues. Topics for further research are presented that will improve the ability to apply ground improvement techniques in Khulna Region.

Key words

Bangladesh, Khulna, Rural roads, Soft Clays, Organic Soil, Earthworks, Settlement, embankment failure, Ground Improvement, Infrastructure research, Transport services research

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See www.research4cap.org

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Acronyms, Units and Currencies

$	United States Dollar (US$ 1.00 ≈ provide conversion to local currencies)
ADB	Asian Development Bank
AFCAP	Africa Community Access Partnership
ASCAP	Asia Community Access Partnership
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing Materials
CBR	California Bearing Ratio
CC	Cement column
ISG	Improved subgrade layer
KUET	Khulna University of Engineering & Technology
LGED	Local Government Engineering Department
LL	Liquid Limit
LTP	Load transfer platform
PI	Plasticity Index
PJVD	Prefabricated Jute vertical drain
PL	Plastic Limit
PVD	Prefabricated vertical drain
SCP	Sand compaction pile
SD	Sand drain
SPT	Standard Penetration Test
USCS	Unified Soil Classification System

Glossary of terms

Approach embankment	A bank constructed from earth materials raised above the surrounding land to raise the highway alignment level with, for example, a river crossing, such as a bridge.
Bearing capacity	Relates to the strength of the soil and ability to sustain an applied load before failure (ultimate limit state) or unsatisfactory deformation (serviceability limit state). Can be expressed as either ultimate bearing capacity (unfactored) or allowable bearing pressure (factored to reduce deformation)
CAPEX / OPEX	CAPEX: Capital expenditure related to non-routine remediation or renewal of an asset, for example an earthwork or culvert. OPEX: Operational expenditure related to routine maintenance activities, e.g. unblocking drainage, or vegetation management.
Consolidation	The process of volume reduction (through reduction in voids within a soil mass) due to the application of load to a soil mass, for example the construction of an embankment. Consolidation increases strength and stiffness of the soil.
Deep foundation	Foundations for structures that are formed by driving or drilling a reinforcement member through a weaker ground to bear on more competent material at depth. Load is supported through either end bearing or shaft resistance.

Deformation	Relates to the degree of movement experienced as a result of loading. Related to the soil stiffness, compressibility, and consolidation.
Effective shear strength	The magnitude of the shear stress that a soil can sustain during long term loading under effective stress conditions (fully drained)
Embankment	A constructed bank constructed from earth materials raised above the surrounding land to prevent flooding or improve vertical alignment.
Foundation soil	The natural soil layer on which an embankment or foundation is constructed
Geotextile	A woven or punched membrane used to provide tensile resistance within, for example, a pavement or earthwork
Ground model	An illustration of the materials, properties, behaviours, and mechanisms operating at a site. Commonly used to depict the application of construction activities and the consequences arising / pertinent issues for design.
Improved sub-grade	Where the sub-grade strength is lower than required, and improved sub-grade layer can be used, comprising higher quality material and in some cases a geotextile layer.
Load transfer platform	Load transfer platforms are used under embankments or structures for spreading vertical loads into an underlying foundation comprising discrete inclusions, such as piles or cement columns.
Plasticity & Atterberg Limits	Laboratory determine soil indicators, that relate to compositions and mineralogy. Often used in correlations to derive geotechnical parameters.
Pore-water pressure	Refers to the pressure of water held within voids in a soil mass. The pressure varies with depth below the phreatic surface and also responds to external loading conditions, depending on the rate of loading and permeability of the soil. For example, the undrained shear strength develops under short term loading conditions, and in a clay soil, the pore water pressures cannot dissipate – hence undrained behaviour that is governed by the presence of water in the soil. As long as stability of the soil is not affected (ability to withstand the applied load), the pore water pressures dissipate as consolidation occurs, and the soil mass strength is improved as the soil particles become more tightly packed. The pore water pressure is subtracted from the total stress level, to give effective stress.
Reflective cracking	Surface cracking in a pavement that is reflective of underlying behaviour.
Settlement – total / differential	Movement of the ground surface resulting from consolidation.
Shallow foundation	Foundations for structures that are constructed near surface in open excavations, generally comprising reinforced concrete strips or slabs.
Stability	Relates to the strength of the soil and ability to sustain an applied load. See also bearing capacity.
Sub-grade	The prepared earth surface on which the road pavement is constructed
Undrained shear strength	The magnitude of the shear stress that a soil can sustain during short term loading

Contents

Executive Summary

This Final Report presents the findings of the Ground Improvement for Khulna Soft Clay Soil research project. The Final Report is the concluding task of seven key milestones in the delivery of the project.

Background and Research Objectives

There are major concerns for the resilience of rural road embankments exposed to an aggressive coastal environment, in areas of high flood risk and where embankments are often constructed on soft soil deposits, sometimes with high compressible organic content. This study collates the relevant findings from existing research, field observations and ground investigation to understand the effectiveness and limitations of existing ground improvement techniques implemented in Khulna region, and to develop appropriate recommendations to overcome the typical construction challenges for road embankments and structures in Khulna region.

Data gathering and ground models

The sources of information used to develop the findings of the Final Report are wide and varied. The principal sources comprise; stakeholder meetings, national/international publications & academic research, site visits and intrusive investigations.

A review of 21No. sites was undertaken to present an overview of the existing field situation. Seven sites were identified for further detailed investigation. Field work at the test sites took the form of in-situ testing using a Panda 2 Probe and shallow hand excavated pits, from which samples were retrieved for geotechnical classification testing. Using the results gained from the above literature review, field surveys and laboratory testing stages, observational ground models have been developed for the 7No. sites to help better understand the deformation mechanisms and assess the likely contributory causes.

The probable cause/mechanisms leading to the defects that are attributable to soft ground are deformation resulting from settlement and stability issues resulting from low bearing capacity or lack of lateral support. Based on the geometrical and geotechnical data collated, simplified analytical models have been used to assess deformation and stability that contribute to the observed defects. Typically expected magnitudes of deformation have been presented, depending on loading.

Ground Improvement Techniques for Soft Soil in Khulna Region

Ground improvement techniques are presented that are considered either (a) technically feasible and (b) within the likely budget for rural road construction. From the literature review, and discussions with local stakeholders, the techniques are mostly in use in Bangladesh, although not necessarily for the rural road application. The ground improvement techniques considered here are: -

- Sand Compaction Pile
- Sand Drain (with surcharge)
- Prefabricated Vertical Drains (with surcharge)
- Geotextile basal reinforcement
- Cement Columns
- Excavate and replace / displacement

The impact of the various techniques with respect to ground deformation and stability is presented together with a review of the practicality of implementing each technique and the relative cost.

Guidelines/Recommendations for Ground Improvement in Khulna Soft Clay Soils

For repair to existing assets, the treatments have been sub-divided into Maintenance, Remediation and Renewal, with definitions of what each category means in terms of expected life and cost. The limitations of some ground improvement methods for remedial applications are also highlighted.

For new build, the design scenarios have been identified and key design attributes matched to solutions with increasing cost and likely effectiveness.

A series of design decision flowcharts have been presented to aid the practitioner in selecting the appropriate ground treatment solution to match the engineering requirement. These are summarised as follows: -

- Chart 1 Design Phase Process Diagram
- Chart 2 Conceptual Design Options Flow Diagram (Minor Bridge)
- Chart 3 Conceptual Design Options Flow Diagram (Minor Culvert)
- Chart 4 Conceptual Design Options Flow Diagram (Bridge Approach Embankment)
- Chart 4a Conceptual Design Options Flow Diagram (Bridge Approach Embankment – approach constructed after bridge)
- Chart 5 Conceptual Design Options Flow Diagram (General Embankment)

To accompany the design flowcharts, typical quantities for ground investigations for each engineering application has been presented as an informative Appendix.

Further research work

Based on the research, appropriate ground improvement techniques to address typical defects and new build scenarios are presented. Some of the techniques are widely used in Bangladesh, but others are not.

A long list of further research topics is presented and recommended topics given that are considered to offer most benefit to the development of suitable ground treatment of soft soils in Khulna Region. The methods where further research is recommended are: -

- Field Trials with sand compaction pile / sand drains to demonstrate plant and skills
- Effectiveness of Basal Reinforcement in limiting settlement and avoiding BC failure
- Shallow soil mixing with admixtures to improve shear strength of foundation soil
- Shallow soil mixing with fibres to improve shear strength of foundation soil
- Spatial mapping and statistical evaluation of data

The next phase of the project may include the following stages: -

- Design Guidelines: Production of design guidelines & training for ground improvement techniques for rural road construction on soft foundation soils including typical designs, standard details and specifications for standard engineering applications

- Construction Guidelines: Development of field guidelines for construction practitioners including capacity building / training of field engineers and construction practitioners for application of the new construction guidelines.

1 Background

1.1 Background to Project

There are major concerns for the resilience of rural road embankments exposed to an aggressive coastal environment, in areas of high flood risk and where embankments are often constructed on soft soil deposits with high compressible organic content. It is noted that the severity of the problem, where road embankments and structures can experience settlement failures relatively early in their lifespan; and the scale of the problem, with over 4500km of village roads, 800km of upazila roads, and 475km of union roads – combine to produce a significant problem that needs to be overcome.

There is a large body of existing research including geological and hydrological studies that are specific to the Khulna region, and further international studies, research projects and innovative engineering projects that have addressed the issues of durability, settlement and seismic behaviour for infrastructure founded on soft and compressible soils.

This study intends to collate the relevant findings from this existing research, to understand the effectiveness and limitations of existing ground improvement techniques implemented in Khulna region, and to develop appropriate recommendations to overcome the typical construction challenges for road embankments and structures in Khulna region.

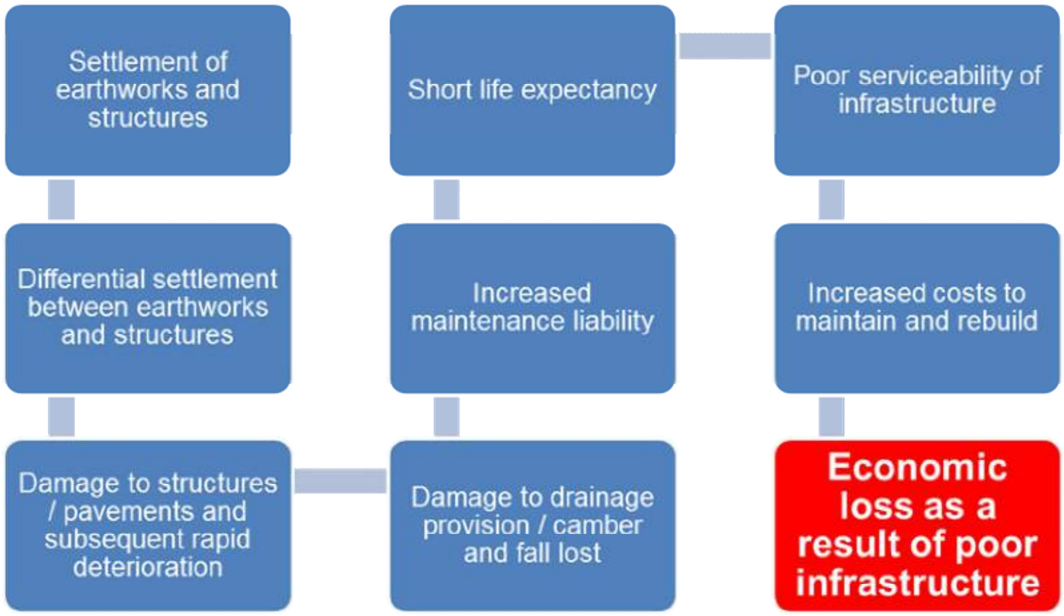
1.2 Failure mechanisms for soft soil and impacts

Whilst the specific failure mechanisms operating on structures and highways embankments in Khulna region are discussed in later Chapters, common failure mechanisms include:

- Settlement of earthworks;
- Failure of earthworks;
- Differential movement between earthworks and structures;
- Differential movement of bridge abutments and piers;
- Culverts structures settling as a result of earthworks construction;
- Creep settlement of organic materials.

Figure 1.1 below demonstrates that the potential impacts listed above, can lead directly to a cycle of economic loss. Investment in the design, methods of construction and maintenance of rural road assets provides significant improvement in transport links, trade and economic development and this is recognised by the Government of the People's Republic of Bangladesh.

Figure 1.1: Potential outcomes from building on soft ground



1.3 Research Objectives

The specific objective of this project identified in the Terms of Reference is 'to establish a cost-effective ground improvement technique(s) which will be applicable in Khulna and other similar regions which have soft soils' and this is to be supported by improvements to understanding in the following particular technical research area:

- The characteristics of the soil in the Khulna Region;
- The existing level of knowledge related to these soils;
- Identification of the current status of the structures in the Khulna Region and identify factors that are causing deterioration;
- Any geographical difference and possible reasons behind such differences;
- Recommendations of the remedial measures to existing structures and guidelines for ground improvements for the construction of new rural roads in the study region.

The final deliverable is a 'Final Report outlining the recommendations on appropriate remedial measures and ground improvement techniques based on the literature review, analysis of existing situation and testing of the soils'. Field and laboratory based research or trials of techniques are outside the scope of the project at this stage.

1.4 Methodology

The Project Methodology involves the completion of the following key tasks accompanied by a technical deliverable:

1. Inception Report and Literature Review;
2. Field Situation Analysis, including some diagnostic field tests;
3. Field and laboratory testing;
4. Laboratory Test Report;
5. Draft Report;
6. Stakeholder Workshop;
7. Final Report.

The Final report presents the findings from all previous phases of the research project. The Final report incorporates stakeholder feedback on the Draft Report and presentations held at the workshop.

2 Sources of information

2.1 Literature Review

A comprehensive literature review was undertaken as part of the project. The full list of publications reviewed is included in the References at the end of the report.

2.2 LGED Design Guidance and Procedural Documents

LGED have provided copies of current guidance and procedural documents that have been developed for use on rural road projects. The list of documents reviewed are noted in Chapter 8.

2.3 Stakeholder meetings

Numerous stakeholder meetings have taken place during the course of the project to directly gain information and request additional material. The records of the significant meetings are provided in Appendix A.

2.4 Field Investigations

Numerous sites, the locations of which were agreed with LGED, were visited to gain the background information on the current situation and defects typical of rural roads in Khulna. A total of 21No. sites were visited and relevant data recorded.

For seven selected sites, ground investigation was undertaken to gain information regarding the foundation soil and embankment fill materials. Soils were subject to in situ testing and laboratory testing,

2.5 Existing Field Investigations

Geotechnical data relating to soil stratification and properties were supplemented by existing ground investigation data and parameters determined from the literature review.

2.6 Stakeholder Workshop – summary report

Direct feedback on the research material gathered and recommendations presented at the Stakeholder Workshop held in September 2017. The record of the workshop is included in Appendix B.

3 Regional Geology and Ground Conditions

3.1 Introduction

The regional geology of Bangladesh and ground conditions in the Khulna Region has been widely described and a summary from the literature review is provided.

Also included is reference to existing borehole logs provided by Prosoil Consultants.

An overall summary of the expected ground conditions in the Khulna Region is given and this is used further in later Chapters of this report.

3.2 Regional Geology

Mollah (2003) explains in his paper entitled "Geotechnical conditions of the deltaic alluvial plains of Bangladesh and associated problems", that: the Himalayan Range, has had a great influence on the evolution of the river systems in Bangladesh and on the development of is extensive river delta, which has a sedimentation history ranging from the Pleistocene to Recent. Mollah explains that from development projects, there is soil test data available for almost the entire Bangladesh deltaic plain and that this information indicates that the subsoil of the deltaic alluvial plains generally consists of sandy material though he emphasises that it may vary significantly over short lateral distances as a result of frequent and erratic occurrences of compressible organic silt and peat mixtures.

The author explains and confirms the known issue that the foundation competency of the upper 6-10 m of ground in the deltaic alluvial plains of Bangladesh has been assessed as 'low to very low'. The characteristics of the soils indicate susceptibility to erosion, piping, liquefaction phenomenon, etc. Furthermore, he adds that the variable nature of the deposits makes the prediction of local soil conditions very difficult. Mollah explains that a uniformly textured fine sand with a 'moderate to high' bearing capacity typically exists in the soil profile, particularly below 20 m depth, but this is obviously a deep layer that would need to be utilized for its engineering properties via piled foundations (he notes that this moderately to high bearing capacity sand stratum is totally absent in swampy areas).

Mollah elaborates on potential construction problems and subdivides them into geotechnical and environmental issues. The former includes foundation problems in buildings, bridges, and hydraulic structures etc., attributed to the weak and erratic nature of the surficial soil (and points out that geotechnical problems are often compounded by the physiographical and geomorphological environment of the country). Flood and river bank erosion are identified as the main environmental problems, considered by the author as posing 'an enormous threat to life, property and construction'. Recommendations to overcome these problems are made.

Arifuzzaman and Hasan (2013) report how, over the past 50 years, Dhaka, the Capital of Bangladesh, has experienced a rapid growth of urban population. They explain how this high population increase has necessitated a rapid expansion of the city. Arifuzzaman and Hasan explain that those areas of Dhaka that have subsoil that can be considered competent for building construction are already exhausted and that as such, new areas are being reclaimed by both government and private agencies using dredged fill from nearby river sources. It is reported that in most cases, the practice for developing such areas is just to fill lowlands (1.5 m to 5.5 m) by dredged soils collected from nearby riverbank and riverbed sources. It is found that the dredged soil is typically silty sand and that the mean grain size and fines content of such material typically varies from 0.148 mm to 0.200 mm and from 17.4 to 27.6%, respectively.

Arifuzzaman and Hasan explain that a very soft organic layer exists below the reclamation filling layer that is highly plastic and highly compressible. This very soft organic layer is determined to exist because filling soil is directly placed on the marshy low land and upon the vegetation and other organic materials. After a time, these organic materials beneath the filling soil decompose and produce the problem soft organic layer. The authors found that the thickness of the soft layer varies in the range from 0.5m to 8.5m. Moisture content of organic layer was determined to vary from 32 to 84% whilst liquid limit and plasticity index vary from 42 to 193% and from 14 to 68%, respectively. It is seen that this organic soil (OL to OH) is very soft in nature and shows high moisture content and highly plastic behaviour. Organic content of the soft soil varies from 4.7 to 28.7%. Unconfined compressive strength and failure strain vary from 6 to 66 kPa and from 7 to 15%, respectively. In addition, initial void ratio, compression index and coefficient of consolidation vary from 0.88 to 3.90, from 0.26 to 1.10 and from 0.22 to 16.85 m2/yr, respectively. The authors summarise this soil as being highly compressible with very low shear strength. SPT N values of the organic layer are only 1 to 2. The filling layer was determined to possess SPT N values of 2 to 11.

The authors observed that settlement of the organic layer varied from approximately 240 mm to 640 mm between times period of 1.8 to 12.7 years, respectively, due to a calculated overburden pressure of 100 kPa. Moreover, the authors felt that the existing organic layer may cause negative skin friction and ensuing difficulties for piled foundations. It is mentioned that further studies are being conducted to develop or design desired suitable ground improvement techniques or alternative foundation systems for such sub-soil conditions.

3.3 Ground Conditions of Khulna Region

Of the materials sourced for the literature review, perhaps the key papers on the geology of the Khulna Region have emerged as “Engineering Geology of Khulna Metropolitan City Area” by Reshad et al (2004) and “Urban Geology: A Case of Khulna City Corporation, Bangladesh” by Adhikari et al (2006). These papers both reflect on metropolitan areas, but where the experience remains relevant to the ground conditions encountered in the rural road scenario.

Reshad et al (2004) explain that the presence of thick soft deposits is the major constraint for the development of the city. The ground of Khulna city is described as comprising ‘compressible and collapsible sediments’ consisting of very soft silt and organic clay-silt. These deposits are explained to be very moist, saline and where non-organic of low plasticity exhibiting low shear strength characteristics and a propensity for liquefaction. The paper attempts to divide the area into characteristic soil complexes and of those developed, the ‘Western Complex’ is described as having the worst subsurface conditions with upper soft soils up to 20m thick (correlating with the findings of Mollah (2003).

Adhikari et al (2006) again discuss how the city of Khulna lies on the Late Holocene - Recent alluvium of the Ganges deltaic plain in the north and Ganges estuarine plain in the south. They described how lithologically, the area is composed of coarse to very fine sand, silt, silty clay and clay in various proportions up to a depth of 300m, explaining how the stratigraphy underlying the city shows ‘seven cycles of sedimentation having age connotation from Upper Miocene to Recent age’.

Adhikari et al used data on geomorphology, stratigraphic litho-succession, soil types, percentage of sand, silt and clay in the soil, liquid limit, plasticity index, natural moisture content (NMC), liquefaction, settlement, and Standard Penetration Test (SPT) data to categorise the Khulna City area into four distinct zones where unit-I is best and unit-IV is ranked lowest for urbanization.

The authors report that the SPT 'N' values of the investigated areas ranged between 1 and 9 from the ground surface to 5mbgl and 1 to 27 from 5mbgl to 15mbgl with a range of liquid limit of 38 to 59%, a range of plasticity index of 9 to 30% and a natural moisture content (NMC) between 17% and 42%.

In line with the findings of Mollah and Reshad et al, Adhikari et al report that the shear strength of the upper subsoil horizons in Khulna City is low with a compressive index in the range of 0.123 to 0.335, indicative of a soil vulnerable to excessive settlement under high load. They explain that 40% of the sediments occur in the 'moderate to high compressibility' zone and the depth range for the sediments is generally less than 5m. Further to this, the authors state that the cohesive nature of soil in the Khulna City area with high colloidal content and high liquid and plastic limits indicates medium to high sensitivity of the soil to moisture, rendering it unable support heavily loaded buildings and structures. The authors add that the shallow ground water of Khulna City (typically 1mbgl) reduces the bearing capacity of the soil and subsoil up to 50%.

3.4 Existing Borehole logs

A number of borehole logs were provided by Prosoil Consultants for use on the research project. Boreholes in close proximity to selected test sites have been reviewed and are included in Appendix D.

The borehole logs provide details on the stratification of the soil and Standard Penetration (SPT) N results. The logs illustrate that where the soil is generally dominated by clay material, the SPT N value is low, between 1 and 5, to depths of 18-20mbgl, representing undrained shear strengths of between 4 and 25kPa, that are reflected in the description very soft to soft silty Clay. Where the soils are more predominantly sandy, the SPT N value is generally higher at over 10 representing a medium dense sand with effective angle of friction of 30-32°. Lower strength sand is present though, recorded at depth of up to 12mbgl, where the SPT N value is between 2 and 10 and this corresponds to a very loose to loose material, with an effective angle of friction of 28°. Groundwater is recorded close to ground level in most of the boreholes.

3.5 Review of geotechnical properties in the Khulna Area

A summary of the geotechnical parameters reported in the literature review is given in Table 3.1 through to Table 3.4 below.

Table 3.1: Summary of grading test data from literature review

Soil samples	Min	Max	USCS Classification
Clay fraction (%)	6[1]	28[1]	Single result of MH, others not defined
Silt fraction (%)	54[1]	88[1]	
Sand fraction (%)	3.5[1]	38[1]	
Organic content (%)	32[2]	71[2]	Not defined

Literature review reference:
[1] – Reshad et al (2004). Engineering Geology of Khulna Metropolitan City Area
[2] – Rafizul et al (2012). The effect of chemical admixtures on the geotechnical parameters of organic soil: a new statistical model

Table 3.2: Summary of plasticity test data from literature review

Soil samples from upper 3m depth	Min	Max	Plasticity classification
Moisture content (mc) %	33[2]	47[1]	Generally, CL, CI, MH, CH
Liquid Limit (LL) %	33[2]	62[1]	
Plastic Limit (PL) %	28[1]	37[1]	
Plasticity Index (PI) %	22[1]	31[1]	
Soil samples from 3m to 6m depth	**Min**	**Max**	**Plasticity classification**
Moisture content (mc) %	21[3]	58[2]	Generally, CL, CI, MH, CH
Liquid Limit (LL) %	27[3]	86[3]	
Plastic Limit (PL) %	18[3]	76[3]	
Plasticity Index (PI) %	NR	NR	No Results available
Soil samples 0m to 20m depth	**Min**	**Max**	**Plasticity classification**
Moisture content (mc) %	11[4]	88[4]	Generally, CL, CH, ML, MH. Also OH, OL.
Liquid Limit (LL) %	24[4]	95[5]	
Plastic Limit (PL) %	2[6]	80[5]	
Plasticity Index (PI) %	9[6]	90[5]	

Literature review reference:
[1] – Reshad et all (2004). Engineering Geology of Khulna Metropolitan City Area
[2] – Alamgir and Chowdhury (2004). Ground improvement methods recently practiced to solve the Geotechnical engineering problems in Bangladesh
[3] – Rabee and Rafizul (2012). Strength and compressibility characteristics of reconstituted organic soil at Khulna region of Bangladesh
[4] – Serajuddin, M (1998). Some geotechnical studies on Bangladesh soils: A summary of papers between 1957-96
[5] – Rafizul et al (2012). The effect of chemical admixtures on the geotechnical parameters of organic soil: a new statistical model
[6] – Adhikari et al (2006). Urban geology: a case study of Khulna City Corporation, Bangladesh

Results with no depth stated have been included in 0-20mbgl range

Table 3.3: Summary of specific gravity, void ratio, compression index data from literature review

Soil samples up to 3m depth	Min	Max	Notes
Specific Gravity	2.5[2]	2.75[3]	
Initial void ratio, e	NR	NR	No Results available
Compression index, Cc	NR	NR	No Results available
Soil samples 3m to 6m depth	**Min**	**Max**	
Specific Gravity	2[4]	2.73[10]	Low result indicative of organic content
Initial void ratio, e	NR	NR	No Results available
Compression index, Cc	0.257[3]	0.391[3]	
Soil samples 0m to 20m depth	**Min**	**Max**	
Specific Gravity	1.12[6]	2.88[3]	Low result indicative of organic content
Initial void ratio, e	0.706[5]	7.962[1]	High result indicative of organic content
Compression index, Cc	0.08[5]	2.179[3]	Large range due to soil types included

Literature review reference:
1 – Sarkar et al (2005). Interpretation of Rice Husk Ash on Geotech Properties of Cohesive Soil
2 – Reshad et al (2004). Engineering Geology of Khulna Metropolitan City Area
3 - Alamgir and Chowdhury (2004). Ground improvement methods recently practiced to solve the Geotechnical engineering problems in Bangladesh
4 - Rabee and Rafizul (2012). Strength and compressibility characteristics of reconstituted organic soil at Khulna region of Bangladesh
5 - Serajuddin, M (1998). Some geotechnical studies on Bangladesh soils: A summary of papers between 1957-96
6 - Rafizul et al (2012). The effect of chemical admixtures on the geotechnical parameters of organic soil: a new statistical model

Results with no depth stated have been included in 0-20mbgl range

Table 3.4: Summary of soil strength data from literature review

Soil samples up to 3m depth	Min	Max	Notes
SPT 'N' value	2[1]	10[1]	12-47 obtained for silty sand layers[6]
Undrained shear strength, c_u (kPa)	-	-	
Soil samples 3m to 6m depth	**Min**	**Max**	
SPT 'N' value	2[2]	4[2]	
Undrained shear strength, c_u (kPa)	12[2]	26[2]	Method of derivation not stated
Soil samples 0m to 16m depth	**Min**	**Max**	
SPT 'N' value	1[1]	29[1]	
Undrained shear strength, c_u (kPa)	12[2]	44[2]	Method of derivation not stated

Soil samples up to 3m depth	Min	Max	Notes
*Highly organic soils	Min	Max	
SPTC 'N' value	1[3]	6[3]	
Undrained shear strength, c_u (kPa)	18.3[22]	41[22]	Derived using laboratory Unconfined Compressive Test

Literature review reference:
[1] – Reshad et al (2004). Engineering Geology of Khulna Metropolitan City Area
[2] – Alamgir and Chowdhury (2004). Ground improvement methods recently practiced to solve the Geotechnical engineering problems in Bangladesh
[3] – Sarkar et al (2015). Prediction of soil type and standard penetration test (SPT) value in Khulna City, Bangladesh using general regression neural network
[4] – Rabee and Rafizul (2012). Strength and compressibility characteristics of reconstituted organic soil at Khulna region of Bangladesh

3.6 Summary of Geology

Drawing the records form the above discussion together, the geology in the Khulna Region can be summarised as:

- Very soft to soft sediments of up to 20m thickness should be expected. Firmer horizon beyond this depth as a result of normal consolidation.
- A composition containing fine silts and sands with sporadic pockets of organic clay-silts and peat;
- An SPT 'N' value of between 1 to 5 in clay materials with a corresponding low undrained shear strength especially at shallow depths;
- SPT N results generally higher where sand predominates the foundation soil, but density range can still be expected to range from very loose to dense within the 20m depth horizon.
- Typically, low plasticity for the fine silts and sands;
- Typically, intermediate to high plasticity for the clay soils;
- Typically, high to extremely high plasticity for the organic clay and peat materials;
- Medium to high compressibility; higher where organic material present in greater quantities.
- Groundwater close to surface <1mbgl;

The presence of peat is significant, as the moisture content can be in excess of 500% (this means the weight of peat soil is substantially due to the water content as opposed to the soil particles) and volume change under load can be very high. Peat soils also exhibit creep settlement that can occur for significant periods of time after the primary consolidation phase has finished, resulting in long term settlement of infrastructure constructed on these soils.

4 Review of Condition of Earthworks and Structures built on soft ground in Khulna Region

4.1 Purpose of Field Review

The overall purpose of the review was: -

- To identify typical sites where remedial works are required
- To identify the reasons why the defects occurred

A broad review of a wide range of sites was undertaken to present an overview of the existing field situation with the purpose of selecting a limited number of appropriate sites for more detailed investigation during Task 3 (Field and Laboratory Testing). Information relating to the sites was recorded using pro-forma to record common data including:

- Earthwork characteristics; length, height, slope angle, adjacent land;
- Sources of water; hydrology, drainage;
- Construction details; drainage, pavement, highway layout;
- Highway structures; bridges, culverts, walls;
- Observed condition; settlement, differential settlement, structural distress, drainage issues;

4.2 Site Selection Criteria

In partnership with LGED, Mott MacDonald developed the below list of classification criteria for the rural road study sites, that fall within three broad classes:

A. Road / Site characterisation.
B. Ground improvement techniques used?
C. Interaction with structures?

The classification criteria are as follows:

A. ROAD / SITE CHARACTERISATION:

1. What is the classification of the road?
   (E.g. upazila)
2. Proximity to a watercourse?
   (If yes, how close by is the watercourse?)
3. What is the elevation of the road?
4. Is the road at risk from flooding?
   (If yes, what are the specifics of this flood risk?)
5. Is the pavement coarse bound or unbound?
   (What materials have been used and in what layer thicknesses?)
6. Are there soft ground conditions?
   (If yes, is the depth of this soft layer(s) known?)
7. Are there instances of organic soils within this soft layer(s)?
   (If yes, what are the horizontal and vertical extents of this material?)

B. GROUND IMPROVEMENT TECHNIQUES USED?

8. Was the existing road constructed using any ground improvement techniques?
   (If yes, are design drawings available?)

9. Is the road on embankment or at grade?
(If on embankment, how high and what are the rough slope angles?)
10. Has the existing road / embankment failed or is it showing signs of distress?
(What are the signs of distress? what is the apparent mode(s) of failure? – if the road hasn't failed, why is this?)
11. Have remedial works been undertaken?
(If yes, are remedial design drawings available?)
12. Did the remedial works involve ground improvement?
(If yes, what techniques were employed?)
13. Were the remedial works successful?
(If yes why? If no why?)
14. Were further works then conducted if the remedial works failed?
(And did this extra work make a positive difference?)

C. INTERACTION WITH STRUCTURES?

15. Does the road interact with structures?
(If yes, what type of structures and how many? E.g. bridges, culverts, walls)
16. What foundations do the structures employ?
(Are design or as-built drawings available?)
17. Are the structural foundations competent or have they failed?
(And why?)
18. Are there issues with differential settlements at road-structure interaction points?
(What magnitude of differential settlement has occurred?)
19. Are there differential settlements outside of acceptable limits?
(Has the road failed, the structure or indeed both?)
20. Were remedial works undertaken?
(If yes, what was done? Are design and as-built drawings available?)
21. Were the remedial works successful?
(If yes why? If not why?)

4.3 Initial Site visits

The site visits were undertaken between 9th and 13th November 2016. A total of 21 sites were visited and these are listed in Table 4.1 below. The detailed observations for each of the 21 sites is included in the previously submitted Field Situation Report.

Table 4.1: List of sites visited

Site Number	District	Sub district	Road ID	Description / name
1.	Jessore	Abhoynagar	241043007	Sundali U.P.Office-Moshihati Bazar Road (Ch. 3107 to 5107m)
2.	Jessore	Abhoynagar	241042009	Alipur RHD-Sundali GC via Rajapur More, Ramsaradham, Arpara Clinic More, Horishpur Reg. Primary School Road
3.	Satkhira	Asasuni	28704200	Kadakati GC - Protapnagar GC via Goaldanga Bazer road at Ch. 4294-32500m
4.	Satkhira	Asasuni	287042003	Budhata RHD - Baka GC road at Ch. 7300-11100m
5.	Khulna	Paikgacha	247642012	Paikgacha GC- Gilabary GC Via Bagularchok Bazar Road.
6.	Khulna	Dacope	247172001	Khona R&H-Garikhali GC (Paikgacha) via Batbunia G.C Road
7.	Khulna	Dacope	247172011	Chalna GC (Gachtala)-Garaikhali GC (Paikgacha) via Laxmikhala & Mozamnagar hat Road.
8.	Khulna	Dacope	247174033	Dacope H/School(Bazar)-Madia Badurjhury culvert road
9.	Khulna	Batiaghata	247122007	Katianangla-Roypur via Sukdara Bazar, Baro Bhuiyan & Kechrabad Road
10.	Khulna	Dumuria	247302001	Dumuria-Baniakhali GC-Baroaria GC

Site Number	District	Sub district	Road ID	Description / name
11.	Khulna	Dumuria	247303003	Baliakhali bazar (Tipna R&H)-Kadamtola bazar-Madartala Bazar via Sovna UP Office Road
12.	Khulna	Rupsha	247752009	Khulna Mongla H/way Kudir Battala- Khajadanga - Hatamtala-Lockpur GC Mongla road via Shamontasena Nutun hat.
13.	Khulna	Terokhada	247942003	Harikhali R & H to patla hat GC
14.	Khulna	Terokhada	247943003	Terokhada Upazila HQ-Sachiadah UP office Road.
15.	Khulna	Terokhada	247942010	Near Katinga bazar
16.	Khulna	Dighalia	247403011	Bir Muktijorda Molla Jalal Uddin Sorak : Gazirhat U.P Office (Molladanga) -Bamondanga -Katenga G.C RHD Road (Digholia Portion)
17.	Khulna	Dighalia	247404005	Gazirhat Jangushia RHD (Bottala More) -Mohisdia RHD
18.	Bagerhat	Rampal	ID not given yet	Khulna coal based power plant connecting road (Under construction)
19.	Bagerhat	Rampal	201732016	Bhaga-Rampal Road
20.	Bagerhat	Mongla	201585005	Kainmary bridge-H/O Niren Biswas
21.	Bagerhat	Mongla	201582003	Mongla- Joymonir goal GC via Chila GC, Baddaiamary Bazar.

4.4 Observational Sites

4.4.1 Site observations / common themes

To categorise the sites to focus further investigation, common features have been identified and from these, representative sites have been identified.

The common features identified from the review of site data are given in Table 4.2.

Table 4.2: Examples of common defects observed

Feature / defect	Example
Differential settlement adjacent to structures (Site 21 – 201582003)	

Feature / defect	Example
Erosion leading to over-steepening of the embankment slopes (Site 3 – 287042008)	
Longitudinal cracking towards the edge of the road pavement (Site 10 – 247302001)	
Lateral spreading of the embankments (Site 11 – 247303003)	
Poor road surface conditions / potholes are widespread (Site 6 – 247172001)	

Feature / defect	Example
Retaining wall / structure deformation (Site 4 – 287042003)	

4.5 Discussion related to common observations

4.5.1 Differential settlement adjacent to structures

The observed defect is commonly found at sites that have soft ground conditions. The adjacent structure has been built with deep, piled foundations and is less susceptible to ground movement. The bridge approach embankments are laid directly onto the soft ground and increase in height towards the bridge, increasing the loading on the sub-grade. The result is that consolidation occurs under the embankment fill and a step between the embankment and bridge occurs. In the example given above, the side slopes are also in a poor condition (see Appendix E Site 21 – 201582003), having been eroded by the flow of the river, and this has exacerbated the development of the step. There are other instances observed at various sites, although not quite as pronounced as the example given.

4.5.2 Erosion

Many instances of embankment erosion were observed. Rising and falling water levels as well as direct erosion by flow of water / lapping (due to wind developing wavelets) can lead to loss of material, particularly fine fill materials with high silt and sand content, leaving a steep profile that will be unable to support vehicle loads and is susceptible to failure.

4.5.3 Longitudinal cracking towards the edge of the road pavement

Longitudinal cracking parallel to the pavement edge is generally indicative of poor lateral containment under traffic loading, or inadequate pavement construction thickness towards the outer edges of the road. On narrow roads, this will be particularly evident as vehicles travel very close to and beyond the pavement edge.

4.5.4 Lateral spreading of the embankments

The site inspections have revealed several examples where the earthwork embankments can be seen to be spreading outwards. The spreading is likely due to a combination of creep of soils under self-weight, transient vehicle loading and as a result of rapid-drawdown following periods of increased groundwater level such as floods or the manipulation of water levels for aquaculture (such as shrimp farming).

4.5.5 Poor road surface conditions / potholes are widespread

Most sites presented poor surface conditions for the road user because of many of the factors presented in the illustrations. Amongst other reasons, poor road surfaces can be caused by; soft ground causing failure/settlement of the embankment fill; deformation of poorly compacted or wet/soft embankment fill; inadequate pavement construction leading to erosion and pot holes. Failures associated with the underlying ground and embankment fill are geotechnical, whereas the latter is a pavement design issue.

4.5.6 Retaining wall / structure deformation

At several sites, retaining walls are used to provide additional support to the embankment. It is likely that the walls were installed to mitigate deformation and lateral spreading of the earthworks materials under vehicle loading. The walls may also have been installed to prevent erosion / over-steepening as described in the Section above.

The example above shows a wall that is deforming under the lateral load applied by the embankment, demonstrating inadequate foundation / embedment depth of the wall within the soft foundation materials. The deformation here is a slow progressive serviceability failure related to the ground conditions as opposed to a brittle failure related to the capacity of the retaining wall structure.

4.6 Intrusive Investigation Sites

Based on the site visits undertaken and the observations made above, the sites presented in Table 4.3 were subject to a more detailed investigation in Task 3. The reasons for selecting the sites are presented in the table. These sites are representative of the geometry, features and defects observed and it was considered that they would yield further useful information for interpretation of deformation mechanisms and development of practical solutions.

Table 4.3: Detailed investigation sites

Site Number	Description / reason for selection
3	To review and assess reasons for loss of edge support, poor surface conditions, and over-steep slopes / erosion.
10	Investigate the causes of longitudinal cracking
11	Investigate the spreading behaviour and poor road surface conditions
12	Review the condition of the palisade wall, bridge approaches and loss of edge support to the highway
13	Investigate the loss of support / slope instability at highway edge
15	Investigate the poor road surface conditions, deformation of retaining wall and steep side-slopes
21	Investigate bridge approach and differential settlement. Retaining wall supports.

4.6.1 Fieldwork

Based on the findings of the field observations made during the Task 2 Field Situation Analysis and on our investigation into commercially available plant and testing facilities summarised in the Inception Report, a programme of limited ground investigation and laboratory testing was proposed for the 7 No. sites.

In the absence of exploratory boreholes formed by cable percussion or rotary means, field work took the form of simple in-situ testing using a Panda Probe and shallow hand excavated pits, from which samples were retrieved for geotechnical laboratory testing. The Panda Probe is a light-weight dynamic cone penetrometer, which uses variable energy and can be operated by one man to test soils to a depth of up to 5m below ground level. The device is hammered into the ground and records material resistance.

A schedule of Panda 2 Probe test locations is presented in Table 4.4.

Table 4.4: Schedule of locations for Panda Probe testing.

Site	Site Name	Area	Test Number	Final Depth (mbgl)	Embankment Height (m)	Test Location Co-ordinates (N,E)
3	Assassuni	Road 287042008	1 2 3	4.7 4.7 4.5	1.2 1.2 1.1	22.59198, 89.21208 22.59184, 89.21209 22.59137, 89.21211
10	Dumuria A	Road 247302001	1 2 3	4.9 3.7 4.5	1.8 2.0 1.6	22.80609, 89.42352 22.80494, 89.42290 22.80522, 89.42314
11	Dumuria B	Road 247303003	1 2 3	4.5 4.2 4.8	1.6 1.6 0.9	22.7819, 89.377570 22.78181, 89.37769 22.78158, 89.37742
12	Rupsa	Road 247752009	1 2 3	4.6 4.5 4.8	0.4 1.5 1.8	22.78648, 89.62713 22.78648, 89.62698 22.78714, 89.62774
15	Terokhada A	Road 247942010	1 2	4.5 4.6	2.2 2.1	22.93618, 89.66647 22.93614, 89.66649
13	Terokhada B	Road 247942003	1 2 3	4.6 4.6 4.6	5.0 5.0 5.0	22.91289, 89.7049 22.91286, 89.70498 22.91294, 89.70477
21	Mongla A & B	Road 201582003	1 2 3 4	4.5 4.5 4.5 4.5	0.5 0.5 0.48 0.48	22.44176, 89.61001 22.44187, 89.60997 22.44218, 89.60809 22.44219, 89.60775

4.6.2 Shallow hand-excavated pits

To provide samples for laboratory geotechnical testing, 2-4 No. shallow hand excavated trial pits were formed at each of the 7 No. study sites. The trial pits were formed at both the top and bottom of the road embankments where conditions allowed. Soil samples were retrieved from varying positions within the earthwork embankments. All samples retrieved were sent to ProSoil Foundation's laboratory in Dhaka to undergo geotechnical laboratory testing.

A schedule of trial pit locations and depth is presented in Table 4.5 together with a schedule of tests undertaken. Full details are provided in the Laboratory Test Report.

Table 4.5: Schedule of samples taken for laboratory geotechnical testing.

Site Number	Trial Pit No.	Sample Depth (mbgl)	Location of trial pit on earthwork	Test Schedule Notes*
3	1.	0.3	Bottom	AL, MC, PSD
	2.	0.0	Top	AL, MC, PSD
	3.	0.3	Top	AL, MC
10	1.	0.25	Top	AL, MC, OC, PSD
	2.	0.25	Top	AL, MC, OC, PSD
	3.	0.3	Top	AL, MC, OC, PSD
11	1.	0.2	Bottom	AL, MC, OC, PSD
	2.	0.3	Top	AL, MC, OC, PSD
	3.	0.3	Top	AL, MC, OC, PSD
12	1.	0.3	Top	AL, MC, OC, PSD
	2.	0.3	Bottom	AL, MC, OC, PSD
	3.	0.3	Top	AL, MC, OC, PSD
15	1.	0.3	Top	AL, MC, PSD
	2.	0.3	Top	AL, MC, PSD
13	1.	0.3	Top	AL, MC, PSD
	2.	0.3	Top	AL, MC, PSD
	3.	0.3	Top	PSD
21	1.	0.25	Top	AL, MC, PSD
	2.	0.25	Top	None
	3.	0.3	Top	PSD
	4.	0.3	Top	PSD

*AL - Atterberg limits, MC - Moisture Content, OC - Organic Content, PSD - Gradings by sieve and hydrometer

5 Field and Laboratory Testing

5.1 Field Testing Results

Panda probe results plots are provided by site in Appendix E. There are 21 No. plots for the 7 No. sites which show cone resistance (in MPa) against depth for each test site. Table 5.1 below provides a summary of results.

Table 5.1: Summary of Panda Probe Results.

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
3	Assassuni	1	4.7	1.2	Reduction in soil strength below road surface from 0 to 0.9 mbgl, before steady increase to end of probe
		2	4.7	1.2	Sharp increase at shallow depth (in road formation) followed by reduction in soil strength below road surface from 0.1 to 0.9 mbgl, before steady increase to 2.0m where strength levels to end of probe
		3	4.5	1.1	General drop in soil strength to 1.5mbgl, before rising to a steady resistance value at 2.0mbgl.
10	Dumuria A	1	4.9	1.8	General slight rise in soil strength to 2.0m then levels off to end of probe.
		2	3.7	2.0	General slight rise in soil strength to 2.0m then levels off to end of probe.
		3	4.5	1.6	Reduction in soil strength below road surface from 0.3 to 0.9 mbgl, before steady increase to end of probe
11	Dumuria B	1	4.5	1.6	Variable strength within the embankment, levelling off to a consistent value from 1.0mbgl to the end of the probe.
		2	4.2	1.6	General slight rise in soil strength to end of probe.
		3	4.8	0.9	Reduction in soil strength below road surface from 0 to 0.7 mbgl, before steady increase to 2.0m then levels off to end of probe.
12	Rupsa	1	4.6	0.4	General slight rise in soil strength to end of probe. Zone of reduced strength from 0.6 to 0.9mbgl
		2	4.5	1.5	Reduction in soil strength below road surface from 0 to 0.7 mbgl, before steady increase to 3.0m then slight reduction to end of probe.
		3	4.8	1.8	Reduction in soil strength below road surface from 0.1 to 0.9 mbgl, before steady increase to 1.8 m then levels off to end of probe.

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
15	Terokhada A	1	4.5	2.2	Reduction in soil strength below road surface from 0.3 to 1.5 mbgl, before steady increase to end of probe.
		2	4.6	2.1	Slight reduction in soil strength below road surface from 0.2 to 1.3 mbgl, before steady increase to end of probe.
13	Terokhada B	1	4.6	5.0	Slight reduction in soil strength below road surface from 0.0 to 2.4 mbgl, before sharp, then steady increase to end of probe.
		2	4.6	5.0	Reduction in soil strength below road surface from 0.0 to 0.9 mbgl, before levelling off to 2.0m, then steady increase to end of probe.
		3	4.6	5.0	General slight rise in soil strength to end of probe.
21	Mongla A & B	1	4.5	0.5	Reduction in soil strength below road surface from 0.1 to 0.4 mbgl, before steady increase to 2.0 m then levels off to end of probe.
		2	4.5	0.5	General slight rise in soil strength to end of probe.
		3	4.5	0.48	Reduction in soil strength below road surface from 0.2 to 0.7 mbgl, before increasing to 2.0m, then levelling off to end of probe
		4	4.5	0.48	Reduction in soil strength below road surface from 0.2 to 0.8 mbgl, before slight increase to end of probe

5.2 Interpretation of Probe Test Results

The results returned from the Panda Probe testing were remarkably consistent across the 7 No. study sites. The cone resistance values are used to provide relative strength rather than absolute values.

For all 7 No. sites tested with the Panda Probe, a marked difference in cone resistance was observed between the placed embankment fill and the underlying natural ground. It was clear that the embankment fills typically displayed a cone resistance of 1.5 MPa to 4 MPa. In the near surface extents of the embankment fill, cone resistance on occasions reached as high as 20 MPa, reflecting the presence of crushed brick content from highly degraded pavement or patch repairs. It is considered likely that the embankment fill underwent no formal compaction during placement, and has experienced little consolidation through self-weight, due to the low height of the embankments and frequent wetting and drying events. This is evidenced on some probes by a decreasing cone resistance towards the base of the embankment.

As the probe advanced into the underlying natural ground (typically between 1.5 and 2mbgl), the resistance then rose steadily with depth to around 10 MPa from 3mbgl to 4.5 mbgl. The steady rise in cone resistance with depth is likely to reflect both normal consolidation of the sediments and also the effects of consolidation of the soils resulting from the embankment loading / overburden. Control probes to the side of the embankments were not possible due to flood waters / aquiculture, so the change in cone resistance as a result of embankment loading cannot be separately quantified.

5.3 Laboratory Testing Results

The testing of samples retrieved from the site works has been conducted in accordance with the applicable American Society for Testing Materials (ASTM) standard for soil testing;

- ASTM D2487 - 11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
- ASTM D422-63 Standard Test Method for Particle-Size Analysis of Soils ASTM

5.4 Summary of laboratory testing undertaken

Laboratory test results from of the samples gathered during the Field & Laboratory Testing stage are presented in Appendix E. A summary of the test results is provided in Table 5.2 through to Table 5.4.

Particle Size Distribution (PSD), Atterberg Limit testing, shrinkage limit, linear shrinkage and organic content testing was scheduled for the samples retrieved from each trial pit.

Table 5.2: Summary of plasticity and organic test results

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	Liquid Limit LL (%)	Plastic Limit PL (%)	Shrinkage Limit SL (%)	Linear shrinkage L_s (%)	Plasticity Index PI (%)	Moisture Content (%)	Organic Content (%)
3	1.	0.3	Clay CH - Fat Clay	52	28	-	-	24	8	-
	2.	0.0		NP	NP	-	-	NP	36	-
	2b.	0.0	Silty Sand CL - Lean Clay	27	13^	-	-	14	-	-
	3.	0.3		62	19	11	26	43	36	-
10	1.	0.25	Clay CL - Lean Clay	47	21	14	18	16	26	6
	1b	0.25	Clay CL – Lean Clay	44	23^	-	-	21	-	-
	2.	0.25	Clay CL - Lean Clay	49	21	-	-	28	27	9
	3.	0.3	Clay CL - Lean Clay with Sand	46	20	-	-	26	13	7
11	1.	0.2	Clay CH – Fat Clay with Sand	49	25	-	-	27	28	2
	2.	0.3	Clay CH - Fat Clay	58	25	-	-	33	30	5
	3.	0.3	Clay CH - Fat Clay	59	25	13	22	34	33	7
	3b.	0.3	Clay CH – Fat Clay	79	27^	-	-	52	-	-
12	1.	0.3	Clay CL – Lean Clay	44	23	-	-	21	25	7
	2.	0.3	Clay CL – Lean Clay	48	20	-	-	28	28	8
	3.	0.3	Clay CH - Fat Clay	54	26	10	36	29	32	8

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	Liquid Limit LL (%)	Plastic Limit PL (%)	Shrinkage Limit SL (%)	Linear shrinkage L_s (%)	Plasticity Index PI (%)	Moisture Content (%)	Organic Content (%)
15	1.	0.3	Clay CL – Lean Clay	41	21	-	-	20	19	-
	2.	0.3	Clay CL – Lean Clay	42	21	-	-	21	14	6
13	1.	0.3	Clay CL – Lean Clay	43	28	-	-	17	17	-
	2.**	0.3	Clay CH - Fat Clay	47	21	11	28	26	26	4.5
	3.	0.3	Fine Sand SM – Silty Sand	-	-	-	-	-	-	-
21	1.	0.25	Clay CH – Fat Clay with Sand	54	21	-	-	33	27	-
	2.	0.25		-	-	-	-	-	-	-
	3.	0.3	Clay CL – Lean Clay	41	24	-	-	17	-	-
	4.	0.3	Fine Sand SM - Silty Sand	-	-	-	-	-	-	-

* Unified Soil Classification System; LL – Liquid Limit; PL – Plastic Limit; SL – Shrinkage Limit; L_s – Linear Shrinkage; PL – Plastic Limit
^ Results calculated using standard relationships, no laboratory results available
**variability within the recovered sample – PSD identified as fine sand, and Atterberg limits as a Fat Clay.

Specific Gravity (G_s) testing was scheduled for the samples retrieved from trial pits, the results are detailed in Table 5.3.

Table 5.3: Summary of specific gravity test results

Site Number	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	Specific Gravity G_s
10	3	0.3	Clay CL - Lean Clay with Sand	2.83
11	3	0.3	Clay CH - Fat Clay	2.82
15	2	0.3	Clay CL – Lean Clay	2.80
13	3	0.3	Fine Sand SM – Silty Sand	2.83
21	3	0.3	Clay CL – Lean Clay	2.69

* Unified Soil Classification System

Particle Size Distribution (PSD) testing was conducted on 21 No. trial pit samples. The PSD plots are presented in Appendix E. A summary of the grain size distribution presented in the plots is give in Table 5.4.

Table 5.4: Summary of PSD results

Site	Trial Pit	Gravel % (75 to 4.75mm)	Coarse Sand % (4.75 to 2.00mm)	Medium Sand % (2 to 0.425mm)	Fine Sand % (0.425 to 0.075mm)	Silt % (0.075 to 0.005mm)	Clay % (<0.005mm)
3	1	0.00	0.00	0.00	1.75	46.38	51.86
3	2	0.00	0.11	2.45	28.25	53.56	15.64
10	1	0.00	0.00	0.00	6.00	53.08	40.92
10	1b	0.00	0.00	0.00	8.95	79.30	11.74
10	2	0.00	0.00	0.00	10.41	62.14	27.45
10	3	0.00	0.00	0.00	16.05	62.52	21.42
11	1	0.00	0.00	0.00	17.28	27.66	55.06
11	2	0.00	0.00	0.00	2.13	52.31	45.56
11	3	0.00	0.00	0.00	2.18	51.70	46.11
11	3b	0.00	0.00	0.00	2.60	46.36	51.04
12	1	0.00	0.00	0.00	5.97	74.68	19.35
12	2	0.00	0.00	0.00	6.73	52.25	41.02
12	3	0.00	0.00	0.00	9.78	31.18	59.04
15	1	0.00	0.00	0.00	14.73	59.82	25.45
15	2	0.00	0.00	0.00	5.45	79.00	15.55
13	1	0.00	0.00	0.00	9.99	68.75	21.27
13*	2	2.58	0.60	0.37	55.95	40.51	0.00
13	3	0.00	0.14	3.43	78.81	17.62	0.00
21	1	0.00	0.00	0.00	20.41	32.88	46.71
21	3	0.33	0.54	1.23	65.17	32.73	0.00
21	4	0.00	0.02	1.05	69.38	21.69	7.85
	Minimum	**0.00**	**0.00**	**0.00**	**1.75**	**17.62**	**0.00**
	Maximum	**2.58**	**0.60**	**3.43**	**78.81**	**79.30**	**59.04**
	Average	**0.14**	**0.07**	**0.41**	**20.86**	**49.82**	**28.72**

*variability within the recovered sample – PSD identified as fine sand, and Atterberg limits as a Fat Clay.

The material is principally fine sand to clay; summarised as follows: -

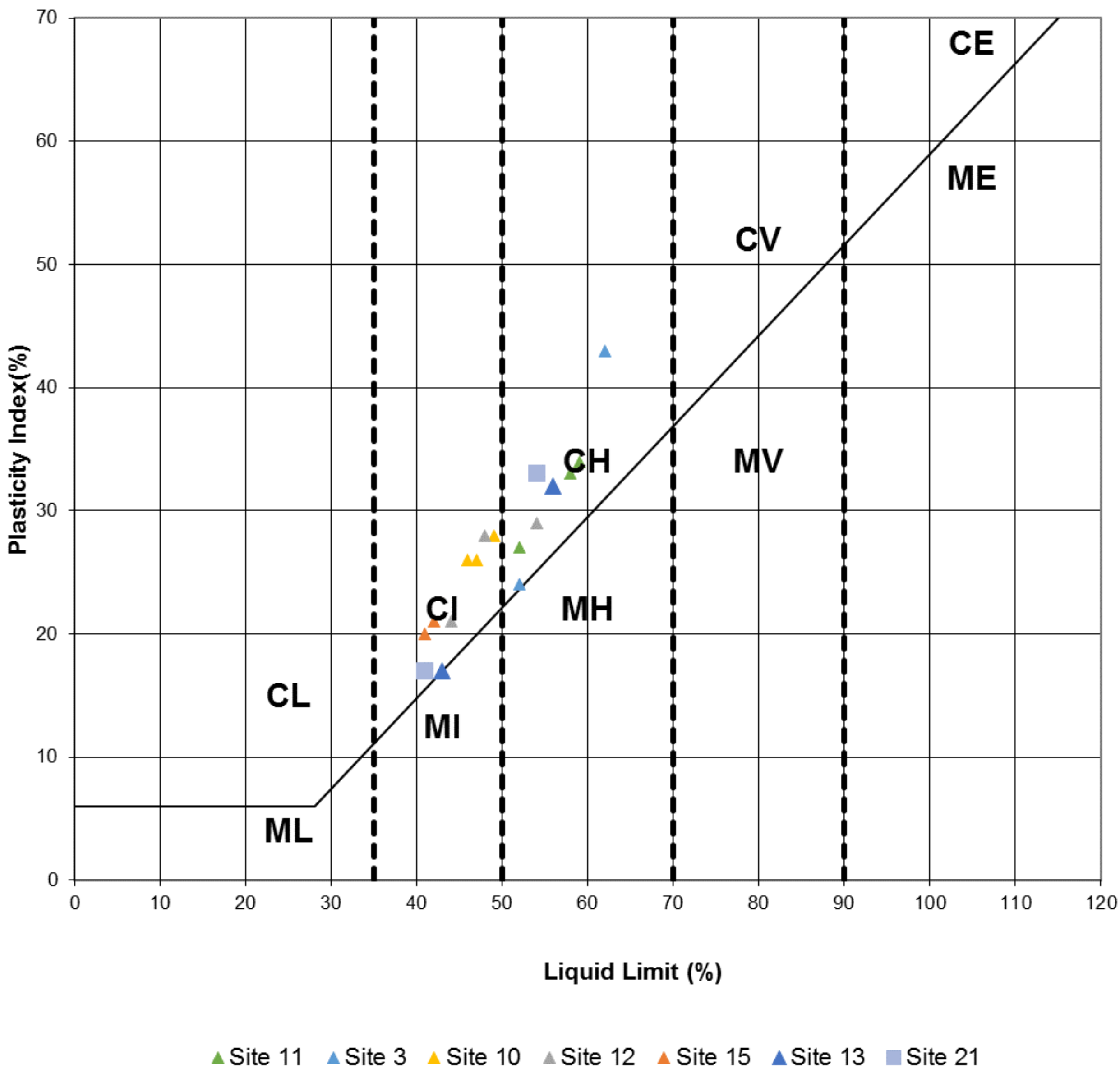
- All trial pit samples contained Fine Sand, the proportion ranging between 2% and 79% with an average of 22%
- All trial pit samples contained Silt size material, the proportion ranging between 18% and 80% with an average of 50%.
- Clay material was found in most samples, with the proportion ranging between 8% and 59% with an average of 29%.
- Variability within samples is observed, and within trial pits at the same site, illustrating localised changes in soil composition (typical of depositional environment).

5.5 Derivation of Geotechnical Parameters

The results of the Atterberg Limit testing are presented in the Tables below. The results are categorised into Fat Clay and Lean Clay and the locations where these soils were found is identified on Table 5.2. Some sites have identified only one clay soil type, and others have both e.g. Site 12.

In accordance with the Unified Soil Classification System (USCS), the Fat Clay has high plasticity (high volume change potential with varying moisture content) and Lean Clay has low plasticity (low volume change potential with varying moisture content). The plasticity results are presented on the A-Line chart of Figure 5.1. The Lean Clay plots as a clay of intermediate plasticity on the chart (CI).

Figure 5.1: A-Line Plasticity Chart



The classification test results are summarised in Table 5.5.

Table 5.5: Summary of plasticity test results

Fat Clay (CH)	Max	Min	Count	Average
Moisture content (mc) %	33	26	6	29.3
Liquid Limit (LL) %	79	47	8	56.5
Plastic Limit (PL) %	28	21	7	24.4
Shrinkage Limit (SL) %	13	10	3	11.3
Linear Shrinkage (L_S) %	36	22	3	28.7
Plasticity Index (PI) %	52	24	8	32.3
Liquidity Index (LI)^	0.24	0.11	6	0.17

* Results based on laboratory test results only
^ Calculated using LI= (mc-PL)/PI

Lean Clay (CL)	Max	Min	Count	Average
+Moisture content (mc) %	28 (28)	13 (13)	8 (8)	21.1 (21.1)
Liquid Limit (LL) %	49 (49)	27 (27)	13 (13)	43.4 (43.4)
Plastic Limit (PL) %	28 (28)	20 (20)	11 (9)	22.5 (22.1)
Shrinkage Limit (SL) %	22 (14)	14 (14)	2 (1)	N/A
Linear Shrinkage (L_S) %	18 (18)	18 (18)	1 (1)	N/A
Plasticity Index (PI) %	28 (28)	14 (14)	13 (13)	23.0 (23.0)
Liquidity Index (LI)^	0.38 (0.38)	-0.55 (-0.55)	8 (8)	-0.01 (-0.01)

* Results based on a combination of laboratory and literature review test results; results in brackets are based on laboratory test results only.
^ Calculated using LI= (mc-PL)/PI
+Site 3, Trial Pit 1, 0.3mbgl sample moisture content discounted as anomalous

Shear vane, shear box or triaxial compressive strength tests on undisturbed samples are required to assess soil strengths. Such test results were not available at the time of writing. However, Liquidity Index (LI) values calculated from a relationship between moisture content, Plastic Limit and Plasticity Index can be used to provide an indication of the undrained shear strength of the soils. The natural moisture content of a soil relative to the liquid and plastic limits can be represented by means of the Liquidity Index LI, where

$$LI = \frac{mc - P_L}{PI}$$

The LI will vary from 0 at their PL to 1 at their LL. The range of LI of -0.7 to 0.4 indicate that the moisture contents of the clays are typically closer to their PL than their LL.

The shrinkage limit (SL) of a soil is the water content, expressed as a percentage of the weight of the soil, at which further loss in moisture will not cause a decrease in its volume. Linear shrinkage is also calculated as part of the shrinkage limit test and this is defined as the decrease in one dimension of a soil mass, expressed as a percentage of the original dimension at the shrinkage limit. When considered in relation to the natural moisture content of in-situ soils, the Shrinkage Limit can be used to determine whether further shrinkage of a soil will occur if allowed to dry out.

Comparison of the shrinkage limit test results for the Fat clays with their moisture content values indicates that these soils are likely to have significant shrinkage potential. A single shrinkage limit test undertaken within the Lean clays again indicates the potential for shrinkage within these soils, however a single result is inconclusive.

An assessment of the extent of swelling potential should take account of the soils mineralogy. However, typically clay soils with LL values of less than 35% and PI values of less than 12% are considered to have low swelling potential (US Army Corps of Engineers, 1990). Given the range of LL and PI values recorded the Fat and Lean clays are both considered to be of medium to high swelling potential. In general, the results indicate higher swelling potential within the Fat clays.

The plasticity index can be used to derive geotechnical parameters for the soil horizons using standard correlations presented in British Standard BS 8002:2015 Code of practice for earth retaining structures. Using the correlation presented in Table 2 of BS8002, for the PI results presented for the Fat Clay, the critical effective angle of friction, Ø' would range between 22° and 25°, with an average value of 24°. For the Lean Clay the effective critical angle of friction, Ø', would range between 25° and 27°, with an average value of 26°.

In terms of engineering properties, the behaviour of the clay materials differs only slightly and in situ variability and ability to define the different horizons in the field would indicate that the more conservative geotechnical parameters e.g. for the Fat Clay, would be adopted for design.

Effective cohesion, c', for the normally consolidated clay soils present would be 0kPa.

Organic content varied from 2-9% where tested and is a relatively small constituent unlikely to have significant bearing on the material properties. Where organic material is identified in greater concentration, for example peat or decaying plant matter, the potential for settlement of overlying soils is greater. It is known that significant organic clay and peat material is present in the Khulna Region, and organic content of this material is noted in Chapter 3.

5.6 Review of site specific parameters vs regional data from literature review

A review of the parameters determined from the site-specific investigation and that from literature has been undertaken, and it is observed that the parameters presented are similar and appear representative. This is with the exception of organic soils, that were not encountered in the test pits.

The grading test data from the literature review show that the soils in the study area are typically fine-grained soils ranging between sandy silts (SM) to high plasticity clays (CH). Organic content is variable ranging from 0% to very high organic contents up to 71%.

The plasticity test data show similar results to site specific laboratory tests with the majority of soils of low to high plasticity depending on their clay content. Although the data is limited there is no appreciable difference between the soils composition or moisture content over the GL and 3m depth range and the underlying soils between 3m to 6m. Moisture contents from the test sites are lower however, and this may be due to the shallow samples, or some drying experienced during sampling.

Undrained shear strength test data are within the very soft to firm range over the upper GL to 3m layer. The 3m to 6m depth range show results in the very soft to soft range. The cone testing in the near surface of the foundation soil, together with the SPT N values from existing BH data supports this.

Specific gravity data from the literature review of 2.50 to 2.77 are lower than the site-specific test results of 2.69 to 2.83, potentially illustrating more organic matter within the soil samples tested and reported in existing literature.

6 Development of Ground Models

6.1 General

For each of the sites where intrusive investigation was undertaken, the initial observations for the site together with an interpretation of the deformation mechanisms have been developed into observational ground models, and these are presented in detail in Appendix F. A Technical Note was prepared by Mott MacDonald (27th March 2018) that provides a detailed explanation of the ground model development in relation to consolidation.

Using the data from the literature review, existing borehole records and field and laboratory test data from the seven sites, a quantitative analytical ground model has been developed to represent a typical site constructed in in a rural location on soft foundation soil. The analytical model allows deformation and stability to be considered and general observations and recommendations to be made in later Chapters.

6.2 Review of key findings / indicators

Table 6.1 below lists the typical defects that were observed during the field visits at all sites and indicates the probable primary and secondary causes that have been identified through the ground models developed and presented in Appendix F.

The defects are categorised as follows: -

- Embankment Defects;
- Structure Defects and;
- Pavement Defects.

The probable cause/mechanisms leading to the defect can also be categorised: -

1) Deformation
    a) Differential settlement
    b) Total settlement
2) Stability
    a) Bearing capacity
    b) lack of lateral support (from foundation soil)
3) Floodwater (erosive action, pore-pressure increase)
4) Adjacent land use (aquiculture, other)
5) shrink/swell cycle (of soils)
6) Embankment construction (quality, materials)
7) Vehicle overloading

Items 1 and 2 can be partially or wholly attributed to soft foundation soil.

Table 6.1: Summary of Typical defects in embankments, structures and pavements and probable cause/mechanism

Typical defect / indicator	Probable Primary cause(s)*	Probable Secondary cause(s)*
Embankments		
The side slopes are eroded / oversteep	Embankment construction (quality, materials) Adjacent land use	Floodwater erosive action
The embankment is spreading	Embankment construction (quality, materials) Shrink/swell cycle of embankment fill	**Total settlement** **Bearing capacity**
The side slopes have failed (rotational failure through foundation soil foundation)	**Bearing capacity**	Embankment construction (quality, materials) Adjacent land use Floodwater (pore-pressure increase)
The side slopes have failed (within embankment material)	Embankment construction (quality, materials, geometry) Adjacent land use Floodwater (pore-pressure increase)	Vehicle overloading Shrink/swell cycle of embankment fill
Leaning trees on slopes (indicative of spreading / slope failure)	Embankment construction (quality, materials)	Shrink/swell cycle of embankment fill **Bearing capacity**
Settlement / loss of alignment	**Total settlement**	**Bearing capacity**
Side slopes are eroded	Floodwater Adjacent land use	
Structures		
There is a step between bridge/culvert and approach	**Differential settlement** Embankment construction (quality, materials)	**Bearing capacity** Shrink/swell cycle of embankment fill / structure backfill
Culvert blocked / below water line	**Total settlement**	
Retaining structure deformed	**Bearing capacity / lateral support from foundation soil**	**Total settlement** Shrink/swell cycle of backfill
Abutments deformed / mis-alignment	**Bearing capacity**	**Total settlement**
Pavements		
There are lateral cracks in the pavement	Vehicle overloading Embankment/pavement construction (quality, materials)	**Total settlement** Shrink/swell cycle of embankment fill
The pavement is undulating / disturbed	Vehicle overloading Embankment/pavement construction (quality, materials)	**Total settlement** **Bearing capacity** Shrink/swell cycle of embankment fill
Wheel rutting of the pavement	Vehicle overloading Embankment/pavement construction (quality, materials)	
Damaged shoulder	Vehicle overloading Embankment/pavement construction (quality, materials)	Adjacent land use Flooding

*probable causes in bold type are related to soft foundation soil

6.3 Analytical Ground Model

6.3.1 General

Based on the geometrical and geotechnical data collated, simplified models have been used to assess the two principal factors (related to the foundation soil) of deformation and stability that contribute to the observed defects.

To undertake modelling, a standardised ground profile has been adopted based on the following: -

- Standard Penetration Test (SPT) N profile from existing BHs in proximity to Site 3, Site 10 and Site 11 have been used to determine undrained shear strength with depth; these boreholes exhibit significant thickness of clay material.
- In addition, assuming no previous construction has occurred (no previous consolidation of the soil) a normally consolidated soil profile has been considered.
- A 20m depth horizon has been considered, comprising very soft to soft clay.
- For specific sites in Khulna, the soil profile will vary and is likely to be a combination of alluvial soils, including clays, silts, fine sands and organic materials. A simplified model is developed where both Lean and Fat clay composition is considered, and differences noted.
- A 2m and 4m high embankment is modelled that is typical of rural roads
- Only the influence of the foundation soil is considered e.g. self-settlement of embankment and stability of the embankment slope itself is not included.
- Only embankment construction is considered
- Consolidation settlement only considered using elastic theory – creep settlement likely to occur due to presence of organic material, but not quantified.
- Undrained stability only considered – for embankment foundations, this is the most critical case.

The outputs of the assessment provide: -

- Typical magnitude of deformation that would be expected for embankment construction at 2 and 4m height
- The theoretical change in foundation soil material properties below the embankment as a result of the deformation (consolidation) caused by the loading (useful for assessment of ground treatment methods relating to surcharging or densification).
- The factor of safety (in the undrained case) and hence stability that would be expected for embankment construction at 2 and 4m height (at the end of construction phase).

6.3.2 Deformation

6.3.2.1 Geotechnical Parameters

In support of the data from literature review presented in Chapter 3 and laboratory test data presented in Chapter 5, Figure 1 of BS 8002:2015 has also been used to obtain a bulk unit weight for both a Fat and Lean Clay and embankment fill. Bulk unit weight of 17kN/m3 has been assigned to the foundation soil (irrespective of whether Lean or Fat Clay) and to represent embankment fill that has been compacted (although no field testing was undertaken), a bulk unit weight of 19kN/m^3 is adopted.

A Poisson's ratio (v') of 0.2 and undrained Poisson's ratio (v_u) of 0.5 has been assumed for both a Fat and Lean Clay based on recommendations presented by Bowles (1997).
Values for undrained shear strength, c_u, have been derived from a SPT-N Value correlation using equation [1] below from Tomlinson (1995).

$$C_u = f1 \times \text{SPT-N Value} \quad [1]$$

Where f1 is a factor related to the plasticity index of the material, and can be derived with reference to Figure 1.5 of Tomlinson (1995). For the plasticity index values presented as mean for Fat and Lean Clay (32% and 23% respectively), the f1 values are 4.5 and 4.9.

A theoretical normally consolidated (NC) undrained shear strength, c_u, profile has been derived from a c_u/σ'_v ratio correlation with plasticity index shown by equation [2] below from Craig (2004). This has been used in comparison to a c_u derived from SPT-N Values.

$$C_u = 0.11 \times \sigma'_v + 0.0037 \times I_p \times \sigma'_v \quad [2]$$

Where σ'_v is the effective vertical stress assuming groundwater at ground level. The profiles are shown on Figure 6.1 and Figure 6.2. Note that the SPT N derived Cu profile is the same for both Figure 6.1 and Figure 6.2 as these are derived from the same source. The theoretical approach uses the average plasticity index for the soil types and the profile is hence different.

A design line for Young's modulus, E, shown by was derived for both a site specific correlated c_u and a theoretical c_u shown by equation [3] below from Burland et al (1979) based on a strain level of 0.4%, representative of very soft clay reported in Bowles (1997).

$$Eu = M * Cu$$

$$E' = \frac{(1+v')}{(1+vu)} \times Eu$$

Where M is 400 for a strain level of 0.4% and v' and v_u is the drained and undrained Poisson's ratio.

6.3.2.2 Design methodology

A serviceability limit state approach has been adopted for the assessment of total settlement for the construction of an embankment on Fat and Lean Clay.

Vertical displacement calculations were undertaken with the use of Oasys Pdisp – Version 19.3 Oasys (2017) to calculate total settlement based on elastic theory using Boussinesq analysis method for constructing a 2m and 4m high embankment on 20m of soft soil comprising Fat or Lean Clay with the stiffness profile determined from the process outline above.

An un-factored load representing a 2m and 4m high embankment of 38 and 76kN/m^2 respectively, has been applied at ground level across an assumed embankment length of 40m and width of 10m. Greater width of road influences (increases) the depth to which the deformation is experienced.

Elastic theory has been used as opposed to one-dimensional consolidation theory, as this is appropriate for a thick layer of relatively homogenous (in terms of strength) alluvial material as is common in Khulna Region.

Figure 6.1: Undrained shear strength with depth determined from SPT N and theoretical normally consolidated approach (Fat Clay)

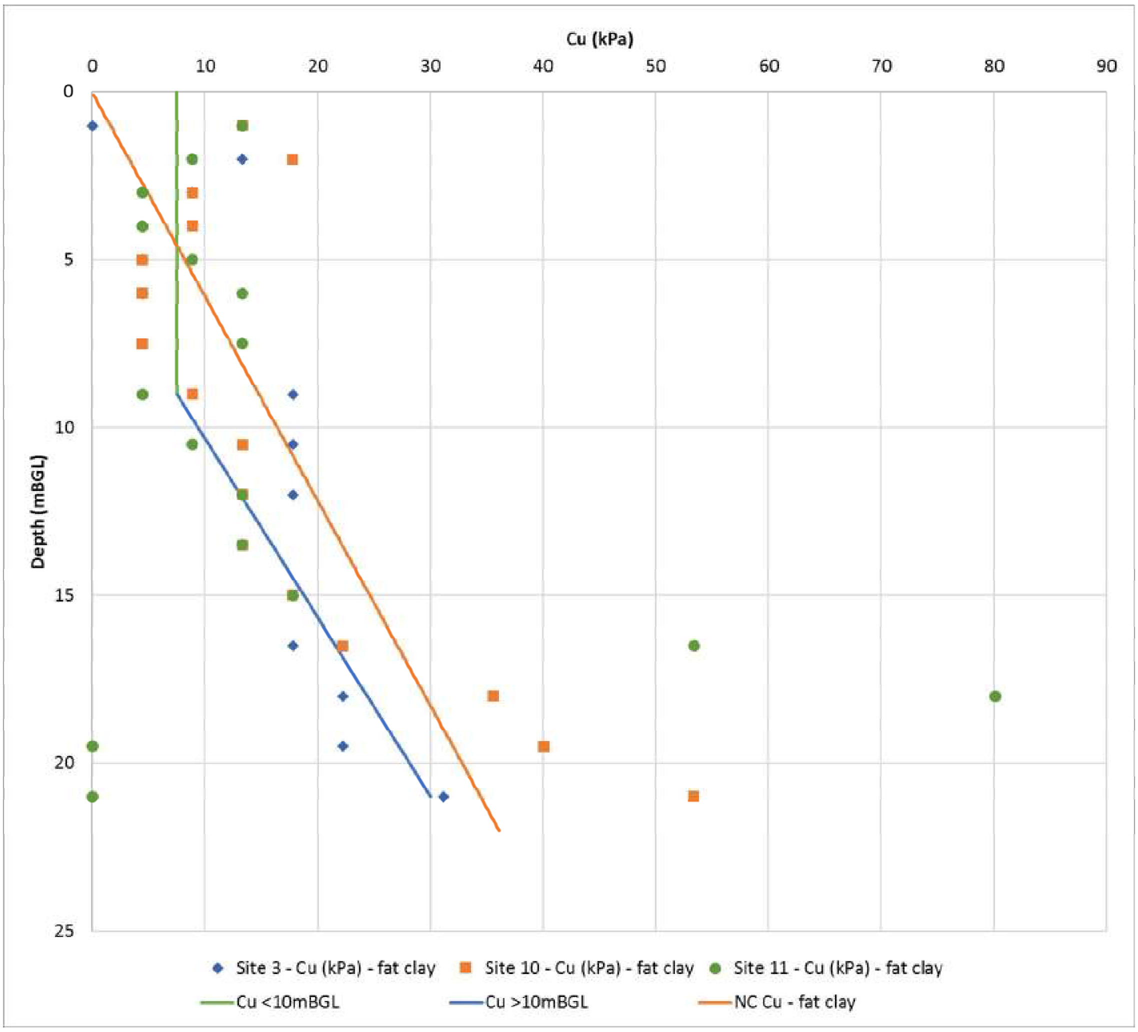
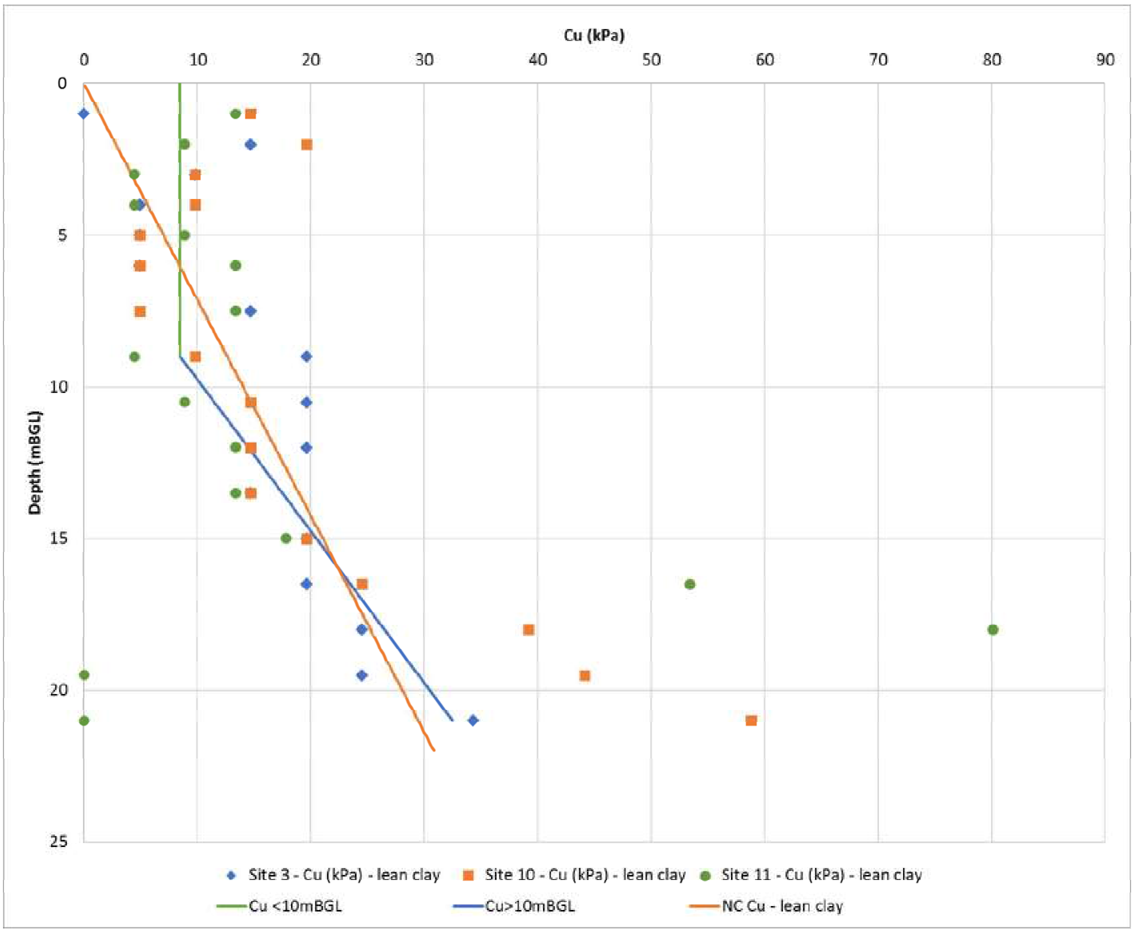


Figure 6.2: Undrained shear strength with depth determined from SPT N and theoretical normally consolidated approach (Lean Clay)



6.3.2.3 Predicted Settlement

The results of maximum total settlement due to constructing a 2m and 4m embankment on Fat and Lean Clay for stiffness parameters derived from a SPT-N value correlation and a theoretical correlation for a normally consolidated (NC) soil have been summarised in Table 6.2 below.

Table 6.2: Predicted settlement for 2m and 4m embankments on standardised ground model.

Geology	Cu profile	Embankment Height (m)	Maximum total settlement (mm)
Fat Clay	Cu derived from SPT-N Values	2	140
Fat Clay	Theoretical NC Cu	2	148
Fat Clay	Cu derived from SPT-N Values	4	281
Fat Clay	Theoretical NC Cu	4	295
Lean Clay	Cu derived from SPT-N Values	2	125
Lean Clay	Theoretical NC Cu	2	156
Lean Clay	Cu derived from SPT-N Values	4	312
Lean Clay	Theoretical NC Cu	4	249

6.3.2.4 Predicted increase in shear strength

A theoretical over-consolidated c_u profile resulting from the construction of a 2m and 4m embankment has been estimated from the addition of an increase in vertical stress with depth due to a strip area (the embankment) carrying a uniform pressure to equation [1] and is shown by equation [4] and Figure 6.3.

$$C_u = 0.11 \times (\sigma'_v + \sigma_z) + 0.0037 \times I_p \times (\sigma'_v + \sigma_z) \qquad [4]$$

Where σ_z is the vertical stress in the vicinity of a strip area carrying a uniform pressure for a linear stress-strain relationship derived from elastic theory reported in Craig (2004) by equation [5]

$$\sigma_z = \frac{q}{\pi}\,(\alpha + \sin\alpha\,\cos(\alpha + 2\beta)) \quad [5]$$

Where q is the uniform pressure on a strip area, α is the internal angle of uniform pressure with depth and β is the external angle of uniform pressure with depth.

Figure 6.3 illustrates the expected strength gain resulting from the construction of a 2m and 4m high embankment on a foundation soil comprising typical Fat Clay. The strength improvement is a critical factor for ground improvement methods relying on consolidation, particularly where surcharging is used.

6.3.2.5 Ground improvement

To provide an indication of the performance of mass strength ground improvement methods on the theoretical model, vertical displacement calculations for the following scenarios have been modelled: -

- Ground improvement yields strength that is 2 times original soil strength
- Ground improvement yields strength that is 4 times original soil strength
- Ground improvement works are carried out to 6mBGL
- Ground improvement works are carried out to 10mBGL
- Original strength profile derived from SPT-N value correlation

Mass strength ground improvement would result from methods that increase the soil density (for example sand compaction piles) or through reinforcement / partial replacement (for example cement columns) – see Chapter 9. In these cases, the improvement in soil strength is dependent on the area improvement ratio, a_s, defined as the ratio of improved area to total area treated.

The maximum settlement for each of the above combinations has been analysed for a Fat Clay foundation soil and is presented in Table 6.3.

Figure 6.3: Improvement in undrained strength resulting from embankment construction on Fat Clay foundation soil

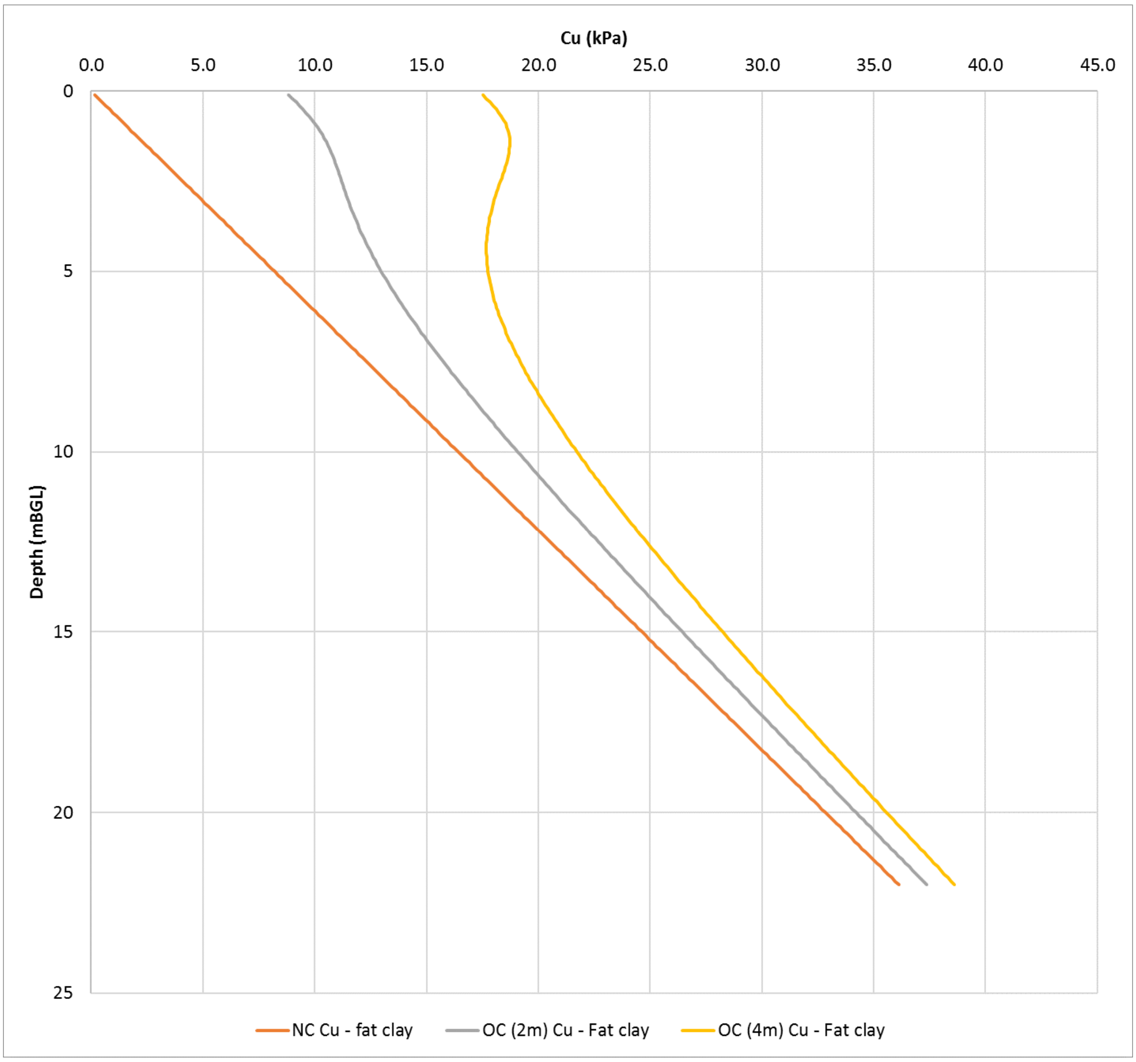


Table 6.3: Predicted reduction in settlement resulting from mass strength improvement of standardised ground model

Embankment Height (m)	Max. Settlement (mm)	Strength /Stiffness Improvement factor	Improvement depth (mBGL)	Estimated Max. settlement after ground improvement works (mm)
2	140	x2	10	85
2	140	x4	10	57
4	281	x2	10	169
4	281	x4	10	114
2	140	x2	6	103
2	140	x4	6	85
4	281	x2	6	207
4	281	x4	6	170

6.3.3 Stability

The calculation of ultimate bearing capacity can be undertaken by the widely-adopted methods introduced by Terzaghi and developed by others since, for example Brinch Hansen – see Tomlinson (1995) or many other standard texts for details.

The ultimate bearing capacity of a foundation soil is calculated from an equation that incorporates the geotechnical properties of the soil and geometry of the applied load. The ultimate bearing capacity is presented as a three-term expression incorporating the bearing capacity factors: N_c, N_q and $N\gamma$, which are related to the angle of friction (ϕ') of the foundation soil.

$$q_u = \underline{c}.N_c + \underline{q_o}.N_q + \tfrac{1}{2}\underline{\gamma}.B \, . \, N\gamma$$

where: -

c = cohesion
$q_o = \gamma$. D (i.e. unit weight x depth)
D = founding depth
B = breadth of foundation
γ = unit weight of the soil removed

For drained loading, calculations are in terms of effective stresses with $\phi' > 0$ and N_c, N_q and $N\gamma$ are all > 0. When considering undrained conditions, the calculations are in terms of total stress and the term c, is the undrained shear strength, cu, and terms $N_q = 1.0$ and $N\gamma = 0$. The resulting equation is:

$$q_u = c_u \, .N_c + q_o$$

where: -

N_c = *5.14 for* $\phi' = 0$

The simplified equation is often adopted where the undrained loading condition is considered more critical than the drained case, for example for construction of embankments on soft clay. A more rigorous approach includes assessment of both the total and effective stress to ensure both stability cases are addressed and the least conservative adopted. This more rigorous approach was not possible within the constraints of the project.

Using the undrained shear strength profile developed in Section 6.3 and illustrated on Figure 6.1 and 6.2, it can be shown that the ultimate bearing capacity, for an undrained shear strength of $8kN/m^2$ = 8 x 5.14 = $\sim 40kN/m^2$. For a new embankment construction that is 2m in height, the applied load if a unit weight of $19kN/m^3$ is used, is $38kN/m^2$. This indicates that the foundation soil is at or very close to failure.

Even if the foundation does not fail, the degree of deformation or settlement will be high an unlikely to be acceptable in terms of serviceability. For foundations, it is usual to apply a factor to the ultimate bearing capacity, typically 3, to provide an allowable bearing pressure that will limit deformation to acceptable levels, giving very low values $>15kN/m^2$. This drives the need for improved support, through either ground improvement or use of piles to transmit the load to depth.

6.4 Factors not related to soft foundation soil

The factors relating to soft foundation soil conditions are considered further in this report in Chapters 8 to 10. The factors that are not related to foundation soil condition or ground improvement are a significant contributory factor in the observational deformation models developed and should not be overlooked. These are: -

- Inadequate pavement design and/or construction & maintenance.

- The asphalt road surface sits atop a herringbone brick layer. These bricks push down easily into the un-compacted embankment fill formation under vehicle load. The movement of the numerous bricks under load results in often terrible reflective cracking in the asphalt layer.
- The bricks are often of low strength and are susceptible to cracking and breaking under load which further exacerbates the reflective cracking problem.
- The Lean Clay and Fat Clay of which the embankments are predominantly comprised, have an intermediate to high volumetric change potential, and so can swell and contract with the regular variations in water level, thus affecting the road pavement.
- Large and often overloaded vehicles are using the road and their weight is deforming the pavement layers causing damage.
- Vehicles regularly drive along the edges of the road when passing each other, and the action of the wheels moving on and off the edge of the pavement layer is causing deep rutting and damage to the road edge which progressively works inwards towards the centre of the road.

Issues with the embankment side slopes and side slope stability are also due to several factors:

- No or inadequate earthworks compaction
- Inadequate design and/or construction of embankment slopes and reinforcement e.g. toe wall.
- Wave action is eroding the side slopes.
- Human modification of side slopes to increase aqua-culture working area.
- Regular variations in water level from aqua culture practice.
- Trees lean outwards from the embankment slopes and as they grow and increase in mass, lean further with potential to eventually uproot and damage the slopes.

7 Life-cycle considerations for constructing rural roads on soft foundation soils

7.1 Asset life-cycle

The overall project aim is to '...reduce construction and life-cycle costs of rural roads in the Khulna region through the provision of appropriate solutions for construction over soft compressible clays'. The principal focus is on the identification and recommendation of suitable ground improvement techniques to address this construction issue. However, the life-cycle cost for rural road infrastructure has a range of influences that contribute to potential damage or distress that may occur as illustrated in Figure 7.1 and Figure 7.2.

In this Chapter, the positive influences that can be made through the project phases are illustrated in brief. It is recognised that it is not the focus of this research project to fully address the entire range of influences upon the life expectancy of Khulna's rural road assets, but it is important to highlight that the effectiveness of ground improvement installation is vastly improved, and often conditional, on the maintenance regimes and full lifecycle approach to management of assets throughout their operational life.

Figure 7.1: Stages within the asset lifecycle

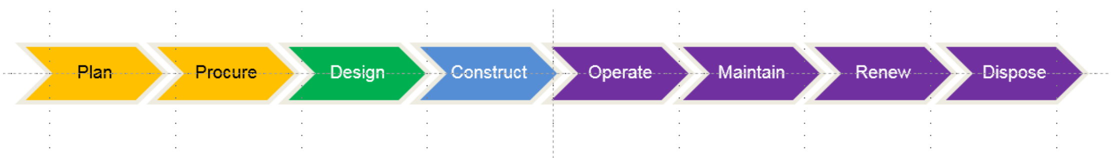
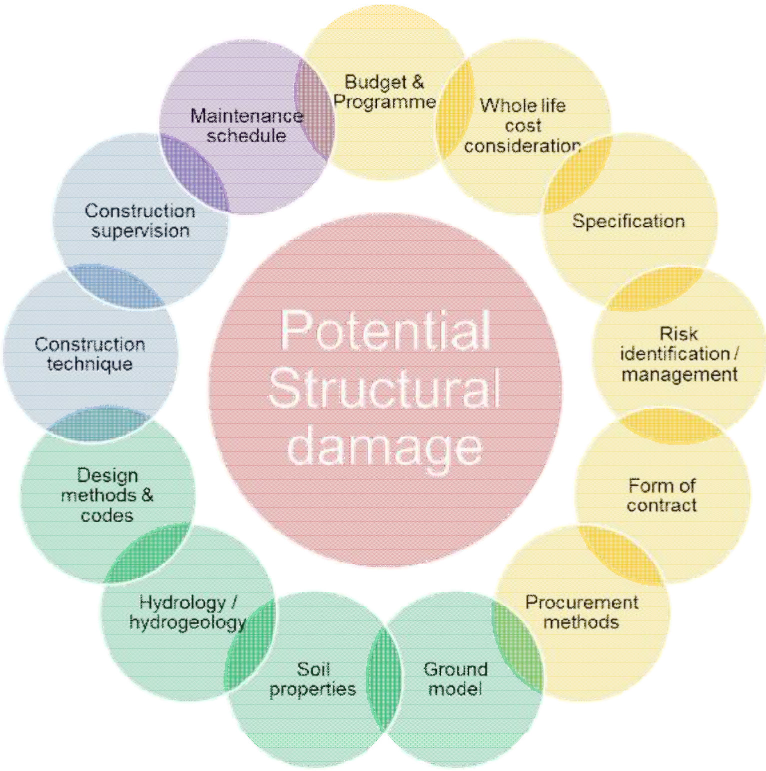


Figure 7.2: Factors that influence the potential for damage to structures and earthworks through the asset lifecycle



7.2 Planning and Procurement

A considered approach to the asset lifecycle, at the earliest stages of project planning and procurement, is considered essential to meeting the long-term aspirations that the asset owner has, and in particular minimising the costs and maximising useful service life.

Implicit in this is the planning of budget and programmes, in full consideration of the additional risk and cost, both at construction and operations phase, associated with specific local risks – in this instance, construction on soft ground.

The significant investment of capital made during design and construction phase, may be wasted if consideration is not paid to appropriate maintenance, for which the cost is budgeted and effectively implemented throughout the lifecycle of the infrastructure asset.

The establishment of a project geotechnical risk register (see example in Appendix H) is beneficial to understanding the geotechnical related hazards and impacts, such that informed decisions relating to risk mitigation can be made through the whole asset lifecycle. For instance:

- Where ground conditions are poorly understood, the risk register should be used to influence the nature and quantity of ground investigation and laboratory testing;
- Where no ground investigation is planned or likely to be available, the impacts and consequences must be understood, and mitigation measures, such as a more robust design, or increased long term maintenance will be understood from the outset;

The risk register provides a valuable focus for decision making throughout the life cycle of the project, and should be updated and reviewed on a regular basis to ensure the risks are being managed effectively, and in line with asset owner expectations.

One illustration of a common pitfall in planning and procurement for soft ground is where contracts will simply specify tighter movement tolerances. This is not always the best way to manage the risk of soft ground, or realistic and achievable within the budgets and programmes available for construction, leading to higher costs for no improvement in lifecycle performance. Instead, setting sensible tolerances whilst understanding and planning for maintenance requirements to manage serviceability levels in the long term can be a more pragmatic and achievable approach.

Similarly, in the procurement phase - setting challenging contractual and programme requirements will almost certainly not lead to a successful outcome on a project where the ground conditions are probably poorly understood. Working together to meet the challenges and being flexible in how difficult situations can be overcome may be more expensive in the design and construction phase, but will lead to significant benefits during operations and maintenance, most notably in reducing the long-term liability of maintenance and renewal that is likely to prove more expensive over a shorter life for the asset.

Note, procurement can relate to design and/or construction services depending on how a project is delivered.

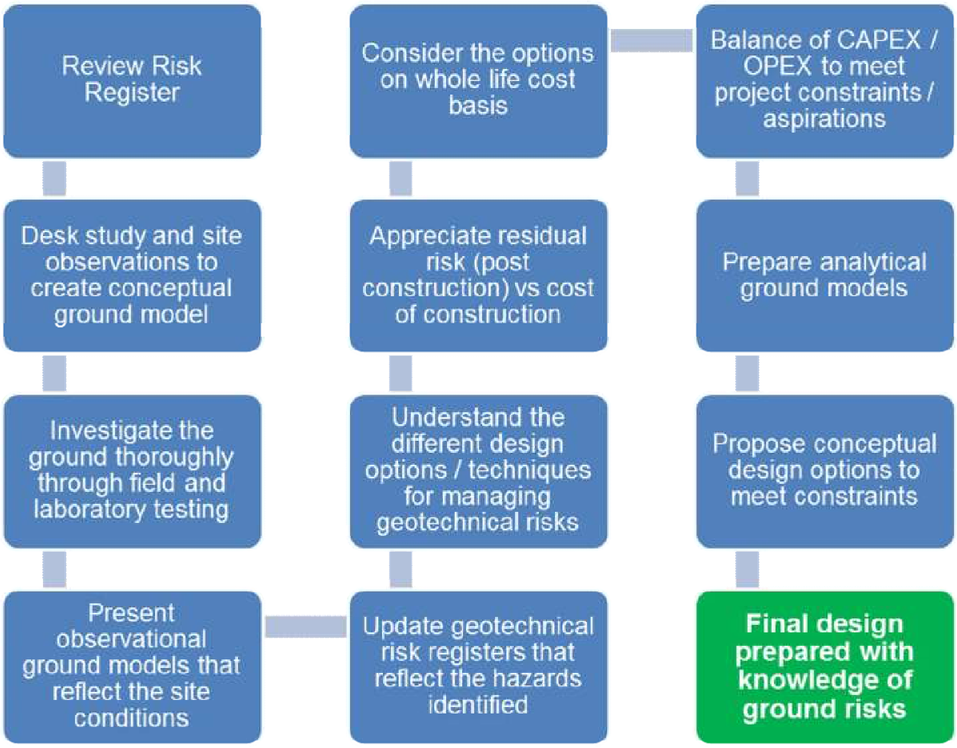
7.3 Design Phase

Once an adequately planned and procured project moves into the design phase, the particular characteristics of the site need to be understood to a level that manages the project risks in line with asset owner expectations. For example, for sites in the Khulna region where consideration of ground conditions might have a high impact on the construction costs and asset lifecycle:

- Where field investigations and analyses show that the impact of overall ground movements is only slight, there is little point in developing a sophisticated ground model and analysis;
- Where there are highly prescriptive specifications associated with movement tolerance on soft ground, basic trial pits and basic laboratory testing to support the design process, will not adequately manage the associated ground risks, with resultant outcomes of serviceability issues or asset failure. In this instance, a more sophisticated approach would be required

With this in mind, Figure 7.3 demonstrates a design process that adequately manages the risk associated with soft ground. This is also included as a flow chart in Figure 10.3 in Chapter 10.

Figure 7.3: Design Phase – Management of Risks associated with Soft Ground



7.3.1 Desktop Studies

For all projects, and particularly ones where ground conditions are envisaged to be problematic, investing in the early stages of a project, through literature review, desk study, observational site visit and ground investigation is strongly recommended. These activities represent a small part of the project cost, but contribute significantly to long term risk mitigation that is required to predict, prevent or manage future ground and structural movement.

7.3.2 Ground Investigation and Laboratory Testing

Developing a good observational ground model is dependent on the quality of the ground investigation contractor, their plant, sampling methods and transportation together with the laboratory testing. The quality of the ground information received from a borehole log or from the laboratory is in itself a risk that needs to be managed appropriately. Greater investment in high quality ground investigation services is required for construction projects or techniques that are more complex or high profile. It is recognised that sophisticated ground investigation is not routine for rural road projects, but the long-term project outcomes will improve if this stage can be completed.

7.3.3 Methodologies for managing Soft Ground

There are many available design and construction methods that could be suited to managing risks of soft ground, for which the following factors need to be taken into consideration:

- Ground Improvement: Cost variance between different ground improvement options can be high, for example pre-loading vs installation of sand compaction piles;
- Programme: For short or time-critical programmes, the costs are likely to be higher, as cheaper options such as pre-loading may not be possible;
- Specifications (Tolerance): Where construction tolerance is small or been specified as very tight the costs will be higher, as more robust solutions are needed;
- Ground Investigation: a technique that requires a great deal of ground knowledge to support design, should not be selected where knowledge of the foundation sub-soil is lacking;
- Lifecycle Expenditure: Lower initial capital expenditure (CAPEX) solutions, usually have a proportionally higher operational expenditure (OPEX) to maintain long term serviceability. That is perfectly acceptable as long as these residual risks are understood, budgeted for, and implemented in operations and maintenance regimes.

In the soft soil environment, for organic rich soils, where there is often an active process of secondary compression (caused by creep settlement), well beyond the primary soil consolidation phase and sometimes throughout the life of a structure, a prior knowledge is powerful – meaning that if the structure is going to move, this can be planned for, throughout design, construction and maintenance.

It is through an understanding of the ground risks, the mitigation potential and relative advantages/disadvantages of various techniques, and the design optioneering process itself, that the final design can be prepared with a degree of confidence. The design phase is where the principal decisions are made that will manage risk through the asset life, and the success of a project in these terms is often entirely dependent on how thoroughly that phase has been conducted. The overall influences that influence selection of ground improvement methods at design stage are typically: -

- Importance of the road to transport network;
- Sensitivity of the structure to deformation e.g. earthwork vs structure
- Geology at the site
- Certainty of the ground model
- Applicability of the method to site conditions and constraints
- Availability of plant and supervision

7.4 Construction Phase

During the construction phase of a project, there are further considerations and challenges associated with construction of roads and structures on soft ground, listed below: -

- Poor working conditions in the formation
- Specialist techniques / materials /plant and sub-contractors
- Longer construction programmes – more complex and / or staged construction
- Potentially greater risk of claims / delays
- Potential for Unexpected ground conditions / site changes
- Greater supervision and quality control required

7.4.1 Construction Supervision

A high standard of appropriately resourced construction supervision and quality control at construction stage is important to ensure that measures to mitigate risks from soft foundation soil at procurement and design stage are not lost for lack of poor site practices and training.

7.4.2 Trial Construction

An effective method to deal with a number of these issues, and common international practice, is to perform trial construction in similar ground conditions and using the proposed technique (or suite of techniques for comparison). The results can then be employed for the construction project in question, but also for future projects to use as a basis for technique selection and construction methodology.

7.5 Operate, Maintain and Renew Phase

Finally, considerations for the operation and maintenance phase, that often attracts the greatest cost, and as emphasised throughout the above planning, design and construction for mitigation of risks associated with soft ground, are summarised below: -

- Cost depends on how previous phases have been executed
- Will depend on the split of capital and operational expenditure (how whole life costing (WLC) has been considered).
- If previous phases are done poorly, maintenance will be frequent and costly and the life expectancy of the infrastructure asset will be short
- If maintenance is planned and part of the WLC investment then this should be accepted part of the scheme
- Sufficient funds to maintain the asset are available, and this is particularly the case for assets built on soft ground.

Long term creep settlement of organic soft soils that are present in the sub-surface soil horizons of Khulna Region (see Chapter 3), present implications for operations and maintenance of rural road assets in the region, and for which mitigation measures must be clearly identified and incorporated into budgets, planning and implementation of maintenance activities.

8 Current Guidance for Design & Construction of Rural Roads

8.1 General

The Government of the People's Republic of Bangladesh provide a number of specifications and guidance documents for rural road applications as follows: -

1. Local Government Engineering Department (1999). Road Pavement Design Manual
2. Planning Commission (2004), Road Design Standards, Standard Designs and Costings for Zila, Upazila and Union Roads, Bridges and Culverts (2004)
3. Local Government Engineering Department and Japan International Cooperation Agency (2005), Road Design Standards, Standard Designs and Costings for Zila, Upazila and Union Roads, Bridges and Culverts
4. Local Government Engineering Department (2004). Technical Specification for Bridges on the Upazila and Union Roads.
5. Local Government Engineering Department (1999). Standard Specifications for Feeder Road Type-B & Rural Road Type-R1 under LGED.

The documents provide design guidance for the construction of rural road embankments and structures, including material specification, slope angles and includes numerous standard details for use in construction scenarios. With the exception of guidance on preparation of foundation soil in (3) and (5) e.g. grubbing and compaction, there is no specific guidance provided on treatment methods for use when the foundation soil strength is low. Also, no design criteria for settlement / differential settlement was noted.

Due to the flat topography in most of Bangladesh, the abundance of water courses and regular flooding, rural roads are generally built on embankments. The geomorphology dictates that the embankments are low, and most observed are 1 to 2m high with some earthworks up to 5m where adjacent to bridge structures e.g. bridge approach embankments. The design guidance concentrates mainly on the new build of such earthworks but also provides direction where dealing with remedial treatments and earthworks widening.

The following Table 8.1 presents a summary of the key geotechnical related points provided in documents (1) to (4), categorised as follows: -

- General
- Ground Investigation
- Sub-grade assessment
- Earthwork Materials
- Construction
- Slope angle
- Slope protection
- Drainage
- Widening

Geotechnical related elements in document (5) are largely the same as in (3); comment on document (5) is not given in the table below to avoid duplication.

Table 8.1: Summary of key geotechnical points in LGED Guidance and Specifications

Category	(1) Local Government Engineering Department (1999)	(2) Planning Commission (2004)	(3) Local Government Engineering Department and Japan International Cooperation Agency (2005)	(4) Local Government Engineering Department (2004)
General	Settlement should always be expected, especially on approaches to bridges where embankments are higher. .	Four categories of road project with definitions. (i) Reconstruction, (ii) New Road Construction, (iii) Widening and (iv) Strengthening. Typical costs (at 2004) provided for road categories above + slope protection works. Six design standards for pavements detailed for different traffic and axle load scenarios – **not dependent** on road classification e.g. Zila, Upazila, Union and Village. Standard geometries for bridges included. Typical plan & elevation (Fig 6) of bridge approaches and slopes for approaches provided (angles / protective measures)	It is noted that where embankments are to be built in swamps or water, that swampy ground will be displaced, typically by sand as directed by the Engineer. It is also stated that the original ground surface (the foundation soil) needs preparation and compaction to a specified limit (95%MDD)	This is a works specification document. Description of piling methods for constructing foundations are included.
Ground Investigation				Section 2.7 states that confirmatory sub-soil investigations shall be carried out at each bridge foundation. Testing and sampling (both disturbed and undisturbed) together with Standard Penetration Testing (SPT) 'shall' be carried out. Includes tests for strength and deformation of soils in addition to classification tests.
Sub-grade assessment	Recommended thicknesses for improved sub-grade layers (ISG) are given where lower CBR values are recorded (between 200 and 450mm). Overseas Design Note 31 (TRL) is included in Appendix C that gives comprehensive guidance on assessment of CBR for road purposes.	Recommended ISG are given in standard pavement details. Appendix D also gives ISG layer thicknesses depending on CBR of sub-grade. ISG is 450mm for CBR 2% and 200mm for CBR = 5%.	ISG layers are to have Plasticity Index <6%.	N/A

Category	(1) Local Government Engineering Department (1999)	(2) Planning Commission (2004)	(3) Local Government Engineering Department and Japan International Cooperation Agency (2005)	(4) Local Government Engineering Department (2004)
Earthwork Materials	Organic material must not be used and pure silt should be avoided.	Appendix D provides useful standard terminology for materials used in road construction	Fill materials to be free of deleterious materials. Liquid Limit less than 50% and Plasticity Index less than 20%. Soaked CBR of sub-grade level >3% at 95% Maximum Dry Density. Moisture constant at time of compaction ±5% of optimum.	Ordinary fill - Liquid Limit less than 50% and Plasticity Index less than 20%. Ordinary fill placed as backfill at time of compaction ±5% of optimum. Soaked CBR of sub-grade level >4% at 95% Maximum Dry
Construction	Local materials preferred to avoid long haulage distances and increased cost.		The distance of borrow pits from the toe of the embankment is given as between 1.5 and 3.0 x height from the toe. Embankment constructed in layers 150mm thick or less within limits of moisture content stated. Mixing with dry material is noted to achieve permissible moisture content limits. Compaction achieved with 'appropriate' mechanical plant in a longitudinal direction along the embankment starting at the outside and working towards centre.	'Unstable soil' encountered at bed level to be removed and replaced as directed. For footing not on piles, care to be taken not to disturb the base of the excavation. Poor foundation material is mentioned and ground strengthening alluded to, but not described how this would take place. Excavation dewatering is covered.
Slope angle	Selection procedure outlined. Stability dependent on type of soil, climatic conditions, drainage and foundation soil material. For different AASHTO soil types, typical slope angles are provided. For typical construction materials in A-4 to A-7 AASHTO category, slopes of 1:3 (v:h) are recommended where inundation is expected.	Not explicitly stated, although standard details for slope protection show 1:2 (v:h)	Not explicitly stated, although standard details for slope protection show 1:2 (v:h)	N/A
Slope protection	Various slope protection measures are listed but example use not given. Vegetation of slopes is common practice.	Appendix H provides various standard details for slope protection. These vary from simple turfing through to embedded palisade walls and brick toe walls. Palisade walls comprise piles and planks (see Plate UPR-UNR-EM1). Piles have 2m embedment and 1m upstand.	Appendix on road embankment protection provides various standard details for slope protection. The 11No. options presented on the Plates (UPR-UNR-EM1 to EM3) vary from simple turfing through to embedded palisade walls and brick toe walls. Same details as in Planning Commission (2004)	Section 6 provides details of bank protective works including; Turf, Rip-Rap, Brick Matressing, Boulder Matressing, Brick/Cement blocks, geo-jute geotextiles and toe walls

Category	(1) Local Government Engineering Department (1999)	(2) Planning Commission (2004)	(3) Local Government Engineering Department and Japan International Cooperation Agency (2005)	(4) Local Government Engineering Department (2004)
Drainage	Importance of drainage on performance of sub-grade and embankment fill is highlighted. Moisture content of sub-grade and embankment fill changes due to a number of causes. (i) Seepage of water into the subgrade (ii) Rise or fall in the level of water-table. (iii) Percolation of water into the subgrade through cracks in road surface. (iv) Transfer of moisture by capillary action from lower layer of soil. (v) Transfer of moisture from shoulder to the pavement edges. As most of the rural roads have low freeboard during rainy season, the water table rises and comes very close to the pavement. As a result, the stability of the subgrade is reduced. Typical drainage layouts to address this are provided, although it should be noted that drainage cannot prevent inundation from sever flood events experienced in Bangladesh.	The importance of cross-drainage is highlighted to mitigate waterlogging and flooding of embankments. Typical length of drainage 'gaps' per km of road are given.		
Widening	Benches should be formed in the existing earthwork to key in the new portion		Benches to be 300mm high and 600mm wide. Materials excavated from benches may be used subject to compliance with materials specification. Typical standard details for road widening of Upazila and Union Roads given in Plates UPR-EW-WD1 and UNR-ES-WD1. Plates show benching and overfilling at edges (to be trimmed on completion)	

9 Ground Improvement Techniques for Soft Soil in Khulna Region

9.1 Overview

The techniques that are described below and summarised in the accompanying Tables 9.1 to 9.6 are those that are considered either (a) technically feasible and (b) within the likely budget for rural road construction. From the literature review, and discussions with local stakeholders, the techniques are mostly in use in Bangladesh, although not necessarily for the rural road application. The ground improvement techniques considered here are: -

- Excavate and Replace / Displacement
- Sand Compaction Pile
- Sand Drain (with surcharge)
- Prefabricated Vertical Drains (with surcharge)
- Geotextile basal reinforcement
- Cement Columns

Cement columns and Geotextile basal reinforcement are included here as they represent significant benefits, although the use in Bangladesh is limited. Some research has been undertaken and this is discussed. Other potential techniques that have benefits but limited research, for example fibre reinforced soil (with jute or shredded plastic) are presented in Chapter 11.

It is notable that there are no clear design criteria for settlement / differential settlement for roads and structures within the current LGED design guidance for rural road construction

Ground improvement by displacement e.g. with sand/rock would commonly be used in the softest of soils or swamp conditions as indicated in LGED guidance (see Chapter 8). These methods are only briefly considered here as they are outside the scope of the research.

9.2 Sand Compaction Pile (SCP)

9.2.1 Methodology and typical use

Sand is fed into the ground through a casing pipe and compacted by means of a hammer or vibration. The SCP technique can be used effectively in loose sand and soft clay materials to affect an improvement in the stiffness and strength of the soil mass. For sands, the improvement in the ground is through densification of the mass to increase density, stability and deformation characteristics. For clays, the introduction of SCPs reinforces the soft foundation soil with columns of stiffer materials, improving overall strength and stability. For discrete thicknesses of soft foundation soil, with a competent layer or bedrock at relatively shallow depth, the SCPs are taken down to this depth and this is termed ‘fixed type improvement’ Kitazume (2005). Where competent ground is too deep or engineering plant cannot reach this depth, the technique is termed ‘floating-type improvement’. The former is preferred as settlement is less. Patterns for the SCP can be square, triangular or rectangular.

For earthworks, the introduction of SCP is most useful for reducing settlement and increasing the bearing capacity of the soil. For structures, SCP can be used at transitions e.g. approach embankments, to limit differential settlement.

A more extensive review of research and experience of the technique is included in the previously submitted Inception Report.

9.2.2 Design Guidance

Guidance on design methods is included in Kitazume (2005). Generally, the effectiveness of the technique increases as the replacement area ratio, a_s, increases. The replacement area is defined by the ratio of the replacement sand area and the overall area being treated. Clearly, the greater the sectional area of sand compared to the overall treatment area, the higher the ratio and greater influence the sand component has on ground characteristics. The soils strength, and resulting bearing capacity, is determined from a composite calculation using Terzaghi's theory (See Section 6.3), whereby the capacity of the clay portion and sand portion are separately calculated and then, using the replacement area ratio, a_s, the combined bearing capacity is given. It is reported that in Bangladesh, a sand volume that is 4 times the volume of the pipe casing is introduced, producing an SCP that is approximately twice the casing diameter.

The deformation characteristics and resulting settlement under load is also determined using the replacement area ratio, a_s, to determine a settlement reduction factor, ß. The settlement reduction factor is applied to the settlement value determined for unimproved soil through one-dimensional consolidation or elastic theory. The settlement factor reduces as the replacement area ratio increases.

Alamgir and Chowdhury (2004) reported on the improvement in strength gained from the installation of 200mm sand piles 8-9m in length, 0.75m spacing in a square arrangement to provide support to a river regulator structure. Pre- and post- construction SPT demonstrated an increase in N value, indicative of strength, of up to 3-4 times original. Mixed sand/stone (fine/medium gravel) columns can also be used, with reported greater strength.

An overall summary of the technique is provided in Table 9.1 below.

Table 9.1: Sand Compaction Pile

Ground Improvement Technique	Sand Compaction Pile (Hammer)
References used	Kitazume (2005), Alamgir and Chowdhury (2004), Harada et al (2015)
Purpose	• To increase strength and stiffness of the foundation soil leading to improved bearing capacity and reduction (not elimination) of total and differential settlement. • Improvement in properties comes from combination of: - o direct support by SCP, o replacing proportion of soft clay foundation soil with sand (greater proportion = better improvement), o the action of densification as the SCP is formed (although this is better in sandy foundation soils) and; o controlled consolidation and hence increase in stiffness of soil mass. • Where no competent horizon to provide end bearing, the SCP could be used to improve bearing capacity of the soil, although settlement would still occur.
Application	• To provide support to shallow bridge foundations and culverts • To provide support to bridge approach embankments (where bridge foundations are piled or supported on SCP) to limit impacts of differential settlement between earthworks and bridge structure • To improve bearing properties of foundation soil below general earthworks that could lead to reduced construction programme • To reduce total settlement of general earthworks to reduce overall maintenance requirements.

Ground Improvement Technique	Sand Compaction Pile (Hammer)
Design and Construction Considerations	• Current methods limit depth of construction to 10-12m, typically shorter. • In Khulna Region, this is typically installed through a 200-300mm diameter pipe to create a 400-600mm diameter SCP (4 times volume increase through ramming action) • Spacing typically 0.7-0.9m in a triangular or square pattern • Depth of soft soil – if >10-12m, current implementation methods will reduce effectiveness (floating vs fixed) • Geometry dictates improvements to strength and stiffness e.g. as spacing between SCP reduces, bearing capacity increases and settlement reduces proportional to the volume of foundation soil replaced by SCP. This has a cost implication. The more that is spent, the greater the improvement. Typically for clay foundation soils, the replacement ratio is 30% or greater of the original soil mass. • Methods for quality control of installation need to be specified. • Availability / source of sand – significant volumes required. • Properties of sand and foundation soil materials, together with geometry of SCP and soil profile is required for design • Straightforward design process using traditional soil mechanics theory subsequently factored to account for proportion of original soil replaced. Simple design charts could be developed.
Advantages	• Proven use in the Region, including labour and plant • Provides direct support from the SCP and densification of the surrounding ground • Permeable nature of SCP provides a drainage path to expedite consolidation settlement
Disadvantages	• Depth limitation may lead to reduced effectiveness in deeper soft soil horizons • Design requires knowledge of the foundation soil properties and geometry • Requires a supply of clean sand • Relatively more skilled labour required • Relatively more specialist plant required • Noisy (vibrating option reduces the hammering action but more specialised plant) • Where replacement ratio is higher, ground heave and waste material may result. • Quality control is difficult / requires supervision

9.3 Sand Drain with surcharge loading

9.3.1 Methodology and typical use

The sand drain (SD) is installed in the same manner as the SCP with sand fed into the ground through a casing, but the degree of compaction is significantly less. A sand volume of twice the pipe casing is reported from discussions in Bangladesh.

The method is used to expedite consolidation of the soft foundation soil, so that bearing capacity during construction can be increased and settlement during operation be reduced. The introduction of vertical drains reduces the length of the drainage path within the foundation soil, that might otherwise be upwards to the surface only. The rate of consolidation can be influenced by the length of the drainage pathways (more drains means shorter drainage paths), and the applied surcharge loading.

Dhar et al (2011) reports that for an equivalent pattern layout, SD of 200mm diameter, are more expensive than prefabricated vertical drains (PVDs), but are more effective in terms of rate of consolidation.

A more extensive review of research and experience of the technique is included in the previously submitted Inception Report.

9.3.1 Design Guidance

Guidance on design methods is included in Dhar et al (2011). The amount of settlement is determined from one dimensional consolidation theory (or elastic theory as presented in Chapter 6 of this report). The time for consolidation can be determined using the theory of consolidation by radial drainage presented by Hansbo (1960, 1979). Determination of time for consolidation requires the geometry and permeability of the vertical drain, and area to be drained (by each drainage element), the coefficient of horizontal consolidation and horizontal permeability (of the in situ soil), Ch and kh, and details of the zone of disturbance created during construction as a result of soil smear (geometry and permeability).

A surcharge loading of 1.2 times future effective stress was suggested by Seah et al (2011) to limit settlement in operation, but this must be balanced against initial bearing capacity, and may require staged construction up to the final surcharging load. As consolidation proceeds, the bearing capacity of the soil increases as demonstrated in Chapter 6.

An overall summary of the technique is provided in Table 9.2 below.

Table 9.2: Sand Drain (with surcharge loading)

Ground Improvement Technique	**Sand Drain (with surcharge loading)**
References used	Dhar et al (2011), Hansbo (1960, 1979)
Purpose	• To increase strength and stiffness of the foundation soil leading to improved bearing capacity / settlement properties through soil consolidation prior to • Improvement in properties comes from combination of: - o controlled consolidation using surcharge loading and hence increase in stiffness / strength of soil mass on completion of surcharging period.
Application	• To improve ground for bridge approach embankments (where bridge foundations are piled or supported on SCPs) to limit impacts of differential settlement between earthworks and bridge structure • To improve bearing properties of foundation soil below general earthworks • To reduce total settlement of general earthworks (following consolidation phase) to reduce overall maintenance requirements.
Design and Construction Considerations	• Current methods limit depth of construction to 10-12m • In Region, this is typically installed through a 200-300mm diameter pipe to create a 300-450mm diameter SD (2 times volume increase – lightly rammed) • Depth of soft soil – if >10-12m, current implementation methods will reduce effectiveness of SDs • Spacing typically 0.7-0.9m in a triangular or square pattern • Geometry dictates improvements to strength and stiffness e.g. as spacing between SD reduces, consolidation is more effective (drainage paths shorter) and volume reduction / strength gain is achieved more rapidly. • Requires sand blanket at embankment formation level • Methods for quality control of installation need to be specified. • Availability / source of sand – significant volumes required. • Properties of sand and foundation soil materials, together with geometry of SD and soil profile is required for design • Straightforward design process using traditional soil mechanics theory to determine final settlement and stiffness. Simple design charts could be developed.

Ground Improvement Technique	Sand Drain (with surcharge loading)
Advantages	• Proven use in the Region, including labour and plant • Provides improved strength and stiffness of the combined soil mass once consolidation has taken place
Disadvantages	• The improvement takes time to develop leading to increased construction programme • Improvement in soil properties not as great as for SCP method. • Depth limitation may lead to reduced effectiveness in deeper soft soil horizons • Can be ineffective in layered or laminated soft soil profiles due to smearing and blockage of horizontal in situ drainage paths • Design requires knowledge of the foundation soil properties and geometry to determine time for surcharging to design strength / stiffness. • Requires a supply of clean sand • Relatively more skilled labour required • Relatively more specialist plant required • Noisy

9.4 Prefabricated Vertical Drain (and Jute Drain) with surcharge loading

9.4.1 Methodology and typical use

Prefabricated vertical drains (PVDs) are installed with a mandrel pushed down into the soft soil. As for the SD, the method is used to expedite consolidation of the soft foundation soil, so that bearing capacity during construction can be increased and settlement during operation be reduced. The method has been widely used on construction projects around the world including Bangladesh, but would rarely be implemented on rural roads due to the requirement for more specialist equipment and materials than the SD technique. Notwithstanding this, the PVD technique is a viable solution and could be implemented over SDs where programme was shorter.

A more extensive review of research and experience of the technique is included in the previously submitted Inception Report.

9.4.2 Design Guidance

PVDs evolved from the SD technique, and the theory is much the same as for SD (see Section 9.3). Guidance on design methods is included in Dhar et al (2011) and Seah (2005). A Chapter within the book entitled Ground Improvement by Kirsch and Bell (2013) is dedicated to the design and construction of PVDs. A useful design and construction procedures flowchart is provided in this document (figure 4.6 of the document).

According to Seah, a surcharge loading of 1.2 times future effective stress is appropriate to limit settlement in operation. This could be higher, but this must be balanced against initial bearing capacity, and may require staged construction up to the final surcharging load. As consolidation proceeds, the bearing capacity of the soil increases as demonstrated in Chapter 6. Dhar reports that a 450mm thick drainage blanket was used at Chittagong port, between the PVD and surcharge fill. For the Suvarnabhumi Airport in Bangkok, Seah stated that the sand drainage blanket was constructed with filter fabrics to avoid contamination of sand during consolidation. Seah stated that the surcharge was removed when degree of consolidation had reached 80% of final estimated amount and settlement ratio was 3-4% (rate of change from past month to current month).

The principal reason for including PVD as an option is the potential for using Prefabricated Vertical Jute Drain (PVJD). The PVJD has been researched by Khan (2009) – PVJD fabricated with similar properties to standard PVD in a laboratory test scenario. Some measured properties are provided and this is a significant avenue of research to avoid the need for expensive materials.

An overall summary of the technique is provided in Table 9.3 below.

Table 9.3: Prefabricated Vertical Drain (with surcharge loading)

Ground Improvement Technique	**Prefabricated Vertical Drain (with surcharge loading)**
References used	Dhar et al (2011), Seah (2005), Kirsch and Bell (2013), Khan (2009)
Purpose	• As for Sand Drain
Application	• As for Sand Drain
Design and Construction Considerations	• Geometry dictates improvements to strength and stiffness e.g. as spacing between PVD reduces, consolidation is more effective (drainage paths shorter) and volume reduction / strength gain is achieved more rapidly. • Availability of PVD or Jute alternative product. • Properties of PVD and foundation soil materials, together with geometry of soil profile is required for design • Requires sand blanket at embankment formation level • Straightforward design process using traditional soil mechanics theory to determine final settlement and stiffness. Simple design charts could be developed.
Advantages	• Proven use in the Region, including labour and plant • Provides improved strength and stiffness of the combined soil mass once consolidation has taken place • Quicker to install than SD • Quality control easier than for SD
Disadvantages	• Moderately expensive when compared to other options considered • Requires a supply of imported PVD or Jute alternative • The improvement takes time to develop leading to increased construction programme • Improvement in soil properties not as great as for SCP method. • Design requires knowledge of the foundation soil properties and geometry to determine time for surcharging to design strength / stiffness. • Can be ineffective in layered or laminated soft soil profiles due to smearing and blockage of horizontal in situ drainage paths • Jute alternative would reduce cost but technique requires more testing / research. • Relatively more skilled labour required • Relatively more specialist plant required

9.5 Cement Columns

9.5.1 Methodology and typical use

Soil mixing, with the addition of a binder, is widely used internationally and is well documented by Kirsch and Bell (2013). The method involves introducing a mixing tool into the soft soil whilst applying a quantity of cement or lime (or other admixtures such as slag and fly-ash have been considered with appropriate testing). The resulting column supports vertical loads through reinforcement of the foundation soil, transferring the load through the improved mass to more competent ground at depth, similar to SCP. The advantage of the CC is that the import of material and waste arisings is much less than for SCP or piling, but the plant and skill levels required are more specialised.

This is the least used and most specialised in Bangladesh – plant may need to be modified to be able to perform this method. Laboratory based research has been performed by Ahsan et al (2014) at Khulna University of Engineering & Technology on the basis that this technique could be used for soft soil improvement in Khulna Region. A laboratory scale trial was performed and this showed significant increase in deformation and stability characteristics of the host soil.

A more extensive review of research and experience of the technique is included in the previously submitted Inception Report.

9.5.2 Design Guidance

Design guidance is provided in Kirsh and Bell (2013). The following points are worth noting: -

- For foundation soil with undrained shear strength <8kPa – install CC in panels or grids to support each other – spacing 0.75-0.85 times column diameter to form a secant wall.
- For embankments, rows to be installed perpendicular to failure plane, forming a barrette or panel with increased shear strength.
- Where the foundation soil strength is >8kPa – individual columns can be installed at 1.3 to 3 times diameter of column
- Columns diameters are typically 0.5 to 1.0m
- Depth depends on soil strength and plant capacity but could be between 3 and 25m depth.
- Strength gain occurs rapidly with cement, slower with lime (10 times in first 5-10 days with cement, up to 20 times original soil strength in long term. Long term strength with lime is similar.
- The ratio of area improvement (area of soil mixing / total area) is proportional to the overall improvement is strength and deformation characteristics. It is reported that improvement area ratios of between 10 and 30% are used in Scandinavia to address settlement. Higher ratios of 30-50% are used in Japan due to seismicity.
- Where stability is the principal concern, improvement ratios are typically higher than for deformation problems.

Seah (2005) indicates that mass improvement calculated based on CC strength and column spacing/diameter with a ratio of 2.5, can achieve foundation soil strength increase of 5-6 times the original mass strength.

Further research to supplement the work by Khulna University is required, together with field trials.

An overall summary of the technique is provided in Table 9.4 below.

Table 9.4: Cement columns

Ground Improvement Technique	Cement Columns
References used	Ashan et al (2014), Seah (2005), Kirsh and Bell (2013)
Purpose	• To provide rigid inclusions within the soft foundation soil that transfer the load to a more competent horizon. Similar to a pile but does not have resistance to horizontal loading or shear forces developed by for example, bridge loading. For earthworks, requires a load transfer platform (LTP) to spread the load between CCs. • Where no competent horizon to provide end bearing, the CCs could be used to improve bearing capacity of the soil, although settlement would still occur.
Application	• To provide support to shallow bridge foundations and culverts • To provide support to bridge approach embankments (where bridge foundations are piled or supported on CC) to limit impacts of differential settlement between earthworks and bridge structure • To improve bearing properties of foundation soil below general earthworks that could lead to reduced construction programme • To reduce total settlement of general earthworks to reduce overall maintenance requirements.
Design and Construction Considerations	• Geometry of CCs and LTP dictates load carrying capacity of improved ground • Properties of foundation soil materials, together with geometry of soil profile including competent layer at depth is required for design. • Cement proportion dictates strength and spacing of CCs – existing research can be used – see Ashan et al (2014) • Straightforward design process using traditional pile design theory to determine spacing and depth. Simple design charts could be developed.
Advantages	• Provides direct support from the CC • Limited or no waste products when compared to SCP as soil is mixed with cement to create the support.
Disadvantages	• Limited use in the Region • Limited by depth of soft soil • Limited by depth capacity of mixing plant (auger) • Design requires knowledge of the foundation soil properties and depth to bearing stratum. • Significant quantities of cement required • Quality control difficult for this in situ mixing technique • Relatively more skilled labour required • Relatively more specialist mixing plant required

9.6 Geotextile / geogrid basal reinforcement

9.6.1 Methodology and typical use

The use of geotextiles in pavement construction is widely documented, providing tensile capacity within the pavement layers, that can increase life and reduce thickness of pavement construction.

Khan et al (2014) describes the application of jute geotextiles for rural road pavement construction. This methodology is proposed for sites where the sub-grade strength was <3%, and jute is used as reinforcement, sandwiched between improved sub-grade (ISG) layers comprising sand. Khan reports that the increase in CBR with time ranges from 1.5 to 7 times the original value, with strength developed from tensile resistance of jute and through consolidation of the sub-grade.

The same principles are applied for basal reinforcement of embankments, where the tensile capacity from the geotextile resists the shear stress developed as the embankment is constructed that leads to bearing failure in the foundation soil. Thus, the use of basal reinforcement increases the allowable height of the embankment that can be supported by the foundation soil.

A more extensive review of research and experience of the technique is included in the previously submitted Inception Report.

9.6.2 Design Guidance

Design guidance is provided in BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills and in CIRIA Special Publication SP123 Soil Reinforcement with Geotextiles (1996). The geotextile layer (or layers) are formed within a sand or granular blanket as for pavement construction, with the addition of anchor blocks at each side. At Kakinda Port in India, Bhagwan et al (date unknown) used woven jute geotextile as basal reinforcement for a number of highway embankments constructed on soft foundation soil. Both Khan and Bhagwan note the short life expectancy of jute, but conclude that the purpose of the geotextile reinforcement, is to reinforce the sub-grade or foundation soil during construction and allow the soil strength to develop through consolidation. Bhagwan indicates a jute life of about 2 years, and the foundation soil strength had developed in this time to make the jute geotextile unnecessary for stability on the Kakinda project. Both authors provide material properties of the jute geotextile that can be used in the BS8006 design methods, including tensile strength and strain.

Bhagwan reported that the anchorage block at either side of embankment comprised a sand filled trench (0.5m square). A 0.3m thick sand cushion was used for protection of the jute whilst filling embankment and additional drainage.

Deformation of the foundation soil still results from the application of load, and total settlement is unaffected. The rate of consolidation may be increased using basal reinforcement as greater surcharge load can be applied, and the rate can be increased further when used in combination with either SD or PVD described earlier in this Chapter. Differential movement of the earthwork, together with deformation resulting from poor construction or materials in the embankment fill may be reduced when using basal reinforcement.

An overall summary of the technique is provided in Table 9.5 below.

Table 9.5: Geotextile / geogrid basal reinforcement

Ground Improvement Technique	**Geotextile / geogrid basal reinforcement**
References used	Bhagwan et al (unknown date), Khan et al (2014),
Purpose	• The geotextile provides shear resistance at foundation soil formation level to mitigate potential for bearing capacity failure or slope instability resulting from the soft soil under embankment loading. • The geotextile application does not limit total settlement but can contribute to reducing differential settlement. • The loading from the earthwork consolidates the underlying soils but at a relatively slower rate when compared to surcharging in combination with SD or PVD.
Application	• For general low height earthwork applications, to improve resistance to slope and bearing capacity failures. • Suitable where road is unsurfaced • Suitable where control of settlement is less important

Ground Improvement Technique	Geotextile / geogrid basal reinforcement
Design and Construction Considerations	• Single or multiple layers applied with within a sand blanket (ISG) • Shear strength and creep resistance properties of geotextile determine geometry and number of layers. • Methods for quality control of materials and construction relatively straightforward. • Straightforward design process using traditional soil mechanics theory to determine bearing capacity (and hence permitted height of embankment during construction) and total settlement. Simple design charts could be developed.
Advantages	• Proven use in the Region for pavement applications and some use for earthworks. Some research sites available and results can be used to validate methods. • Readily available materials (if Jute / bamboo used) • Labour intensive method increases employment (as compared to mechanised more technical solutions) • Methods for quality control of materials and construction relatively straightforward.
Disadvantages	• Design life of geotextile natural products is relatively short; long term improvement relies solely on consolidation of foundation soil as result of load applied by earthwork construction. • Total settlement is not reduced • Not suitable for structures • Not suitable for surfaced roads as cracking and deformation will be evident (unless surcharged before pavement layers completed)

9.7 Excavate and Replace / Displacement

9.7.1 Methodology and typical use

Where soft ground exists above a layer of more competent ground, depending on the depth of the soft ground, it is feasible to excavate out or displace the weak soil and replace it with a stronger material comprising granular soil e.g. boulders / cobbles / gravel and sand. The solution aims to improves both the bearing capacity of the ground and the immediate and long-term settlement. Both techniques are more suited to where a competent horizon exists at relatively shallow (<5m) depth. Given the typical Khulna region ground profile, these techniques may be less effective than others, particularly regarding settlement reduction.

CIRIA C573 (2002): A Guide to Ground Treatment provides further details of typical usage, but the following Sections provide a brief summary.

9.7.2 Excavate and Replace

The soft material is excavated (often in bays) and replaced with granular material. Access is provided from a previously improved area, and the operation progresses along the length of the infrastructure, excavating and replacing the material until the end point is reached. The technique involves significant import of structural fill and export/disposal of waste material, that may be very soft or even comprise a soil slurry. The technique can be used below ground water table, and this requires coarser granular material (boulders / cobbles that can largely self-compact) to be placed first, until a platform can be established approximately 0.5m above standing water.

9.7.3 Displacement

The soft material is physically displaced by the action of depositing the higher strength material on the ground surface. The soft ground is continually forced to fail by exceeding the bearing capacity / shear strength, plastically deforming the soil. The displaced material moves laterally away from the improvement area in soil 'waves' and the deformation associated with the waves can be significant and extend for some distance beyond the works (as much as 10 times the thickness of the soft horizon). The proximity of adjacent structures, and susceptibility to ground movement, should be carefully considered if using this solution.

9.7.4 Design Guidance

The design of these methods is limited to ensuring that the ground conditions and depths to be removed / displaced are achievable, excavation slopes are stable and that a source of higher strength material is available and of appropriate specification.

For excavate and replace, the maximum depth that this method can be used is dependent on the following factors: -

- Excavator reach; the length of the arm on the excavator will dictate the maximum depth that can be excavated (unless a multistage excavation is used – high unlikely for rural road project). A depth of 4-6m is typically the maximum depth that can be achieved.
- The stability of the excavation; with soft ground, particularly where high groundwater or surface water is present, the stability of an open excavation will significantly limit the depth and effectiveness of this method due to sidewall collapse, making the process difficult to achieve a satisfactory and measurable improvement in the strength of the ground.

For displacement technique, CIRIA C573 suggests an economical depth of 5m but potentially up to 15m. Significant volumes of material are used to facilitate the displacement and the overall effectiveness (in respect of reducing compressibility) can be difficult to assess, particularly where patches of soft soil are likely to trapped below the higher strength soils.

The costs and practicalities of undertaking both of these methods must be carefully considered against the techniques where the ground remains in place and the strength increased by the consolidation phase. The desired outcomes must be realistic also. Where soft soil exists to significant depth, these methods can improve the bearing capacity, and to some extent reduce surface settlement, but consolidation of the soil below the improved horizon will still occur.

An overall summary of the techniques is provided in Table 9.6 below.

Table 9.6: Excavate and Replace / Displacement

Ground Improvement Technique	**Excavate and Replace / Displacement**
References used	CIRIA C573: A Guide to Ground Treatment
Purpose	• To improve the bearing capacity and reduce surface settlement by replacing / displacing weak soil with a stronger material
Application	• Most suitable where competent horizon is present at relatively shallow depth (<5m) • Can be used for many purposes if the complete soft soil horizon is replaced. • If only partial replacement is possible, still applicable: - o for general low height earthwork applications, to improve resistance to slope and bearing capacity failures. o where control of settlement is less important o where road is unsurfaced

Ground Improvement Technique	Excavate and Replace / Displacement
Design and Construction Considerations	• Limited design required – trials and observational method during construction used. • Methods for quality control of materials relatively straightforward. • Quality control of the effectiveness / completeness of the replacement process is difficult
Advantages	• Relatively simple technique, with standard plant. • Where complete soft soil horizon is removed, the post construction consolidation is limited, reducing the impact on construction programme for above ground infrastructure.
Disadvantages	• Generally not suitable for structures (where soft soil >5m) • Large volumes of imported structural fill • Large volumes of exported waste material (for excavate and replace) • Impact on adjacent structures can be significant for displacement technique

9.8 Ground Improvement methods – settlement and bearing capacity

The methods described above have varying influence on the amount and rate of ground deformation and soil stability, or bearing capacity. The following Table 9.7 provides a summary of this information.

Table 9.7: Influence on settlement and bearing capacity

Ground response	Sand Compaction Pile	Sand Drain with surcharge	PVD with surcharge	Cement Column	Geotextile basal reinforcement	Excavate and Replace / Displacement
Initial Settlement	Low if SCP bears on competent strata	Low if bearing capacity failures is avoided	Low if bearing capacity failures is avoided	Low as load is transferred to competent strata at depth	Low if bearing capacity failures is avoided	Low if soft soil is completely replaced / displaced, and competent strata exists below
Consolidation Settlement	Low if SCP bears on competent strata. Where SCP toe is floating, consolidation will be greater	High but occurs relatively rapidly – the method works by encouraging consolidation	High but occurs relatively rapidly – the method works by encouraging consolidation. Rate of consolidation potentially greater than Sand Drain for same spacing	Low as load is transferred to competent strata at depth	High but takes considerable time to develop, depending on soil characteristics and geometry.	Low if soft soil is completely replaced / displaced, and competent strata exists below. Consolidation will be greater if soft soil remains in place below improvement zone.
Residual Settlement*	Low if SCP bears on competent strata	Low to medium – depends on effectiveness of surcharging during consolidation phase	Low to medium – depends on effectiveness of surcharging during consolidation phase	Low as load is transferred to competent strata at depth	Where no surcharging used, residual settlement will continue until reaches equilibrium (unless peat soils present, where creep settlement will continue)	Low if soft soil is completely replaced / displaced, and competent strata exists below

Ground response	Sand Compaction Pile	Sand Drain with surcharge	PVD with surcharge	Cement Column	Geotextile basal reinforcement	Excavate and Replace / Displacement
Bearing Capacity	Medium to high depending on sand replacement ratio	Increase in bearing capacity due to increase in soil strength during consolidation phase	Increase in bearing capacity due to increase in soil strength during consolidation phase	High if CC bears on competent strata	Increase in bearing capacity due to increase in soil strength during consolidation phase. Takes considerable time to develop strength	Significant increase, depending on quality of replacement material.

Source: Table developed on a theme produced by Seah (2005)
*where organic soils are present, creep settlement can occur over the life of the asset for methods reliant on consolidation e.g. drainage, surcharging, basal reinforcement.

In general, where limiting deformation and improving stability are of higher importance, for example bridge foundation construction, ground improvement with SCP or CC is likely to be needed to enable construction to proceed. Where programme is less critical and greater deformation can be tolerated initially, whilst still providing overall stability, other more economic solutions such as SD, PVD or basal reinforcement may be implemented.

9.9 Typical Costs / complexity

The selection of technique also needs to consider the construction and cost aspects. The following Table 9.8 provides an indication of the relative cost of the techniques, a measure of complexity and plant, materials, and labour requirements.

The local availability of skilled workforce and plant is likely to be a significant deciding factor between techniques, even if the technical outcomes are the same.

Table 9.8: Relative cost / complexity of ground treatment methods

Construction aspect	Sand Compaction Pile	Sand Drain with surcharge	PVD with surcharge	Cement Column	Geotextile basal reinforcement	Excavate and Replace / Displacement
Relative Cost	$$$	$$	$$$	$$$$	$	$$$+
Relative Complexity	Medium	Medium	Medium	High	low	Medium
Labour-based solution	No	No	No	No	Yes	No
Skills/experience in Khulna Region	Yes	Yes	No	No	No	Yes
Specialist Plant required	Yes – casing pipe and mechanism to push / hammer / vibrate to depth and compact the sand.	Yes – pipe casing and mechanis m to push / hammer / vibrate to depth and lightly compact the sand.	Yes - Mandrel tool and mechanism to push / hammer / vibrate to depth	Yes – mixing tool and plant to inject binder	Standard earthworks plant	Standard earthworks plant
Availability of plant in Khulna Region	Yes	Yes	No	No	Yes	Yes

Construction aspect	Sand Compaction Pile	Sand Drain with surcharge	PVD with surcharge	Cement Column	Geotextile basal reinforcement	Excavate and Replace / Displacement
Materials required	Sand Geotextile* (*For embankment applications require load transfer platform)	Sand for drains and blanket	PVDs and sand for sand blanket	Cement binder Geotextile* (*For embankment applications require load transfer platform)	Jute geotextile and sand for basal mattress + anchor blocks	Granular fill – cobbles / boulders if placed below groundwater table
Availability of materials in Khulna Region	Yes	Yes	No – but available in Bangladesh	Yes	No – but available in Bangladesh	Yes

Source: Table developed on a theme produced by Seah (2005)

10 Guidelines/Recommendations for Ground Improvement in Khulna Soft Clay Soils

10.1 General

In the preceding Chapters, the following information has been established: -

1) Typical rural road construction methods
2) General considerations for construction on soft foundation soils.
3) Typical defects occurring on existing rural roads
4) Probable deformation mechanisms responsible for defects
5) Potential ground treatment measures available
6) Anticipated performance in respect of deformation and stability of the potential ground treatment measures
7) Constructability and cost issues related to potential ground treatment measures

In this Chapter, the information is brought together for remedial and new build applications to provide guidance on suitable ground treatment methods to meet requirements. For remedial applications, these are classified by defect type observed and hence the underlying cause. For new build, the applications are identified and suitability of techniques suggested in order of increasing cost and likely effectiveness.

10.2 Remedial Applications

It has been identified that defects occurring on rural roads have many causes, some due to construction methods/quality, some due to usage and others as a result of soft foundation soil materials. It is important to establish the reason behind the defect through proper assessment before measures are implemented, or an inappropriate solution may be applied and the cause of defect (for an existing road), or anticipated problem (for a new road) will not be addressed.

For example, embankment slope failure may be due to the geometry, materials and moisture content within the embankment material itself, and nothing to do with the soft foundation soil. Treating the soft foundation soil with one of the methods illustrated would not prevent this occurrence. However, an embankment built in accordance with guidelines at appropriate slope angles, using specified materials and with applicable drainage is likely to fail if built on a soft foundation soil where bearing capacity is exceeded.

In this Section, the defects that can be attributed either partially or wholly to the performance of a soft foundation soil, are identified, and treatments suggested. The scale of treatment applied depends on the severity of the defect in respect of safety, impact on service and available budget. With this in mind, the treatments have been sub-divided as follows: -

Table 10.1: Treatment category

Treatment Category	Description
Maintenance	• **Ground treatment generally not in this category** • Small scale cyclical (or reactive) work that does not materially change the infrastructure. • A short-term solution to improve the immediate impact of the defect on safety and serviceability of the asset. • Further maintenance will be required on a regular basis to maintain the level of service. • Low initial cost (CAPEX) • Higher operational costs as regular maintenance (OPEX)
Remediation (*also termed repair, refurbishment, rehabilitation or heavy maintenance*)	• **Ground treatment options can be considered here, but not for all applications** • Non-routine work to address defects in the asset to restore performance. • A medium-term solution that has a positive impact on the performance, although some maintenance should be expected • Moderate to high initial cost • Low to moderate operational cost
Renewal	• **Ground treatment options can be implemented** • Where the asset is beyond its useful life or changes to the geometry of the asset e.g. widening is required. • A long-term solution, where the asset is completely renewed • High initial cost (CAPEX) • Low operational cost (OPEX) • Potentially high impact on service during construction

Table 10.2 gives lists the typical observed defects introduced in Chapter 4 and 6, the key design attributes for ground treatment, and suggested treatments in the categories listed in Table 10.1 above. For the observed defects where ground treatment would not be beneficial, no options are presented. Accompanying flow diagrams for determining the likely cause of defects observed relating to earthworks and structures are provided in Figure 10.1 and Figure 10.2 respectively.

10.2.1 Limitations of ground improvement methods for remedial applications

Many of the ground improvement methods available worldwide and those that are considered suitable for use in Khulna Region, offer benefits because the deformation and strength characteristics of the existing soil are modified, often in situ, to provide a cost-effective treatment. The simplest, and lowest initial cost solutions generally rely on surcharging the soil, with / without drainage or soil reinforcement, within the limits of stability, hence consolidating the soil with the inherent increase in shear strength and stiffness. If this mechanism is used for remedial applications, the impact of this consolidation on the existing earthwork or bridge structure must be taken into consideration. The use of prefabricated vertical drains, sand drains and basal geotextiles are hence of less use, unless a more widespread renewal is to be implemented. Sand compaction piles or cement columns reduce the amount of total settlement, and these solutions, even though more expensive, are likely to be more effective and reduce the potential for damage to the unaffected part of the earthwork.

Similarly, measures such as sand compaction piles, where densification of the foundation soil is achieved through repeated compaction and the lateral deformation that results, must be carefully considered, particularly if this is proposed on a bridge approach where the abutments are piled. The lateral force of installing the sand compaction pile could damage the piles causing further damage and costly repairs. A non-displacement technique in this circumstance would need to be implemented, potentially cement columns or piles with a load transfer platform, before reconstructing the approach embankment.

Table 9.1 through to Table 9.6 in Chapter 9 provide an indication of these issues that must be considered when proposing ground treatment methods for existing infrastructure, most of which are limited to the renewal category.

Table 10.2: Summary of Remedial Applications and ground improvement techniques

Observed Defects / indicators	**Key design attribute (for ground treatment)**			**Ground Improvement Technique**
		Increasing initial cost (CAPEX) and likely effectiveness		
	(1) Primary (2) Secondary	**Maintenance**	**Remediation** **Option (A) or (B)**	**Renewal** **Option (A) or (B)**
Embankments				
The side slopes are oversteep	n/a			
The embankment is spreading	(2) Deformation - Total settlement (2) Stability - Bearing Capacity / lateral support from foundation soil	*Pavement only surface maintenance*	*Provide toe retaining structure (LGED Standard details may be applicable)*	(A) Reconstruct entire embankment cross-section with basal geotextile reinforcement. Further improvement when used in combination with either prefabricated vertical drains / sand drains and surcharge
The side slopes have failed (rotational failure through foundation soil)	(1) Stability - Bearing Capacity (2) Deformation - differential settlement	*Make safe by isolating traffic from failed area*	Need to avoid differential settlement of existing and new embankment. (A) Reconstruct failed slope foundation with sand compaction piles and geotextile load transfer platform/blanket. Sand compaction piles in rows perpendicular to plane of failure.	(A) Reconstruct entire embankment cross-section with basal geotextile reinforcement in combination with either prefabricated vertical drains / sand drains and surcharge
Leaning trees on slopes	(2) Stability - Bearing Capacity			
Settlement / loss of alignment	(1) Deformation - Total settlement (2) Stability - Bearing Capacity	*Pavement only surface maintenance*	*Pavement reconstruction. Could incorporate geotextile to improve performance*	(A) Reconstruct entire embankment *cross*-section with basal geotextile reinforcement in combination with either prefabricated vertical drains / sand drains and surcharge
Side slopes are eroded	No direct benefit from ground treatment			

Observed Defects / indicators	Key design attribute (for ground treatment)			Ground Improvement Technique
Structures				
There is a step between bridge/culvert and approach	(1) Deformation - Differential settlement (2) Stability - Bearing Capacity	*Pavement only surface maintenance to bring up level*	*No solution – difficult to underpin approach embankment with ground treatment methods – potential damage to abutment and foundations due to ground densification*	(A) Reconstruct entire approach embankment cross-section with basal geotextile reinforcement in combination with Sand compaction piles or cement columns. Proximal to bridge foundations, ground treatment needs careful execution to avoid damage to substructure and superstructure.
Culvert blocked / below water line	(1) Deformation - Total settlement	*Ensure culvert capacity is not compromised by removing any debris / blockages at inlet and outlet and through culvert bore.*		Deformation of culvert likely to have occurred due to settlement of embankment. (A) With consolidation more complete, replace existing culvert channel / pipe or add parallel structure at higher level. Culvert headwalls (if present) may require additional consideration
Retaining structure deformed	(1) Stability - Bearing Capacity / lateral support from foundation soil (2) Deformation - Total settlement	*Repair any obvious defects e.g. wiring, mesh*	(A) For embedded structures, e.g. palisade, add intermediate supports in front of wall (B) If relatively shallow embankment, reconstruction of pavement with geotextile could limit impact on stability of foundation soil and lateral forces acting on wall.	Replace retaining structure (A) for gravity structures, e.g. gabion/brick, provide suitable foundation, Sand compaction piles or cement columns, before reconstruction (B) For embedded structures, e.g. palisade, reconstruct with deeper embedment and/or closer spacing of piles. Consider use of geotextile in backfill to reduce lateral load on wall.

Observed Defects / indicators	Key design attribute (for ground treatment)			Ground Improvement Technique
Abutments deformed / mis-alignment	(1) Stability - Bearing Capacity (2) Deformation - Total settlement	*No solution*	*No ground treatment solution – underpinning structure with piles would be potential option*	Replace abutment structure (A) for assumed shallow foundation, sand compaction piles or cement columns before reconstruction, although may opt to pile.
Pavements				
There are lateral cracks in the pavement	(2) Deformation - Total settlement	*Pavement only surface maintenance*	*Pavement reconstruction. Could incorporate geotextile to improve performance* *If toe wall present, see 'retaining structure deformed'*	(A) Reconstruct entire embankment cross-section with basal geotextile reinforcement. Further improvement when used in combination with either prefabricated vertical drains / sand drains and surcharge
The pavement is undulating / disturbed	(2) Deformation - Total settlement (2) Stability - Bearing Capacity	*Pavement only surface maintenance*	*Pavement reconstruction. Could incorporate geotextile to improve performance* *If toe wall present, see 'retaining structure deformed'*	(A) Reconstruct entire embankment cross-section with basal geotextile reinforcement. Further improvement when used in combination with either prefabricated vertical drains / sand drains and surcharge
Wheel rutting of the pavement	No direct benefit from ground treatment			
damaged shoulder	No direct benefit from ground treatment			

Figure 10.1: Classification of embankment defects

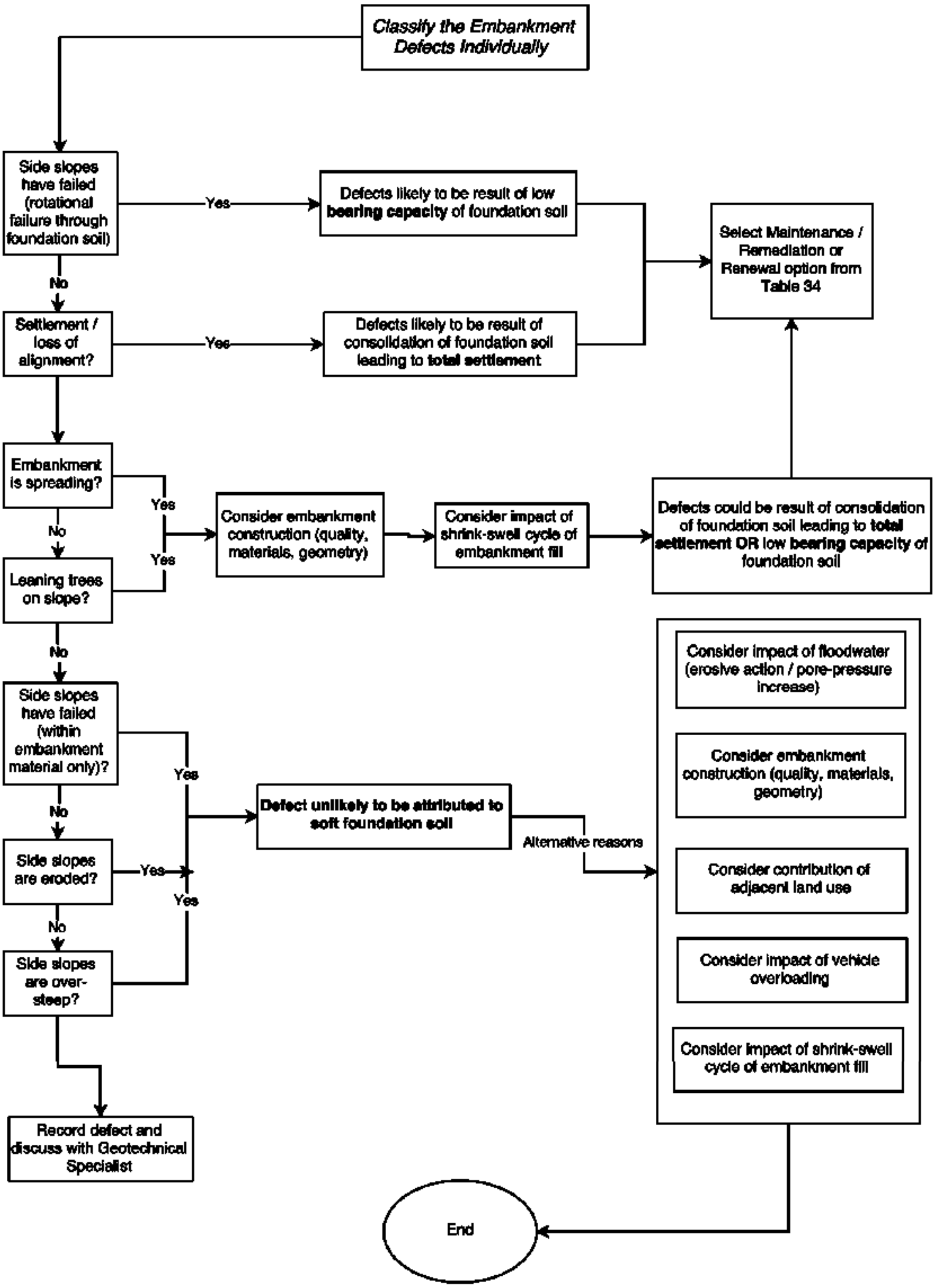
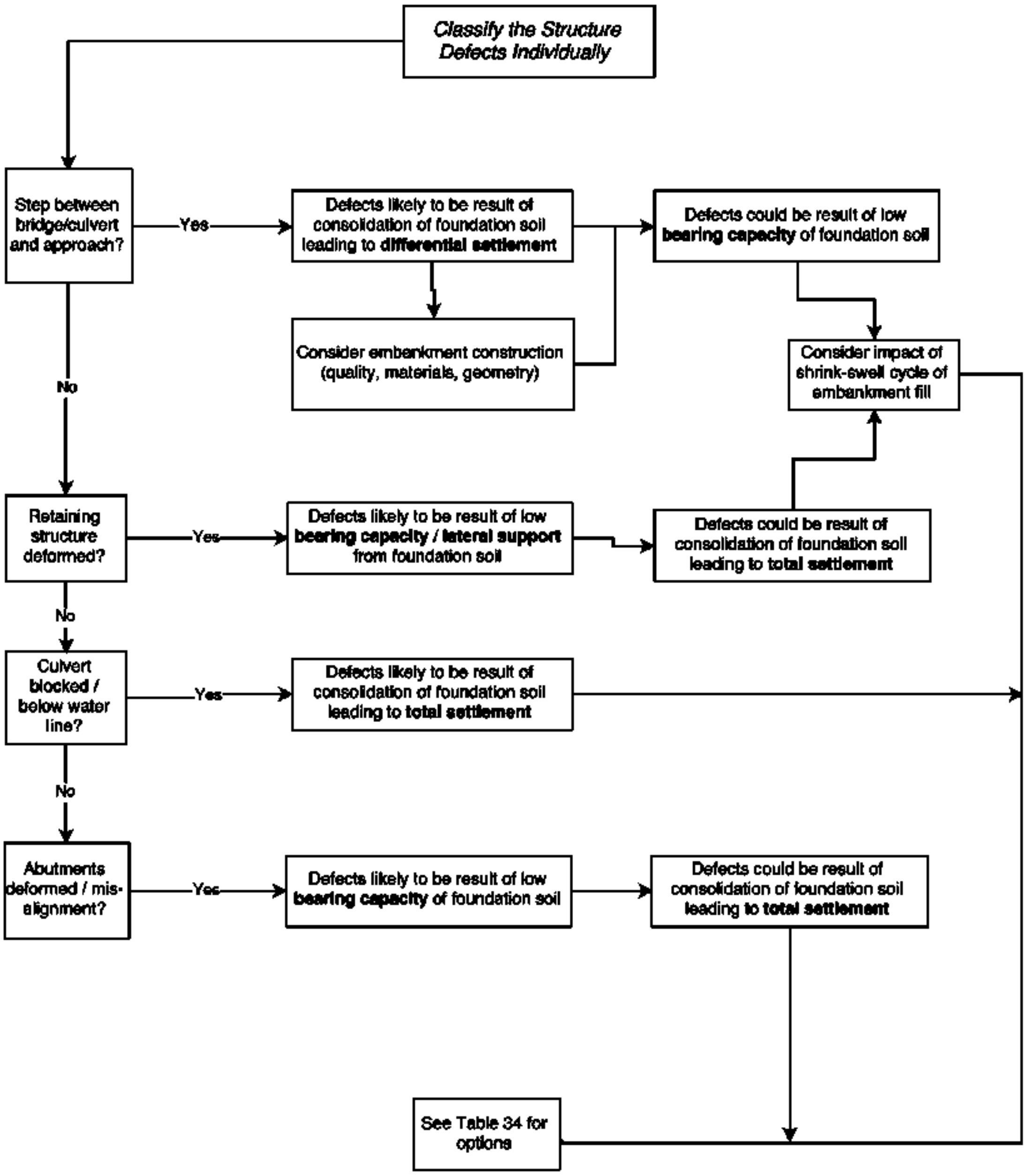


Figure 10.2: Classification of structure defects



10.3 New build and widening applications

Implementation of ground improvement methods for new build applications is far simpler than for remediation of an existing road (see comments in Section 10.2.1). The impact of constructing adjacent to existing infrastructure still has to be considered, particularly in situations where widening, or raising the height of an earthwork is considered. Widening of bridge structures is likely to be limited on rural roads, but culvert widening to match earthworks would be more common.

10.3.1 Situations where ground improvement should be considered

The following Section should be read in conjunction with Table 10.3 and the Design Options Flow Diagram provided in Figure 10.3 to Figure 10.8 listed below: -

- Chart 1 Design Phase Process Diagram (based on Figure 7.3)
- Chart 2 Conceptual Design Options Flow Diagram (Minor Bridge)
- Chart 3 Conceptual Design Options Flow Diagram (Minor Culvert)
- Chart 4 Conceptual Design Options Flow Diagram (Bridge Approach Embankment)
- Chart 4a Conceptual Design Options Flow Diagram (Bridge Approach Embankment – approach constructed after bridge)
- Chart 5 Conceptual Design Options Flow Diagram (General Embankment)

10.3.1.1 Minor bridge foundation

When constructing bridges with significant spans and abutment loadings is high, and limiting deformation is critical, piled foundations for abutments and piers are the clear choice where foundation soils are soft.

For smaller bridges, with relatively low abutment loading, ground treatment methods may be appropriate. Usually the controlling factor will be tolerance of the superstructure to deformation, or differential movement, between the abutments, and bridge designers will specify what is required from the abutments and foundations. The techniques considered that offer the most control over deformation is sand compaction piling and cement columns. Both methods improve the stiffness and strength of the improved material, providing improved bearing capacity and deformation characterisitcs under foundation loading. As noted in Chapter 9, the sand compaction pile may improve the strength/stiffness by a factor of 3-4 times. It should be noted that the cement column is unreinforced and cannot accomodate lateral load or shear forces developed by a bridge and consitututes a mass strength improvement method. Depth constraints on both these techniques may limit use, however, where soft ground is present to beyond the depth that treatment can be applied (see summary tables in Chapter 9).

The other ground techniques rely on consolidation of the ground to develop strength and stiffness and this deformation is unlikely to be acceptable for a bridge foundation within construction programme constraints.

10.3.1.2 Culvert foundation

The performance of foundations for culverts is important to ensure both the stability of the structure, but also to ensure hydraulic function is not impaired due to consolidation of the foundation soil and consequent loss of freeboard. Where the culvert is constructed as part of an overall road construction project, the use of prefabricated vertical drain or sand drain with surcharge may be appropriate, particularly if this approach is used for controlling deformation of adjacent earthwork embankments. Use of sand compaction piles or cement columns could be implemeted if programme is tight, or where the culvert is installed after the earthwork has been constructed, or as additional hydraulic capacity in an existing earthwork.

10.3.1.3 Bridge approach embankment

The transition between earthworks and structures is the location where differences in foundation stiffness are most notable (the bridge at Mongla Site 21 is a good example of this). Differential movement between, for example, a piled bridge abutment and an earthwork constructed on the adjacent foundation soil, without any means of improvement can be significnat and lead to poor levels of service, high levels of maintenance, and damage to vehicles. Bridge approach embankments, by their nature, are often higher than other general embankments in lowland applications, and exert higher loads on the foundation soil. This can lead to settlement as noted above, but also but also impart additional vertical and horizontal load on bridge supports, for example negative skin friction and lateral load. Negative skin friction occurs as the soil material surrounding the pile consolidates and the shear forces acting at the soil/pile boundary, instead of providing support, act in the opposite sense, leading to additional loading being applied, that could overstress the pile and foundation.

Lateral loading occurs as the foundation soil moves outward in response to the vertical loading, and this mechnism may be more critical than the development of negative skin friction if the approach embankment is built rapidly, and pore water pressures cannot dissipate. The lateral loading on the piles manifest as additional shear force and bending moment, that are very likely to damage a piled foundation and lead to problems with the abutment and superstructure.

The ideal situation is to create a transtion zone between the earthwork, where stiffness of the foundation is less critical, and the bridge structure, where foundation stiffness is critical and tolerance to movement less. This can be achieved through application of sand compaction piling, cement columns and sand drains / prefabricated vertical drains with surcharge depending on construction programme. The bearing capacity of the soil limits the rate of construction for consolidation techniques (sand drains / prefabricated vertical drains) as indicated in Section 6.3.3.

Ideally the construction of the approach embankment and ground improvement works should precede the construction of the bridge abutment and foundations. This limits the potential for negative skin friction and lateral loading on piled abutments that can develop from consolidation (sand drains / prefabricated vertical drains) and densification (sand compaction piling) techniques. If this is not possible, cement columns together with an load transfer platform would be a preferred option in close proximity to the bridge.

10.3.1.4 General embankments

Low height embankments may not require ground improvement. It can be seen in Section 6.3, that deformation in the order of 150mm, or 8% of the embankment height, should be expected for a typical 2m high earthwork using the generic ground model. If this can be tolerated, with the expectation of some cracking and spreading observed in the pavement and embankment, the additional expense associated with ground improvement may not be warranted.

Where higher embankments are to be constructed, the stability of the foundation soil becomes more critical in the short term, and staged construction may be required to reach final level with causing a bearing capacity failure through the foundation soils. Typically, in the soft foundations soils of Khulna Region, embankments of 2m high and above would fall in this category, but this may be lower for very soft soil situations. Timing thus becomes more challenging where multiple stages of construction are needed and it may be beneficial to either (a) prevent bearing capacity failure using basal reinforcement or (b) expedite consolidation through use of sand drains / prefabricated vertical drains with surcharging.

The former is cheaper, but deformation should be expected over a longer period as consolidation is not accelerated as in the use of sand drains / prefabricated vertical drains. The two options (a) and (b) could also be used together.

Excavation and replacement or displacement are also options, particularly if the depth of soft soil is limited (<5m) or where improvement to bearing capacity, as opposed to settlement, is the main consideration. It is, however, likely to be more expensive than other solutions due to the significant volume of import/export material (see Table 9.7 and Table 9.8 for comparison of options).

Table 10.3: Summary of New Build Application and applicable ground improvement techniques

New Build Application	Key design attribute	Ground Improvement Technique		
		Increasing initial cost (CAPEX) and likely effectiveness		
		(1)	(2)	(3)
Minor bridge foundation	Deformation	Sand compaction piles	Cement columns	Piles*
Culvert foundation	Deformation	sand drains / prefabricated vertical drains with surcharge	Sand compaction piles	Cement columns
Bridge approach embankment	Deformation and stability	Sand drains or prefabricated vertical drains with surcharge with/without Geotextile basal reinforcement	Sand compaction piles + load transfer platform	Cement columns + load transfer platform
General embankment	Stability	Geotextile basal reinforcement	Sand drains or prefabricated vertical drains with surcharge	Sand drains or prefabricated vertical drains with surcharge + Geotextile basal reinforcement or excavate & replace / displacement

*Piles are not a ground improvement method, but are widely used for bridge foundations in Bangladesh.

Figure 10.3: Chart 1 Design Phase Process Diagram (based on Figure 7.3)

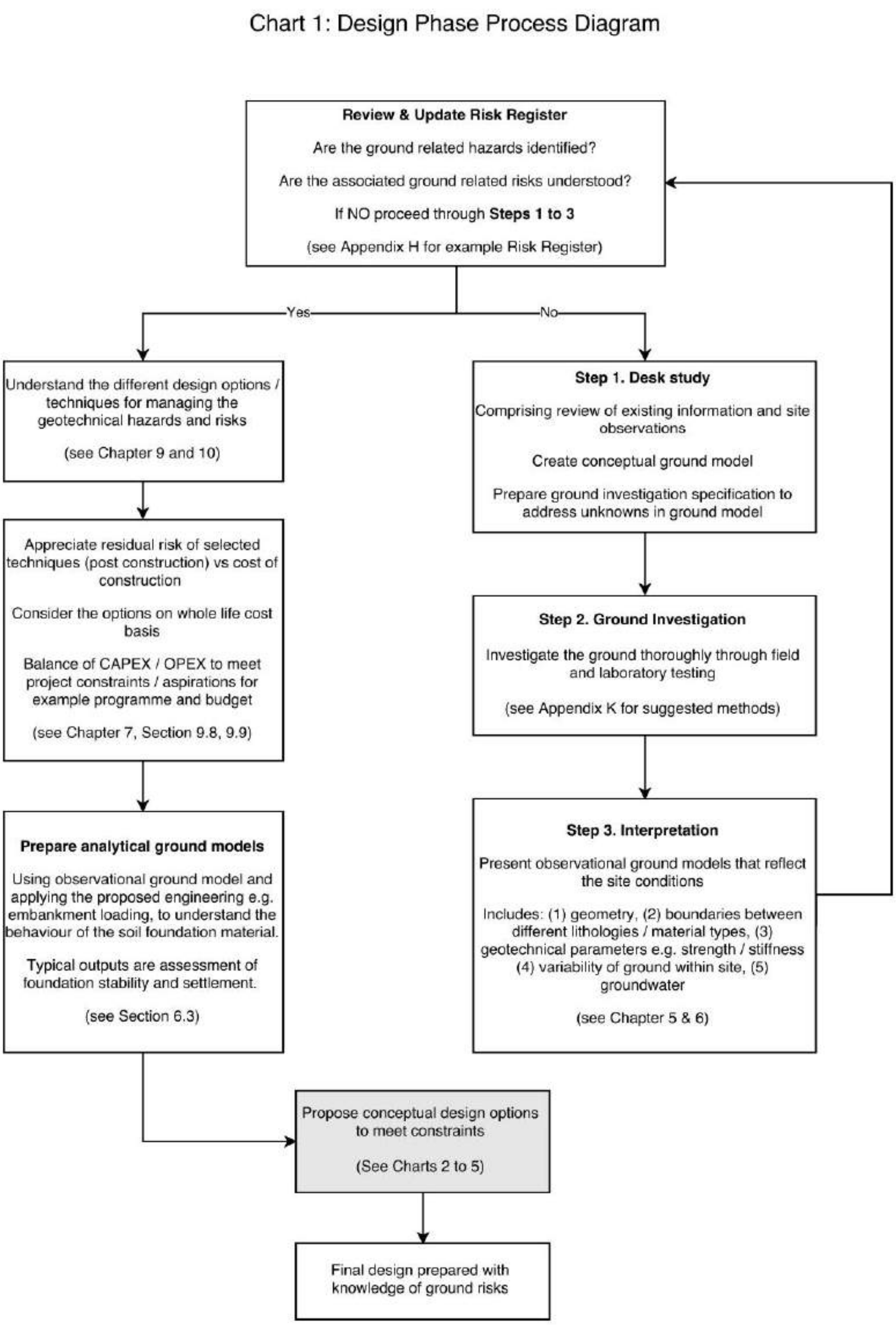


Figure 10.4: Chart 2 Conceptual Design Options Flow Diagram (Minor Bridge)

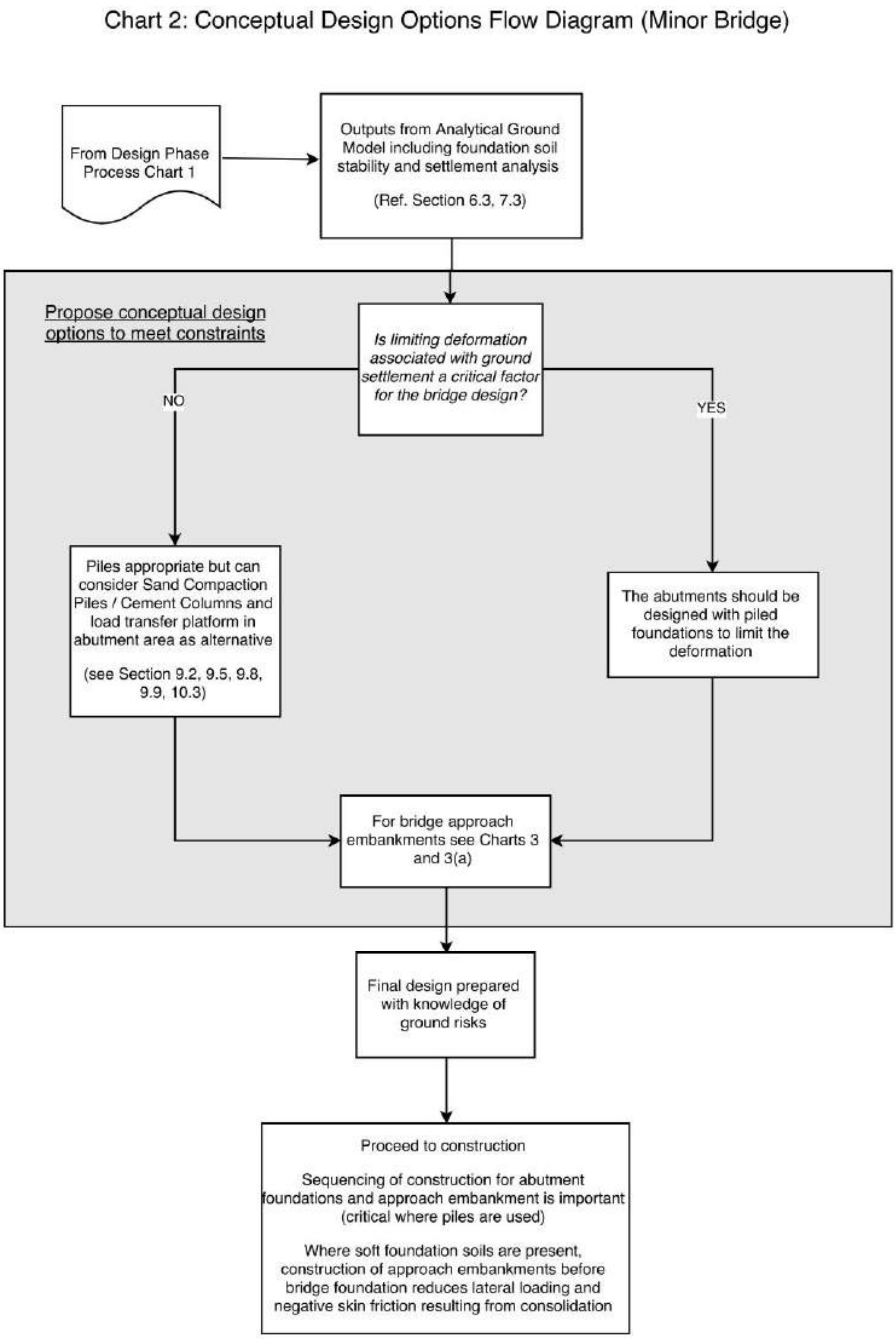


Figure 10.5: Chart 3 Conceptual Design Options Flow Diagram (Minor Culvert)

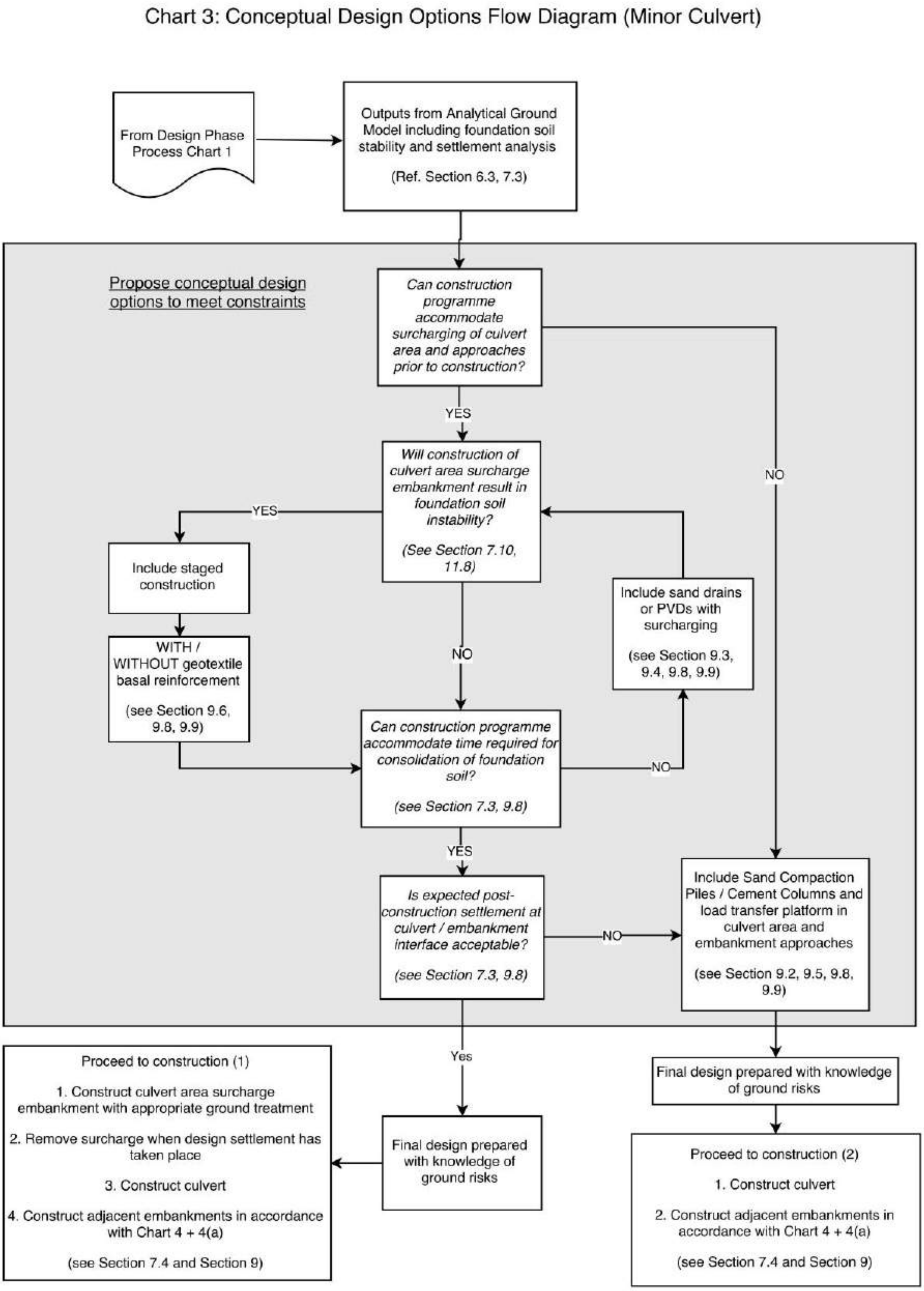


Figure 10.6: Chart 4 Conceptual Design Options Flow Diagram (Bridge Approach Embankment)

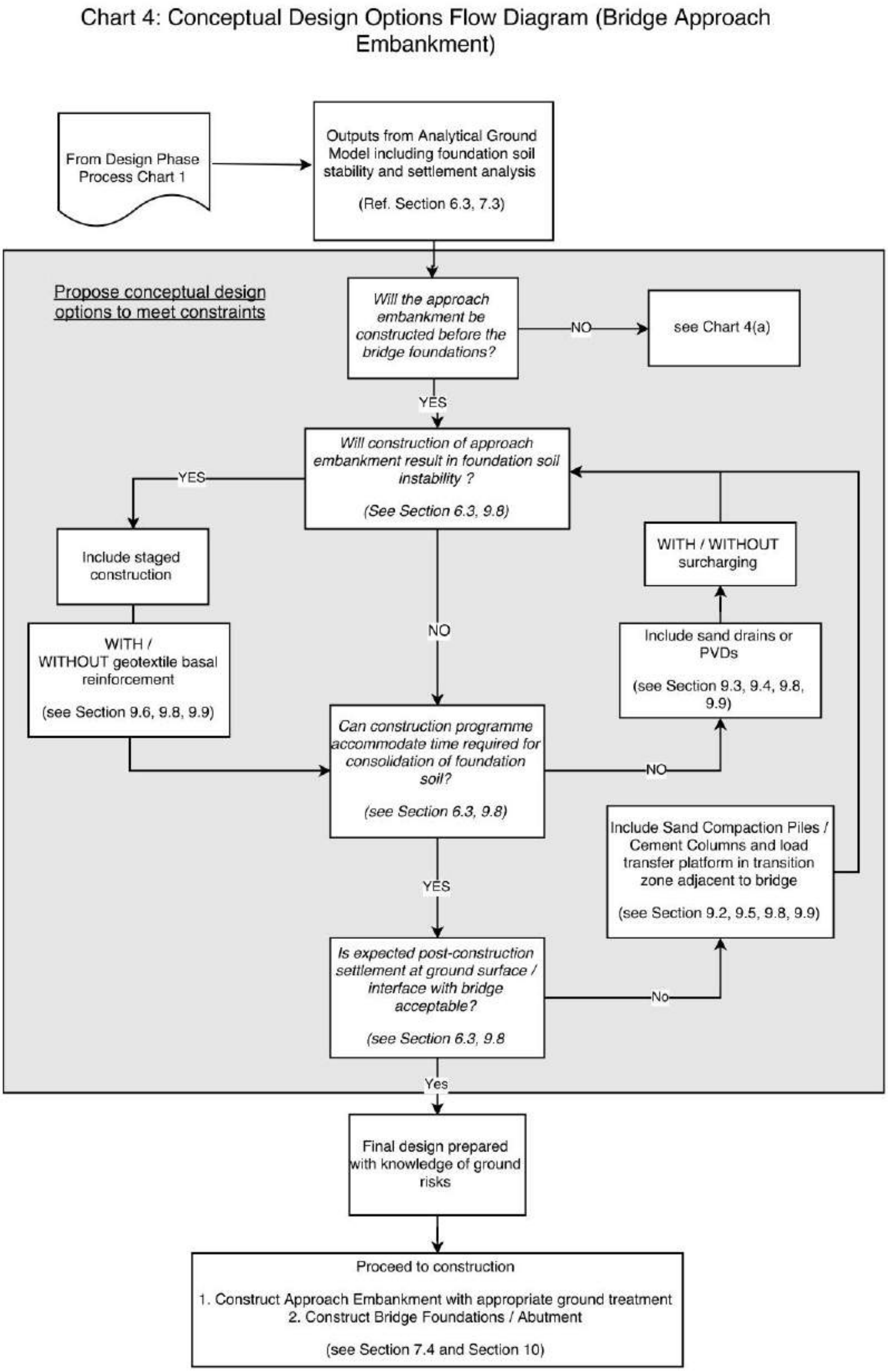


Figure 10.7: Chart 4(a) Conceptual Design Options Flow Diagram (Bridge Approach Embankment – approach constructed after bridge)

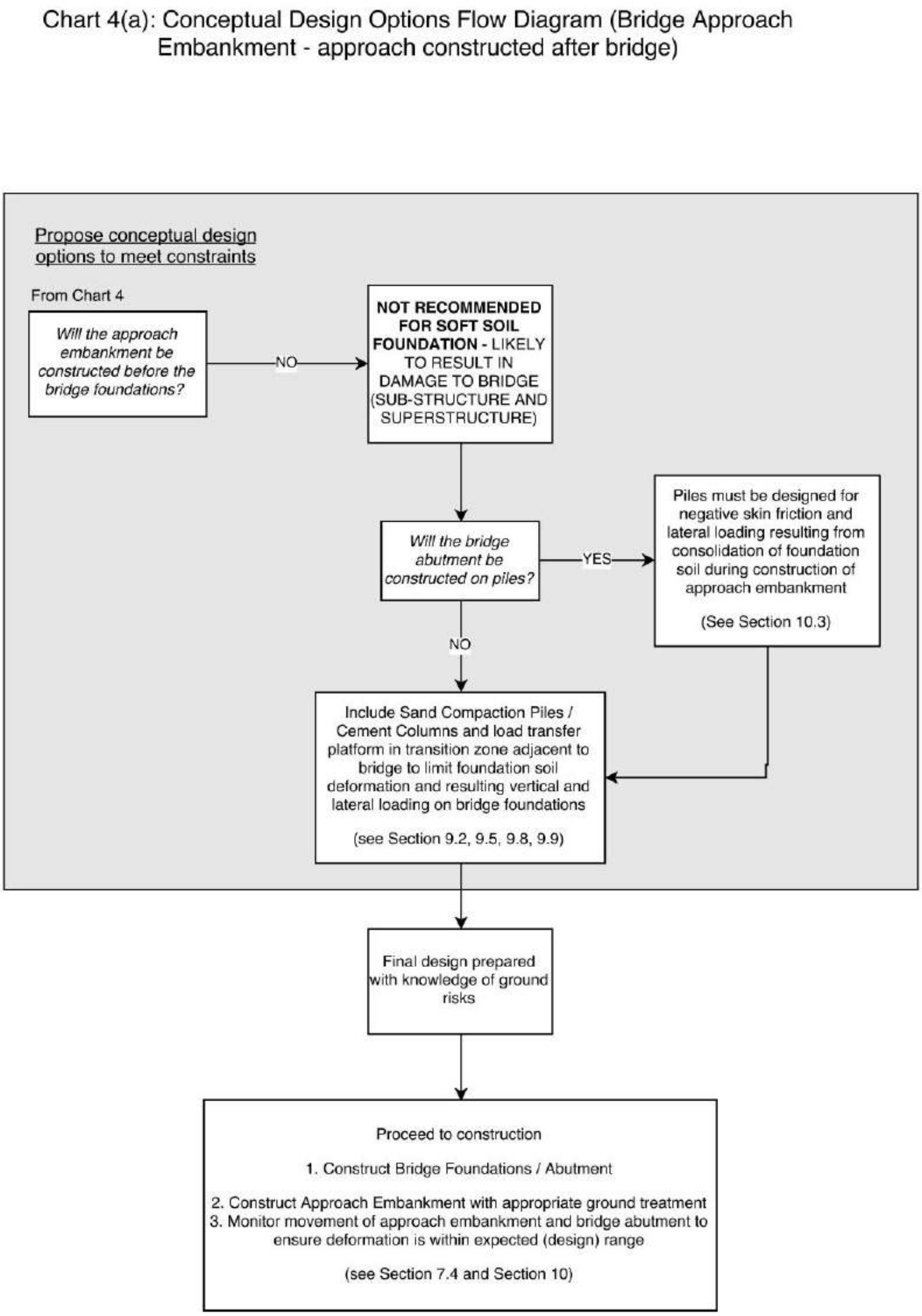
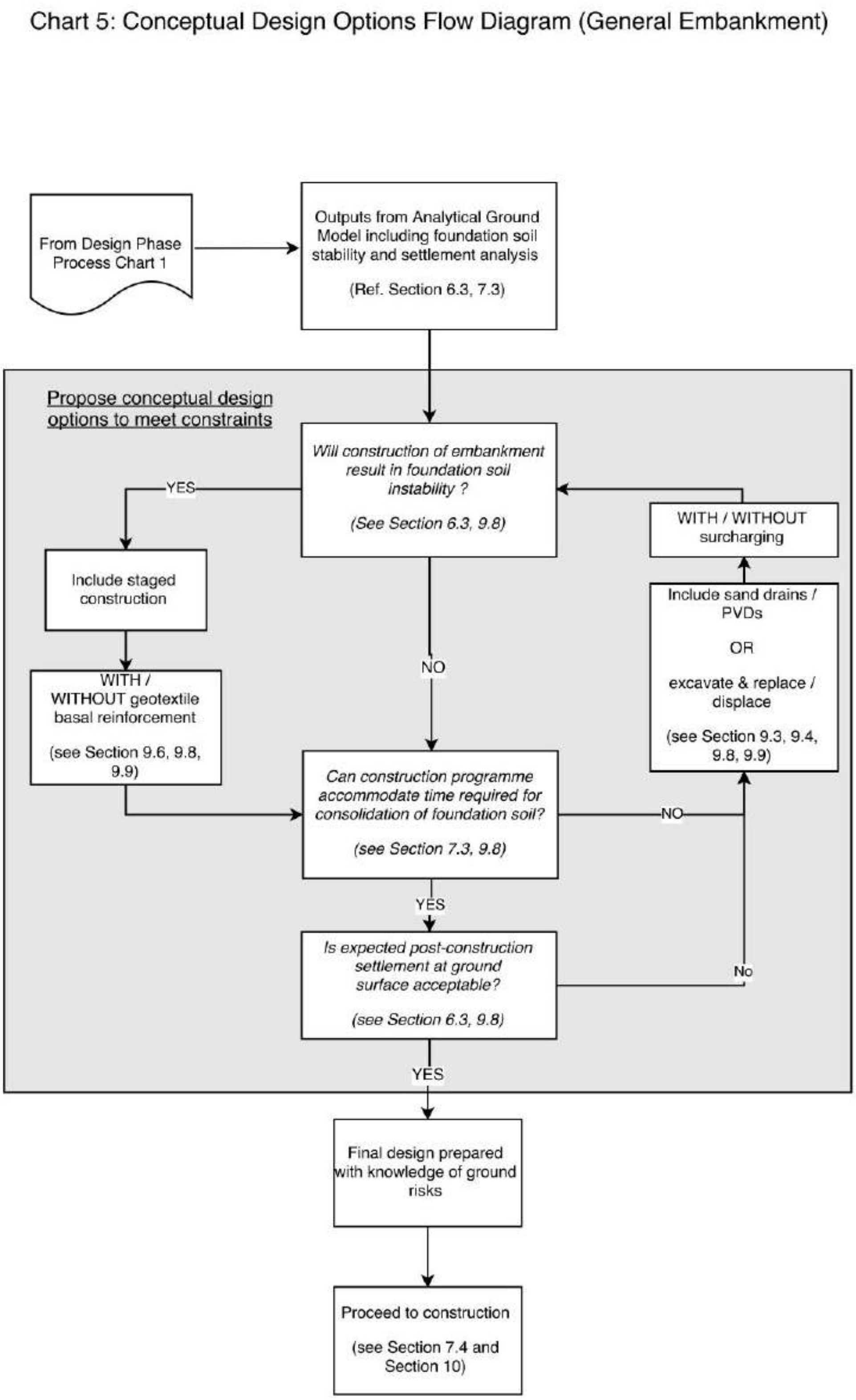


Figure 10.8: Chart 5 Conceptual Design Options Flow Diagram (General Embankment)



11 Further research work

11.1 General

The research project has reviewed existing literature on ground improvement techniques, geological & ground investigation data and site observations of typical rural roads and the deformation problems faced. Based on the research, appropriate ground improvement techniques to address typical defects and new build scenarios are presented. Some of the techniques are widely used in Bangladesh, but others are not. It is also clear that the successful application of the techniques for rural road applications has not been widely tested or documented to demonstrate success or otherwise. Sites where ground improvement had been used for rural roads were not available or did not exist for the purposes of the research project.

A critical element for implementation is to demonstrate the techniques in the field under realistic construction conditions. Proposals for site trials and monitoring of the ground improvement techniques listed and the fibre mixing options are given.

For all solutions, asset management and ground investigation, a series of suggested research topics and improvements are provided that will provide greater confidence in the solutions provided and aim to simplify the design process through use of standard details / designs and charts.

11.2 Further research themes

Areas for further research were discussed at the Stakeholder Workshop. A long list of topics is presented in Table 11.1) and from these, a number have been selected that represent the most valuable research topics that would refine, simplify and facilitate the implementation of ground improvement methods discussed. These selected items are discussed in more detail in Section 11.3.

Table 11.1: Areas for future development and research – long list

Area for further research	Description – long list
Asset Management	• Links to Policy and Guidance document – frequency / pro-forma • Condition surveys • Data Recording / access • Accessibility of BH logs and testing data • Training • Life cycle costing – importance of considering asset life-cycle in the design stage (from Stakeholder Workshop)
Ground Investigation / validation	• Alternatives to BHs / SPT N • Correlation with Panda probes to get Cu / N profile cheaply • Simple basis for determining OC – leading to parameters Cc / Su • Database of BH logs and test results for LGED to access (to avoid need for GI) • Use of piezocone or light Dutch Cone Penetrometer should be investigated (from Stakeholder Workshop) • Spatial mapping and statistical evaluation of data that can be used as preliminary site data (from Stakeholder Workshop)

Area for further research	Description – long list
Applicable to all / many Ground Treatment Methods	• Scale test required / fully monitored • Plant and operatives – applicability to rural road locations • Constructability and quality control • Retro-fit through existing earthwork as remedial / improvement (up to underside of existing earthwork) – not near bridge foundation…. • Settlement / load profiles for different techniques
Sand Compaction Piles	• Hammer only / hammer + vibrate • More sophisticated methods e.g. SAVE-SP • Impact of depth / spacing / diameter • Impact of floating piles • How does variable stratification impact settlement? • Use with / without jute LTP (earthwork/bridge) – jute deterioration a problem for long term LTP • Alternatives to sand – shells, glass? Durability during compaction
Sand Piles / PVD with surcharge	• Jute PVD – effectiveness vs synthetic • Alternatives to sand – shells, glass? Durability less of a problem as lighter compaction
Cement Columns	• Can admixtures be effectively mixed in situ (mass / columns) • Cheaper alternatives to cement • Fabrication of tools • Use for underpinning as remediation of existing structures
Basal Reinforcement	• Use in preventing BC failure in soft ground embankment construction • Life expectancy of jute • Use with other techniques
Shallow soil mixing	• Use of basic farming e.g. plough and or standard construction plant
Fibre reinforcement	• Plastic bags – shredded and mixed with soils • Combination of fibres and admixture • Suitable plant for mixing – see 'Shallow Soil Mixing' • Jute fibres – Sonthwal and Sahni (2015) • Could these be used in SCP for greater shear resistance (slope stability)
Re-use of waste products/materials	• Tyres / tyre bales for earthwork construction or creating cellular raft. • Tyres considered as waste? • Plastic bags – shredded and mixed with soils
Field application of existing Laboratory based testing	• Can admixtures be effectively mixed in situ (mass / columns) • Can admixtures be effectively mixed ex-situ and placed (mass)

11.3 Recommended Further Research Topics (short list)

11.3.1 Field Trials with sand compaction pile and sand drains to demonstrate plant and skills for rural applications

It is appreciated that the sand compaction piling and sand drains are used already on larger scale road and buildings projects, but the use on rural roads is more limited. It is hence important to establish whether the skills, plant and quality control aspects of these solutions can be successfully delivered in accordance with design and specification. A field trial would be the most obvious way of undertaking this validation exercise, using local supply chain. The quality of the installation, particularly for sand compaction piling, is essential in providing the overall improvement in deformation and strength characteristics that would be needed for a bridge abutment, for example. The structural engineer designing the abutment and bridge superstructure needs to have confidence in the design bearing capacity and settlement characteristics, or else a piled foundation would need to be specified.

Field trials could also be used for training and capacity building purposes of the local labour market and LGEDs.

11.3.2 Effectiveness of Basal Reinforcement in limiting differential settlement and avoiding BC failure

Basal reinforcement using synthetic polymer geotextiles is widely used internationally to support the loads of new embankment infrastructure. The use of geotextiles in Bangladesh is generally restricted to pavement strengthening layers.

The use of jute as an embankment basal reinforcement technique has been recommended for implementation in earlier Chapters due to the lower cost and increased use of labour when compared to other solutions. However, field results of application are largely untested and records of performance of this technique are unavailable in Bangladesh (or not made available to the research team). So, it would be highly beneficial to implement the technique, using international design standards, and closely monitor the site over a number of years, including pre- and post- consolidation ground investigation works.

11.3.3 Shallow soil mixing with admixtures to improve shear strength of foundation soil

Shallow soil mixing with standard available plant, or basic modifications to plant can be researched and trials made. The technique is widely used on its own or in combination with, for example, cement columns. But the addition of alternative admixtures has received some research in the laboratory that has proven of benefit to soil properties.

11.3.4 Shallow soil mixing with fibres to improve shear strength of foundation soil

This method has been demonstrated using jute in the laboratory in India, and could represent an economic method to improve foundation soils at relatively shallow depths. It is more likely to be effective in mass improvement rather than discrete columns, and the lifespan of the fibres also needs to be considered. But initial results show that the soil strength initially at CBR=3%, can increase to >5% with the addition of jute fibres (1% of dry weight of soil). The mixing of soil with waste e.g. plastic bag strips, or shredded tyres could also be looked at.

11.3.5 Spatial mapping and statistical evaluation of data

As part of the project, a paper titled 'Statistical Evaluation of Bearing Capacity of Khulna Sub-Soil' prepared by KUET was reviewed. The work described in the paper was also subject to discussion at the Stakeholder Workshop.

The paper describes spatial mapping of bearing capacity based on a set of boreholes and test data and how this can be used for assessing the likely bearing capacity of soils to enable optioneering to take place. Further soil maps based on information that Government and Educational facilities have collated would be very beneficial for assessing likely hazards, and could form part of the Ground Model development and Risk Register presented in the report. Along with geological maps, aerial photos and other sources of information, these would be invaluable and cost-effective way of establishing engineering solutions before ground investigation takes place. Further development and population of the inventory are recommended.

11.4 Future stage(s) of the project

As stated above, there are many areas where further research would benefit, particularly with the field application and validation of the techniques outlined in the report. This was particularly felt by the attendees at the Stakeholder Workshop – see feedback in Appendix B. Fieldwork and piloting techniques is outside the current or immediate future scope of the Project.

Confidence in the process and effectiveness of the techniques provides the basis for standard design details and specifications that can be adopted in the future.

The next phase of the project may include the following stages: -

1. Design Guidelines: Production of design guidelines for ground improvement techniques for rural road construction on soft foundation soils including;
    - Provision of typical designs for standard engineering applications
    - Provision of standard details that can be used for ground improvement applications
    - Provision of standard specifications that can be used for ground improvement applications
    - Provision of design charts that may be used for simple ground improvement applications
    - Provision of training for scheme designers for application of the new design guidelines;

2. Construction Guidelines: In order to support future rollout of the ground improvements solutions for the rural roads network in areas of soft soil foundations, this phase would involve the development of field guidelines for construction practitioners including capacity building / training of field engineers and construction practitioners for application of the new construction guidelines.

12 Conclusions

There are major concerns for the resilience of rural road embankments exposed to an aggressive coastal environment, in areas of high flood risk and where embankments are often constructed on soft soil deposits, sometimes with high compressible organic content. This study has collated the relevant findings from existing research, field observations and ground investigation to understand the effectiveness and limitations of existing ground improvement techniques implemented in Khulna region, and has developed appropriate recommendations to overcome the typical construction challenges for road embankments and structures in Khulna region.

The sources of information used to develop the findings of the Final Report are wide and varied. The principal sources comprise: -

- Stakeholder meetings;
- Publications and academic research on the geology of the Khulna Region;
- Publications relating to different types of ground treatment methods available worldwide and those that have been used in Khulna Region;
- Observational visits to earthworks and structures to determine indicators of deformation mechanism;
- Intrusive investigation data and laboratory testing (existing information from literature and new data from selected sites) to characterise the ground;
- Publications and academic research on new ground treatment methods that could be developed into field techniques.

Using the detailed investigation and analysis of the results, observational ground models have been developed for to help better understand the deformation mechanisms and assess the likely contributory causes. The key defects categories identified were as follows: -

- Embankment Defects
- Structure Defects (related to ground conditions);
- Pavement Defects observed.

The probable cause/mechanisms leading to the defects have been categorised: -

1) Deformation
    a) Differential settlement
    b) Total settlement
2) Stability
    a) Bearing capacity
    b) Lack of lateral support (provided by foundation soil)
3) Floodwater (erosive action, pore-pressure increase)
4) Adjacent land use
5) Shrink/swell cycle (of soils)
6) Embankment construction (quality, materials)
7) Vehicle overloading

Ground improvement techniques have been presented that are considered either (a) technically feasible and (b) within the likely budget for rural road construction together with guidance for implementation to deal with specific construction issues. From the literature review, and discussions with local stakeholders, the techniques are mostly in use in Bangladesh, although not necessarily for the rural road application. The ground improvement techniques considered within the report are: -

- Excavate and Replace / Displacement
- Sand Compaction Pile
- Sand Drain (with surcharge)
- Prefabricated Vertical Drains (with surcharge)
- Geotextile basal reinforcement
- Cement Columns

Topics for further research are presented that will improve the ability to apply ground improvement techniques in Khulna Region. The recommended further research includes:

- Field Trials with sand compaction pile and sand drains to demonstrate plant and skills for rural applications
- Effectiveness of Basal Reinforcement in limiting differential settlement and avoiding BC failure
- Shallow soil mixing with admixtures to improve shear strength of foundation soil
- Shallow soil mixing with fibres to improve shear strength of foundation soil
- Spatial mapping and statistical evaluation of data

The next stages of the project may include the following: -

1. Design Guidelines: Production of design guidelines for ground improvement techniques for rural road construction on soft foundation soils.

2. Construction Guidelines: In order to support future rollout of the ground improvements solutions for the rural roads network in areas of soft soil foundations, this phase would involve the development of field guidelines for construction practitioners including capacity building / training of field engineers and construction practitioners for application of the new construction guidelines.

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Appendix A: Stakeholder meetings

Meeting	Key Attendees	Notes
Start-up meeting (09-08-16)	Md. Abul Bashar (LGED superintending engineer) Md. Shahidul Haque (LGED superintending engineer) Md. Abul Kalam Azad (LGED Additional Chief Engineer (Implementation) Les Sampson (ReCAP/Cardno) Chandra Shrestha (ReCAP / Cardno)	Ian Duncan presented the project proposal to the assembled LGED members
Blue Gold Program (10-08-16)	Mofazzal Ahmed (Deputy Component Leader)	Useful notes gained on construction methods and constraints, maintenance and typical ground improvement methods that are implemented
Follow-up meeting with LGED + laboratory visit (11-08-16)	Md. Abul Bashar (LGED superintending engineer) + LGED design team	Discussed various issues: - Current practice for embankment construction and remediation Typical form of contract Ground investigation capability Construction plant and staffing Availability of data sources such as GIS, geological maps
Follow-up meeting with LGED (18-08-16)	Tapas Chowdhury, Senior Assistant Engineer, Design Unit + LGED Design Team	Discussed various issues: - Current practice for asset management LGED experience in soft ground + design methodology LGED owned plant and GI equipment Sites for initial assessment
Technical Meeting (07-11-16)	Md. Abul Bashar (LGED superintending engineer) + LGED design team	Presentation of Inception Report.
Technical Meeting (07-02-17)	Md. Abul Bashar (LGED superintending engineer) + LGED design team	Presentation of Field and Laboratory Testing results
Technical Meeting (11-07-17)	Md. Abul Bashar (LGED superintending engineer) + LGED design team	Review of Draft Final Report and recommendations for inclusion in Final Report.
Stakeholder Workshop (20-09-17)	See Appendix B	Delivery of Draft Final Report findings. See Appendix B

Appendix B: Stakeholder workshop questions and responses

The purpose of the workshop was to demonstrate the progress of the project against the above stated aims; provide technical training and capacity building through the content of the project presentation and response to technical questions; and obtain feedback for the ongoing development, uptake, and embedment of the project findings and recommendations. The technical feedback and recommendations from the workshop have been incorporated into the Final Report and subsequent phases of the project, for which this Stakeholder Report acts as a supporting document.

Agenda

09:30 – 10:00	Walk-in and registration (MM)
10:00 – 10:40	Inaugural Session to be chaired by Mr. Md. Abdul Kalam Azad, Additional Chief Engineer (Implementation) and Chairperson, ReCAP-ASCAP Steering Committee • Welcome address by Mr. Abdul Bashar, Superintending Engineer, LGED; • Speech by Jasper Cook, Team Leader, ReCAP; • Speech Mr. Md. Abdul Kalam Azad, Additional Chief Engineer, LGED; • Speech and Inauguration of working session, Mr. Shyama Prosad Adhikari, Chief Engineer, LGED.
10:40 – 11:00	Tea Break
11:00 – 13:00	Working Session to be chaired by Mr. Md. Khalilur Rahman
11:00 – 11:20	Presentation on “Overview of Current Ground Improvement methods used internationally & in Khulna Region (MM)
11:20 – 11:40	Presentation on “Field Survey for Sites in Khulna Region” (MM)
11:40 – 12:30	Laboratory Testing & Ground Models from Sites in Khulna Region (MM)
12:30 – 13:00	Open Discussion
13:00 – 13:30	Summary Session, to be chaired by Md. Abdul Kalam Azad, Additional Chief Engineer (Implementation)

Attendees

The list of those who attended the event are included in Table 12.1 below:

Table 12.1: List of Attendees from Workshop

#	Name	Organisation & Designation
1	Md. Abul Kalam Azad	Local Government Engineering Department, Additional Chief Engineer(Implementation)
2	Md. Mohsin	Local Government Engineering Department, Additional Chief Engineer, Integrated Water Resources Management (IWRM)
3	Md Joynal Abedin	Local Government Engineering Department, Additional Chief Engineer (Maintenance)
4	Mohammad Anwar Hossain	Local Government Engineering Department, Additional Chief Engineer (Urban Management)

#	Name	Organisation & Designation
5	Md. Khalilur Rahman	Local Government Engineering Department, Additional Chief Engineer
6	Iftekhar Ahmed	Local Government Engineering Department, Additional Chief Engineer
7	Jasper Cook	Chief Scientific Advisor, Research for Community Access Partnership (ReCAP/AsCAP)
8	Les Sampson	Team Leader, Research for Community Access Partnership (ReCAP/AsCAP)
9	Maysam Abedin	Regional Technical Manager, Asia, Research for Community Access Partnership (ReCAP/AsCAP)
10	AKM Sahadat Hossain	Local Government Engineering Department, Senior Engineer, Integrated Water Resources Management (IWRM)
11	Khondakar Ali Noor	Local Government Engineering Department, Senior Engineer (Design)
12	Md. Abul Bashar	Local Government Engineering Department, Senior Engineer, Integrated Water Resources Management (IWRM)
13	Abdur Rashid Khan	Local Government Engineering Department, Senior Engineer (Training)
14	Md. Mosleh Uddin	Local Government Engineering Department, Senior Engineer (Admin)
15	Md Abu Md Shahriar	Local Government Engineering Department, Project Director, Greater Faridpur Rural Infrastructure Development Project (GFRIDP)
16	Bipul Chandra Banik	Local Government Engineering Department, Project Director (UDIP, KBS)
17	Md. Kamrul Ahsan	Local Government Engineering Department, Project Director, Barisal Division Rural Infrastructure Development Project (BDRIDP)
18	Md. Jashim Uddin	Local Government Engineering Department, Project Director, Bangladesh Agricultural Infrastructure Development Program (BAIDP)
19	Md. AKM Lutfur Rahman	Local Government Engineering Department, Project Director, Coastal Climate Resilient Infrastructure Project (CCRIP)
20	Gopal Chandra Debnath	Local Government Engineering Department, Project Director, Haor Infrastructure and Livelihood Improvement Programme (HILIP)
21	Gopal Krisna Debnath	Local Government Engineering Department, Project Director, Small Scale Water Resources Developmemt Project (SSWRDP)
22	Sachin Chandra Halder	Local Government Engineering Department, Project Director (Patuakhali-Barguna)
23	Mahbub Imam Morshed	Local Government Engineering Department, Assistant Chief Engineer
24	Md. Nazrul Islam	Local Government Engineering Department, Deputy Project Director (Large Bridges Construction)
25	Md. Zahidur Rahman	Local Government Engineering Department, Executive Engineer (Design)
26	Md. Azherul Islam	Local Government Engineering Department, Executive Engineer (Design)
27	Md. Abadat Ali	Local Government Engineering Department, Executive Engineer (Design)
28	Md. JM Azad Hossain	Local Government Engineering Department, Executive Engineer, Third Primary Education Development Programme (PEDP III)
29	Md. Tarikuzzaman	Local Government Engineering Department, Executive Engineer (Quality Control)
30	Md. Abdur Rahim	Local Government Engineering Department, Executive Engineer (Quality Control)
31	Syed Abdur Rahim	Local Government Engineering Department, Executive Engineer (Maintenance)
32	Abdul Monzur Md. Sadeque	Local Government Engineering Department, Executive Engineer (Planning)

#	Name	Organisation & Designation
33	Mahbub Alam	Local Government Engineering Department, Executive Engineer, Third Primary Education Development Programme (PEDP III)
34	Md. Maksalim	Local Government Engineering Department, Executive Engineer, Third Primary Education Development Programme (PEDP III)
35	Ripon Hore	Local Government Engineering Department, Assistant Engineer, Research & Development
36	Partha Kumar Sarker	Local Government Engineering Department, Assistant Engineer, Quality Control
37	Manas Mondal	Local Government Engineering Department, Assistant Engineer, Chief Engineer's Office
38	Sheikh Anisur Rahman	Deputy Project Director (Emergency Cyclone Recovery and Restoration Project, Local Government Engineering Department)
39	Md. Khorshed Alam	Local Government Engineering Department
40	Md. Enamil Hoque Khan	Local Government Engineering Department, Sr. Assistant Engineer, Quality Control
41	Md. Anwarul Islam	Local Government Engineering Department
42	Khan Md Rabiul Alam	Local Government Engineering Department, Media Consultant, Second Rural Transport Improvement Project (RTIP-II)
43	AKM Mostofa Morshed	Local Government Engineering Department
44	Md. Zakir Hossain	Local Government Engineering Department
45	Dr. Mehedi Ahmed Ansary	Bangladesh University of Engineering and Technology
46	Dr Abdullah Al Mamun	Roads and Highways Department, Director, Road Research Laboratory
47	Dr Jiban Kumar Sarker	Bangladesh Water Development Board, Executive Engineer (Design)
48	Salma Akter	Roads & Highways Department, Executive Engineer, Road Research Laboratory
49	Engr. Jiban Krishna Saha	Second Rural Transport Improvement Project (RTIP-II), Quality Assurance Specialist (Field Level Official)
50	Mostadar Rahman	Senior Consultant, Design
51	Ahmed Nawaz	Municipal Government and Services Project (MGSP), Deputy Team Leader
52	Ahsan Habib	Euroconsult Mott MacDonald, Sustainable Rural Infrastructure Improvement Programme, Deputy Team Leader
53	Mohammad Imran	Euroconsult Mott MacDonald, Design & Construction Engineer
54	MD Shamsul Islam	Euroconsult Mott MacDonald, Khulna Clay Project, Deputy Team Leader
55	Gazi A Rahmani	Euroconsult Mott MacDonald, Senior Business Development Advisor
56	Abu Fattah	Local Government Engineering Department, Media Consultant
57	Ian Duncan	Euroconsult Mott MacDonald, Khulna Clay Project, Team Leader
58	Richard Lebon	Euroconsult Mott MacDonald, Khulna Clay Project, Project Manager
59	Abdullah Al Baky	Euroconsult Mott MacDonald, Khulna Clay Project, Research Associate

Example Design Scenario presented at Workshop on 20-09-17

Example Design Scenario – new approach embankment

Ian Duncan

Approach embankment

New-build

Scenario: -

Rural road

New river crossing

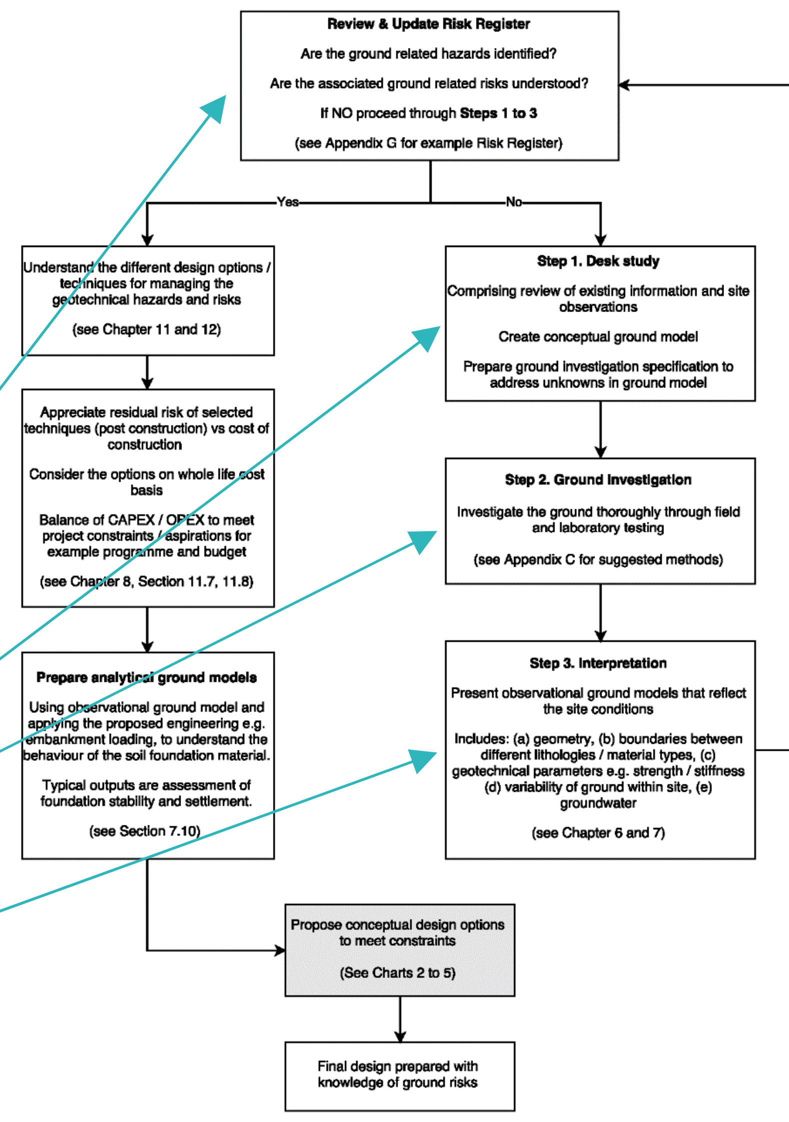
Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

4m high embankments

- Consider the ground related hazards & associated risks
- Undertake initial studies and investigations
- Develop the ground model

Chart 1: Design Phase Process Diagram (based on Figure 16)



Approach embankment

New-build

Scenario: -

Rural road

New river crossing

Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

4m high embankments

- Consider the ground related hazards & associated risks
- Undertake initial studies and investigations
- Develop the ground model

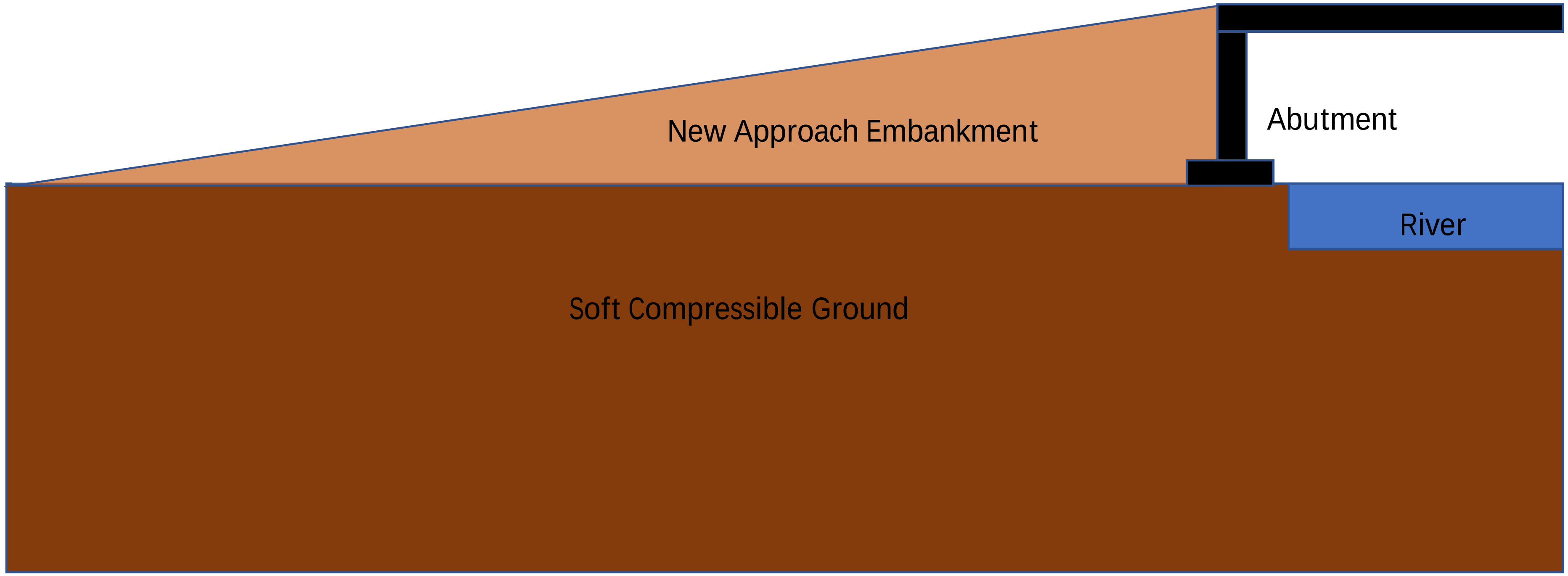
- What are the principal hazards?
  - Soft ground
- What are the potential consequences of constructing the bridge and embankments?
  - Deformation, instability, structural damage
- How are the hazards recorded and managed?
  - Geotechnical Risk Register
- What studies and investigations should be carried out to mitigate risks?
  - Walkover, intrusive investigation, lab testing
- What is the most effective way to represent the hazards, constraints and geological conditions
  - A Ground Model

Risk Register

Table 41: Geotechnical Risk Register

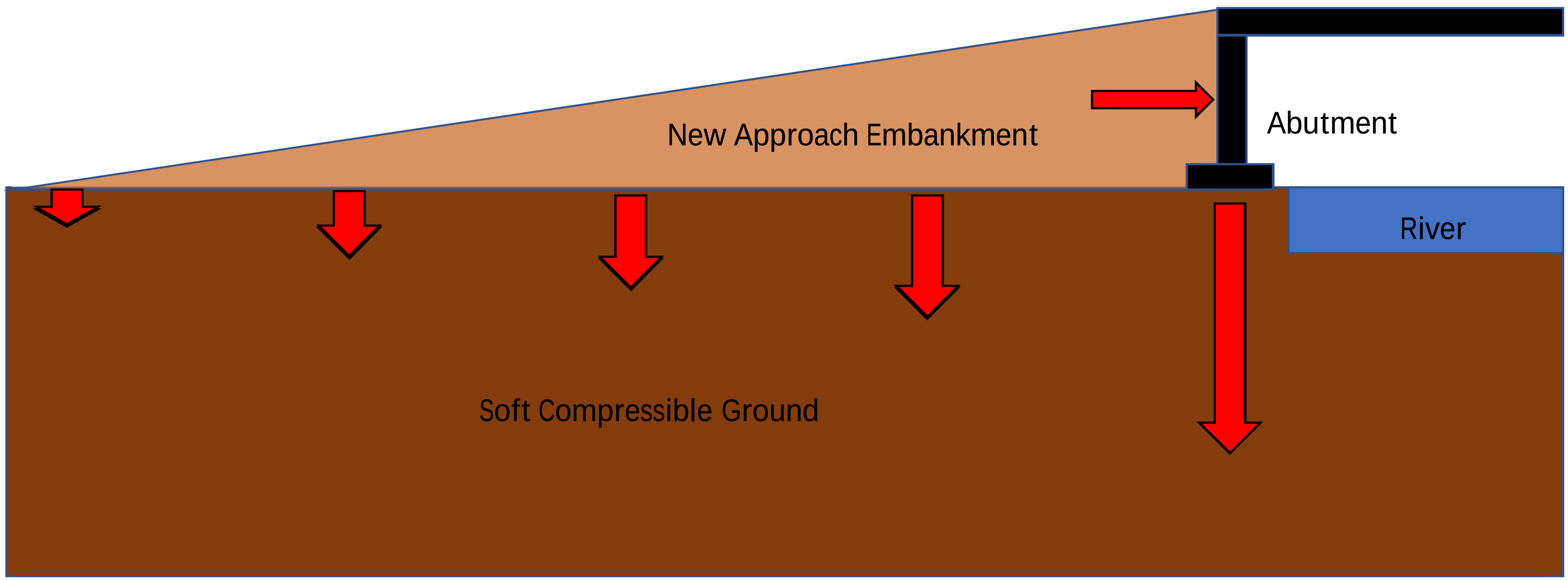
Hazard	Consequence	Impact	Likelihood	Current Risk	Risk Type	Potential Control Measures	Impact	Likelihood	Residual risk
Alluvium Deposits	Variable lithologies of poor engineering quality. Soft, compressible soils (in places), variable thickness.	4	3	H	C,T,H&S,E.	Detailed ground investigation and associated geotechnical laboratory testing to allow a detailed ground model and set of parameters be determined for use within design.	3	2	A
Embankment Fill materials	Variable lithologies and engineering properties.	3	2	A	C,T,H&S,E.	Detailed ground investigation to identify any areas of the Embankment Fill (Made Ground) and associated properties.	2	2	N
Inadequate ground investigation.	Unforeseen ground conditions, inappropriate design parameters	4	4	S	C,T,H&S,E.	Conduct a ground investigation based on a detailed desk study	2	1	N
Changes in the groundwater and flooding conditions.	Detrimental effect on earthworks stability y.	3	1	N	C,T.	Monitoring of groundwater levels during and after ground investigation.	3	1	N
Lack of suitable material for earthworks on site.	Excessive import of acceptable materials and / or disposal of unacceptable onsite materials.	3	4	H	C,T,H&S,E.	Schedule appropriate earthworks acceptability testing as part of the ground investigation. Programme earthworks into a season with favourable weather. Consider improvement of onsite soils. Monitoring and testing of soils throughout earthworks.	2	2	N

Ground Model



* Drawing not to scale

Ground Model – load from earthworks and abutment



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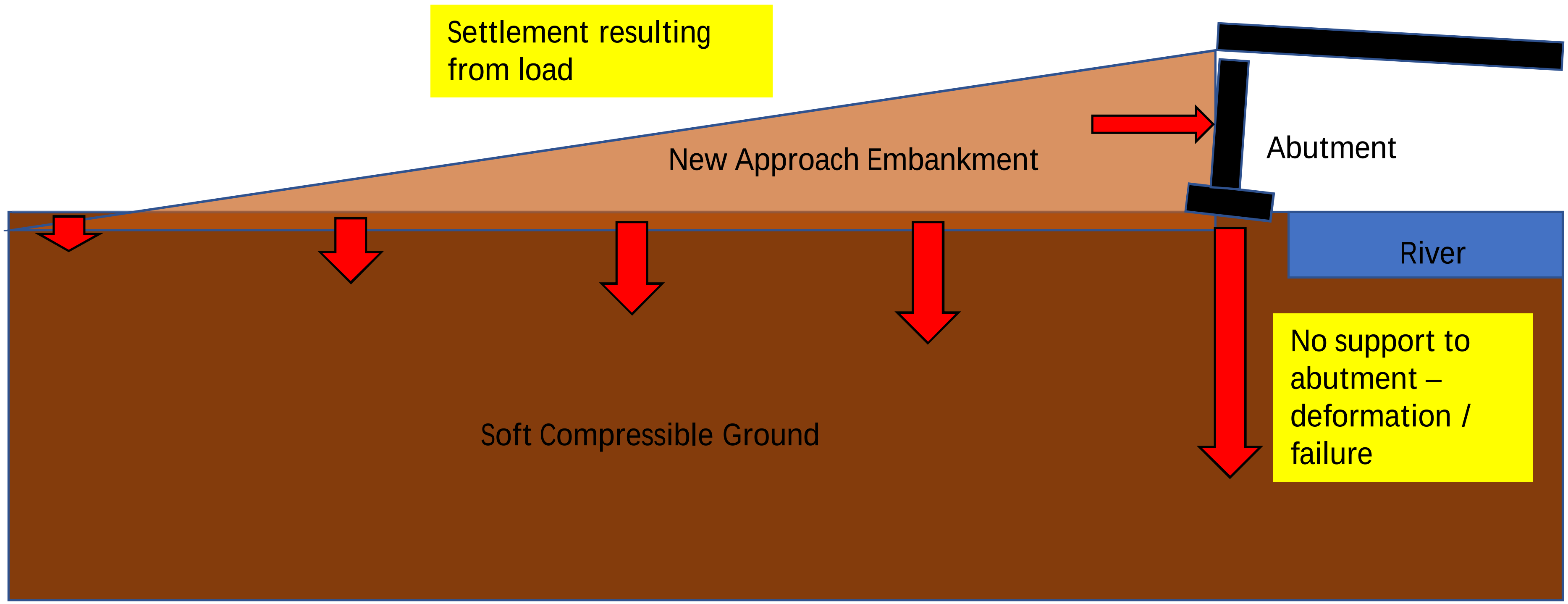
Ground Model – ground response to load – settlement, failure



* Drawing not to scale

Approach embankment

New-build

Scenario: -

Rural road

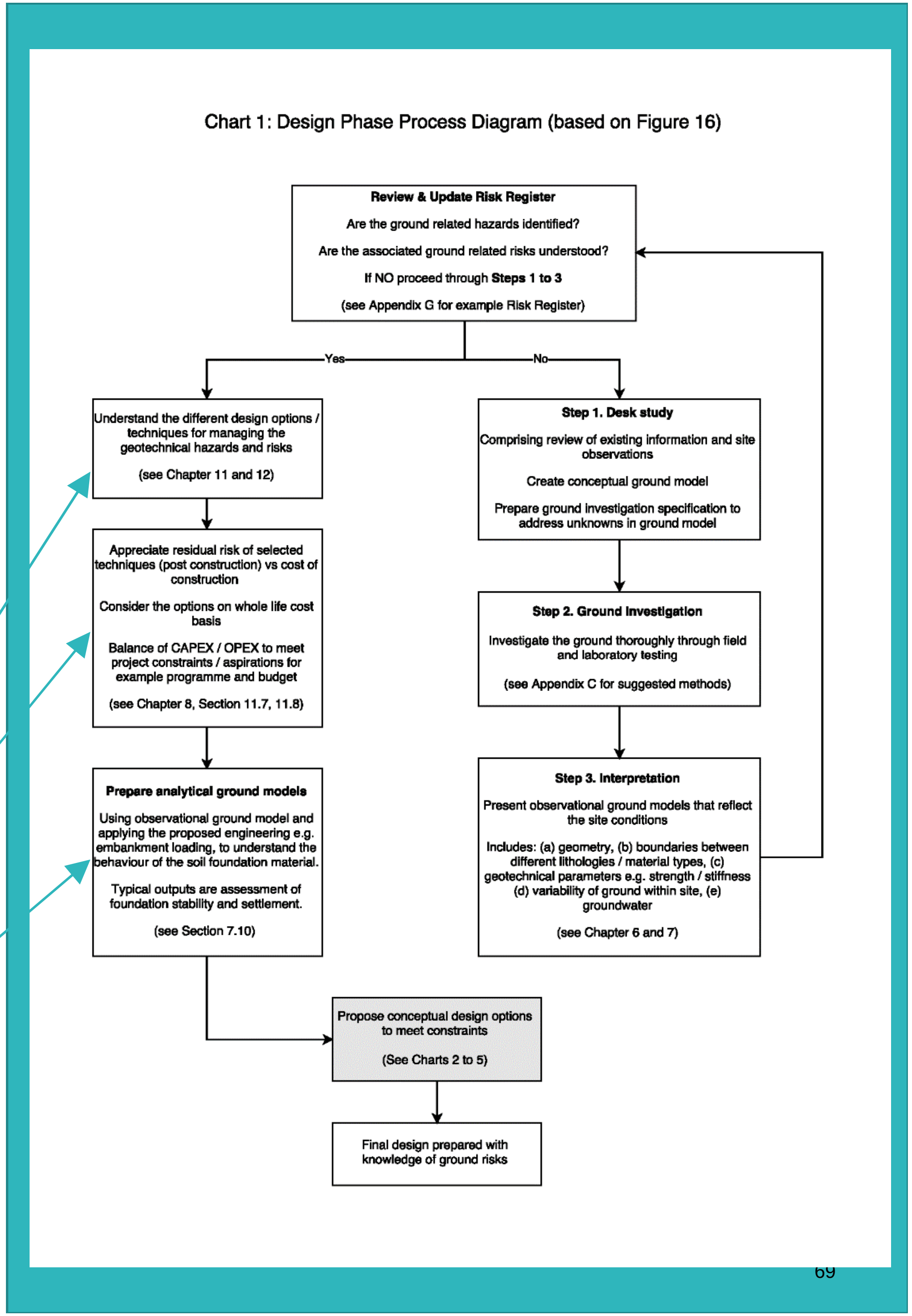
New river crossing

Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

4m high embankments

- Different options / techniques?
- Consider the residual risks
- Whole life cost – balance of capital and maintenance budget



Approach embankment
New-build

Scenario: -

Rural road

New river crossing

Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

4m high embankments

- Different options / techniques?
- Consider the residual risks
- Whole life cost – balance of capital and maintenance budget

- What are the likely design options?
  - Piles for abutment
  - Earthworks – surcharge, drains, geotextile
- What constraints are applicable to the methods?
  - Programme
  - Complexity / skills
  - Cost
  - Materials, plant
- What are the residual risks of these methods?
  - Long term movement
  - Damage to piles
  - Differential movement

Approach embankment

New-build

Scenario: -

Rural road

New river crossing

Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

4m high embankments

- Prepare analytical model based on the investigation and proposed engineering

- What are the inputs for analytical model?
  - Engineering Geometry / loadings
  - Geological boundaries
  - Geotechnical properties of foundation soil – which?
- What are the key outputs of the analytical model?
  - Limit states (ultimate / serviceability)
  - Amount of deformation
  - Rate of deformation
  - Stable / unstable

Approach embankment

New-build – Build Embankment First

Scenario: -

Rural road

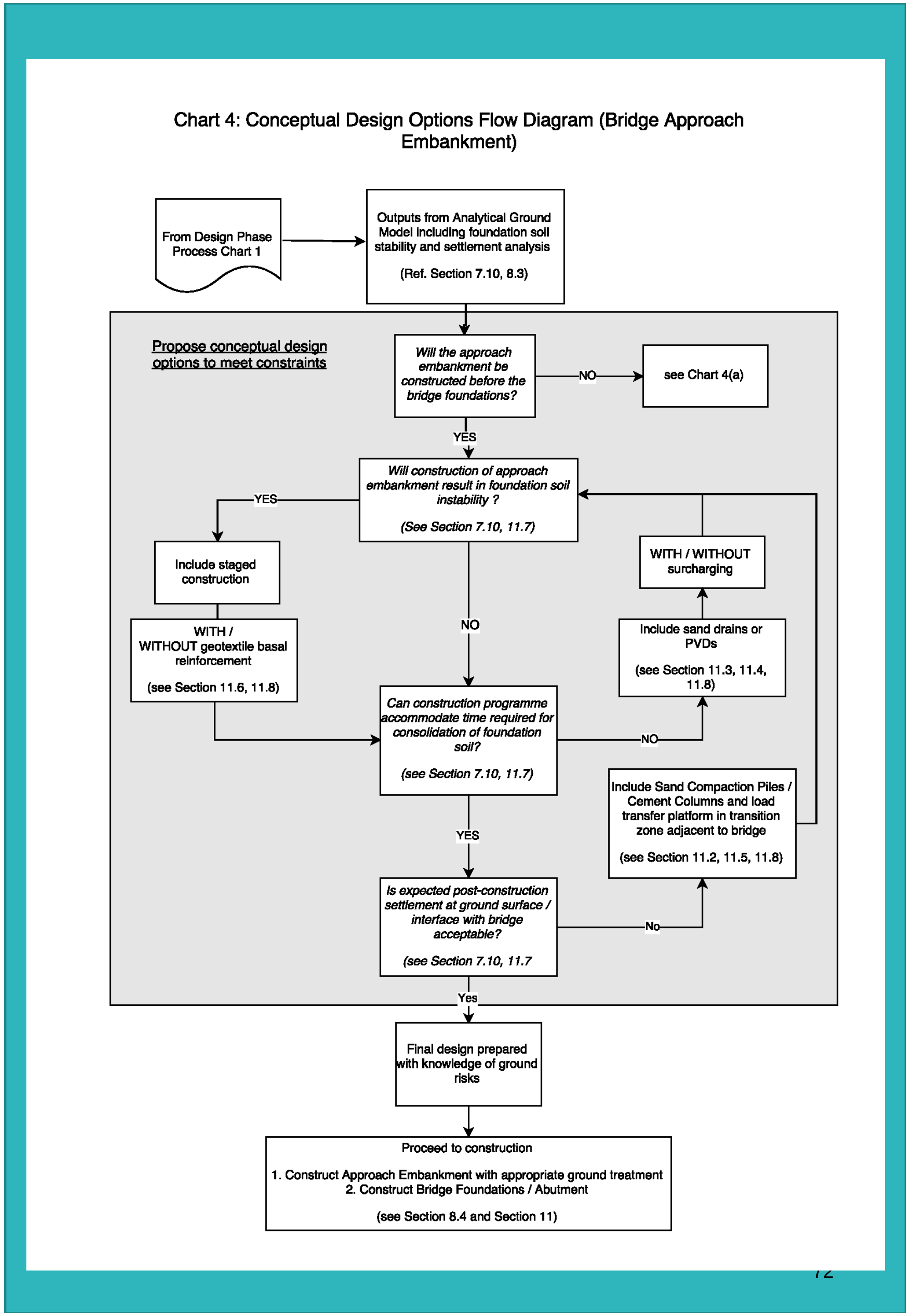
New river crossing

Selected engineering – single span bridge with abutments and earthwork approach ramps

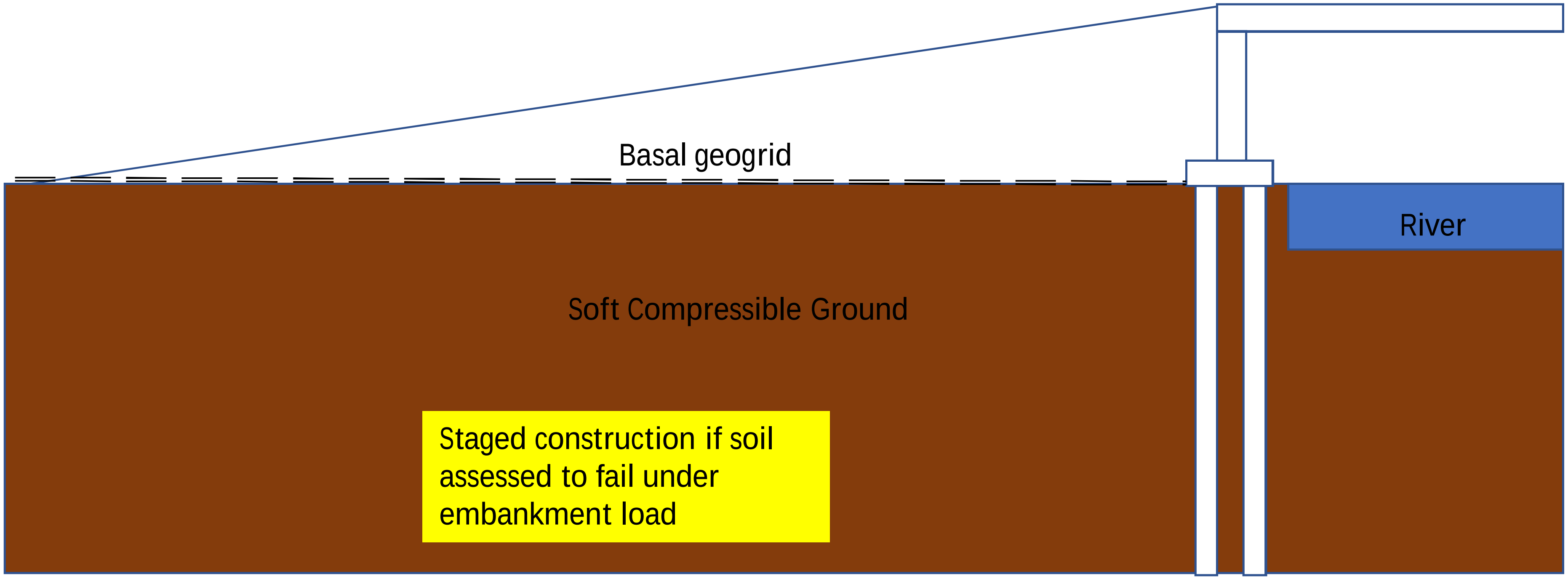
20m + soft ground

4m high embankments

- Select the appropriate Chart and work through the decision making process
- Embankment constructed before / after bridge (Chart 4 / Chart 4(a))

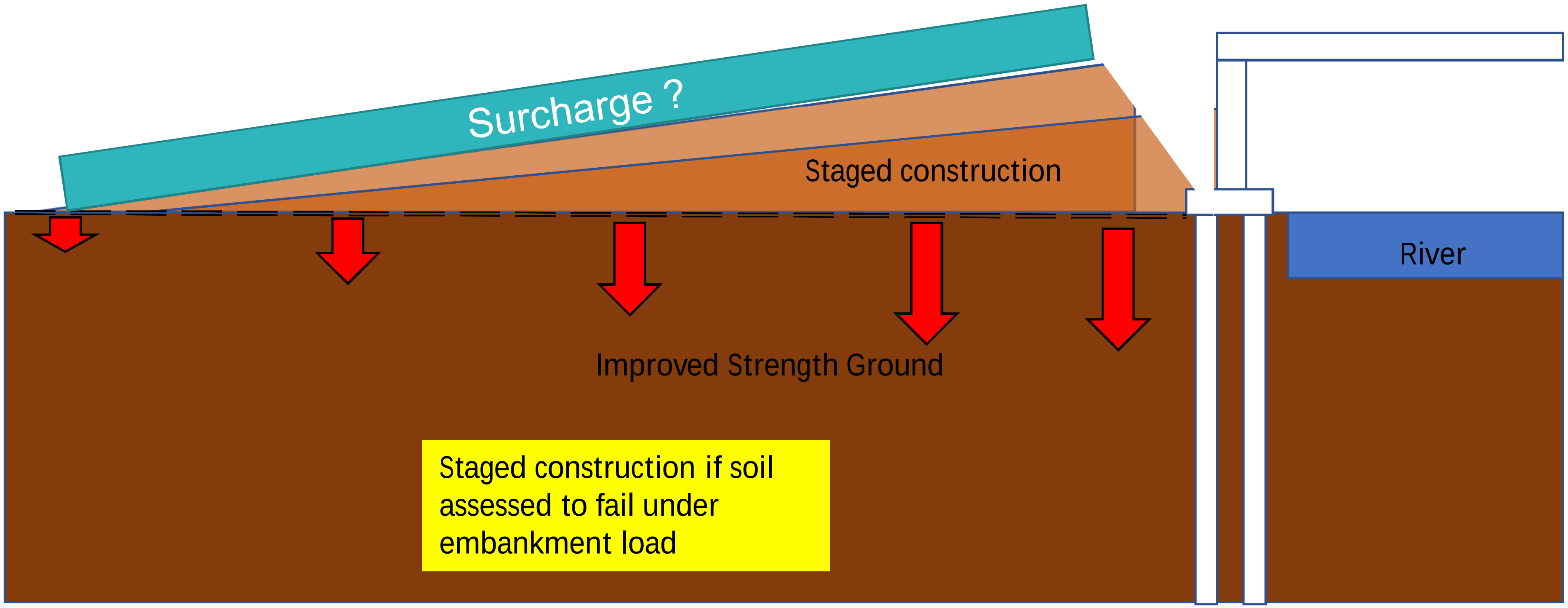


Build embankment first – staged construction with basal geogrid (+ surcharge); support abutment with piles



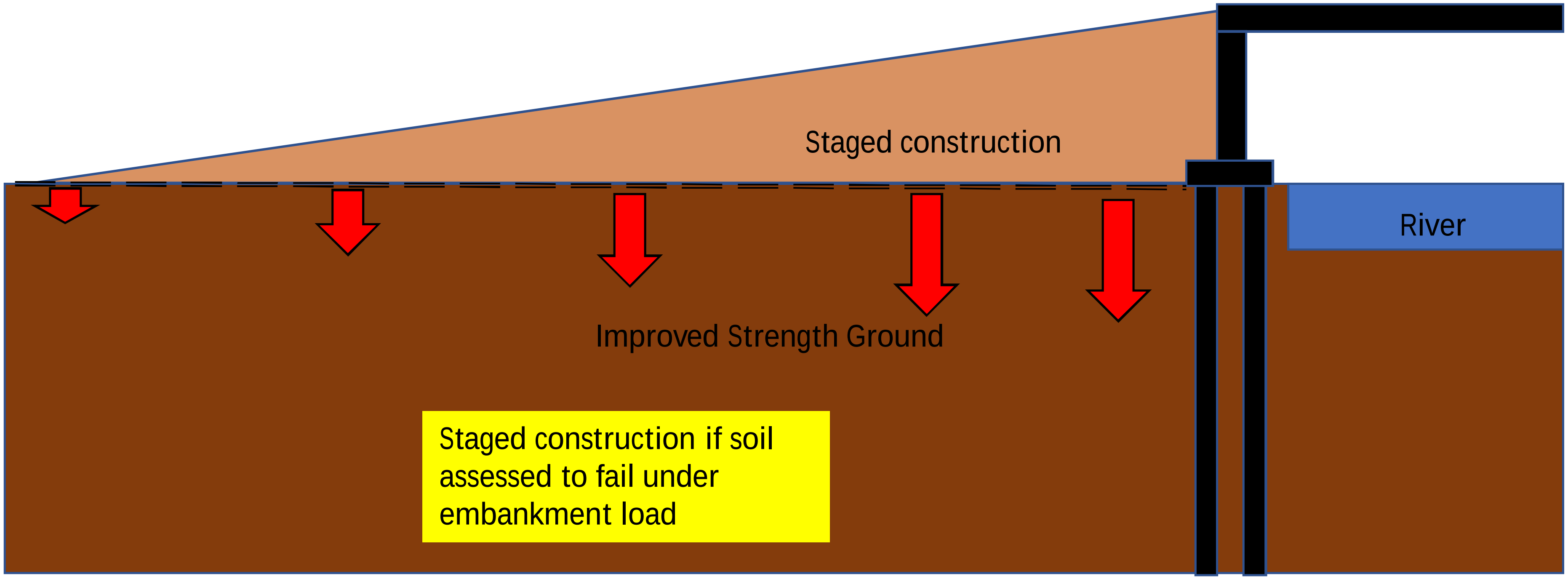
* Drawing not to scale

Build embankment first – staged construction with basal geogrid (+ surcharge); support abutment with piles



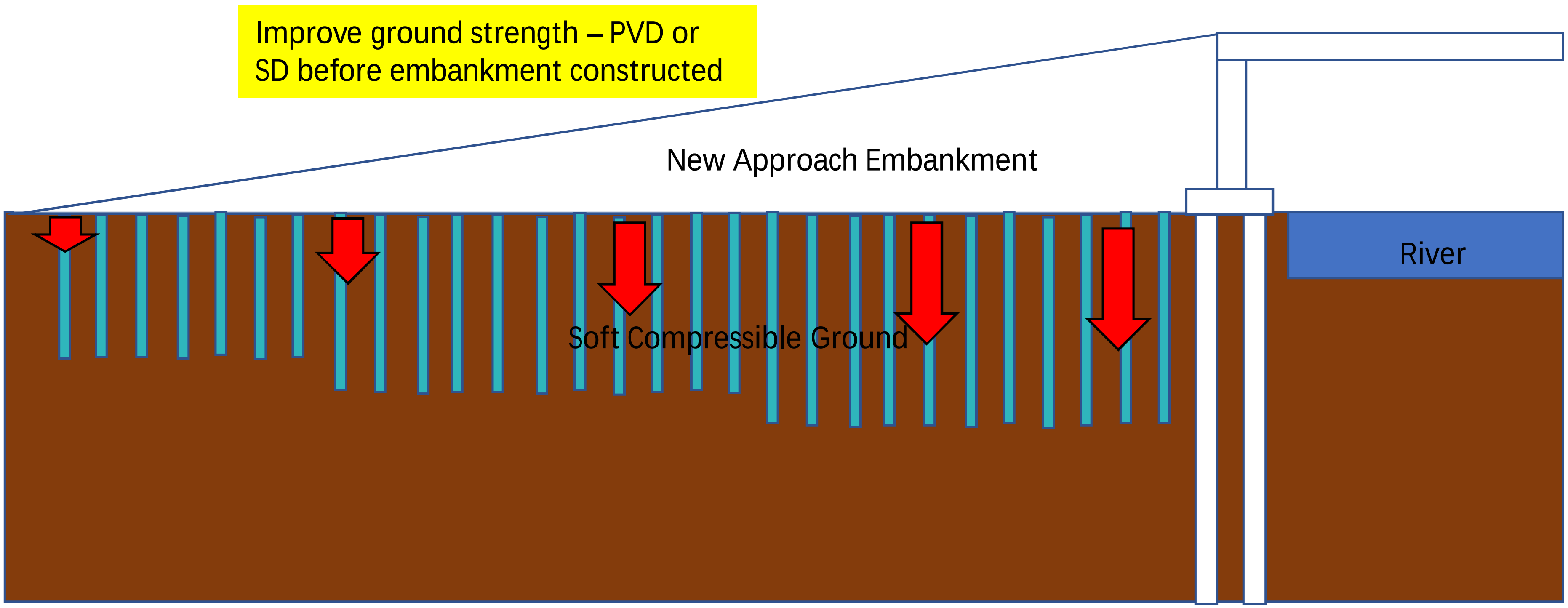
* Drawing not to scale

Build embankment first – staged construction with basal geogrid (+ surcharge); support abutment with piles



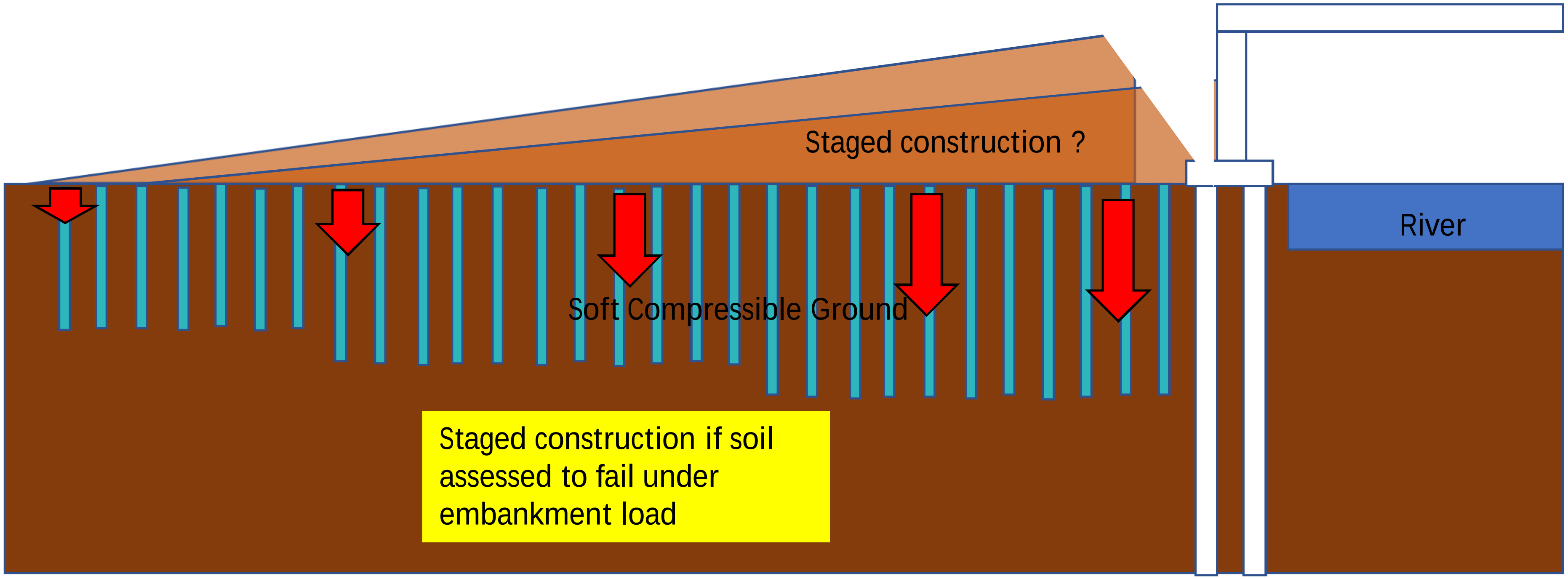
* Drawing not to scale

Build embankment first – with vertical drains (+ surcharge); support abutment with piles



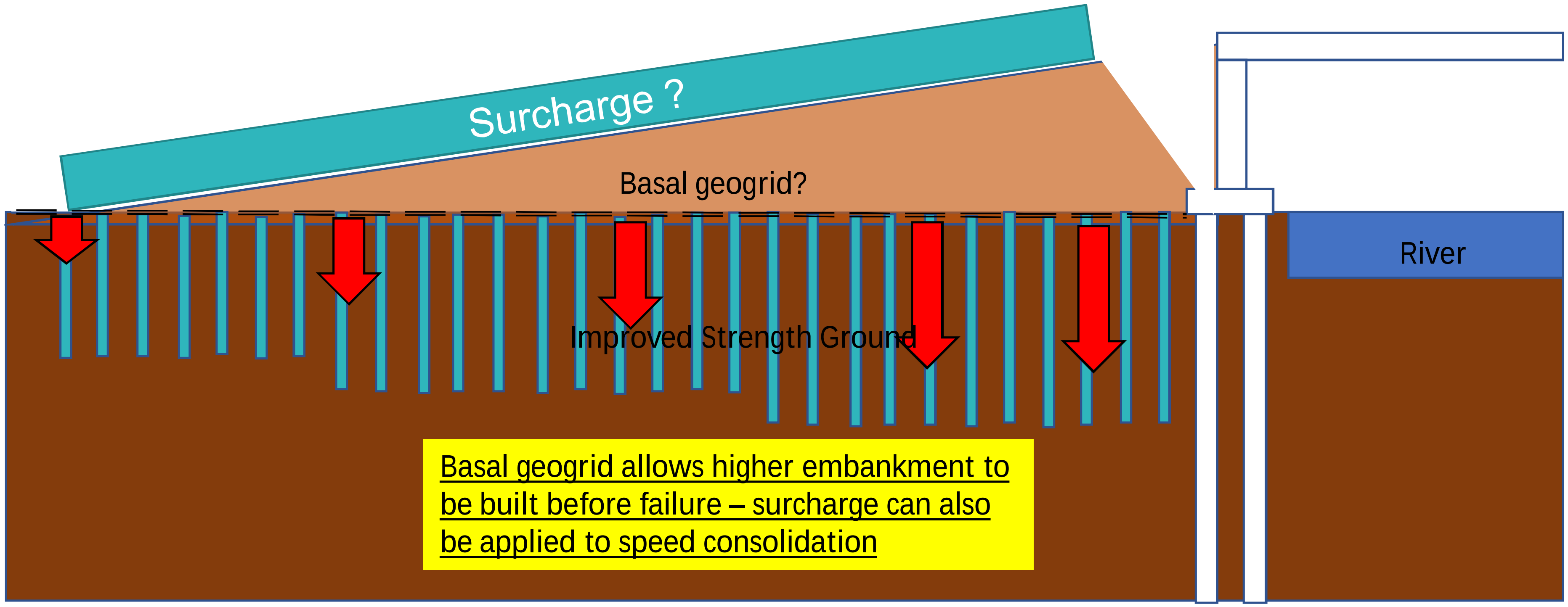
* Drawing not to scale

Build embankment first – with vertical drains (+ surcharge); support abutment with piles



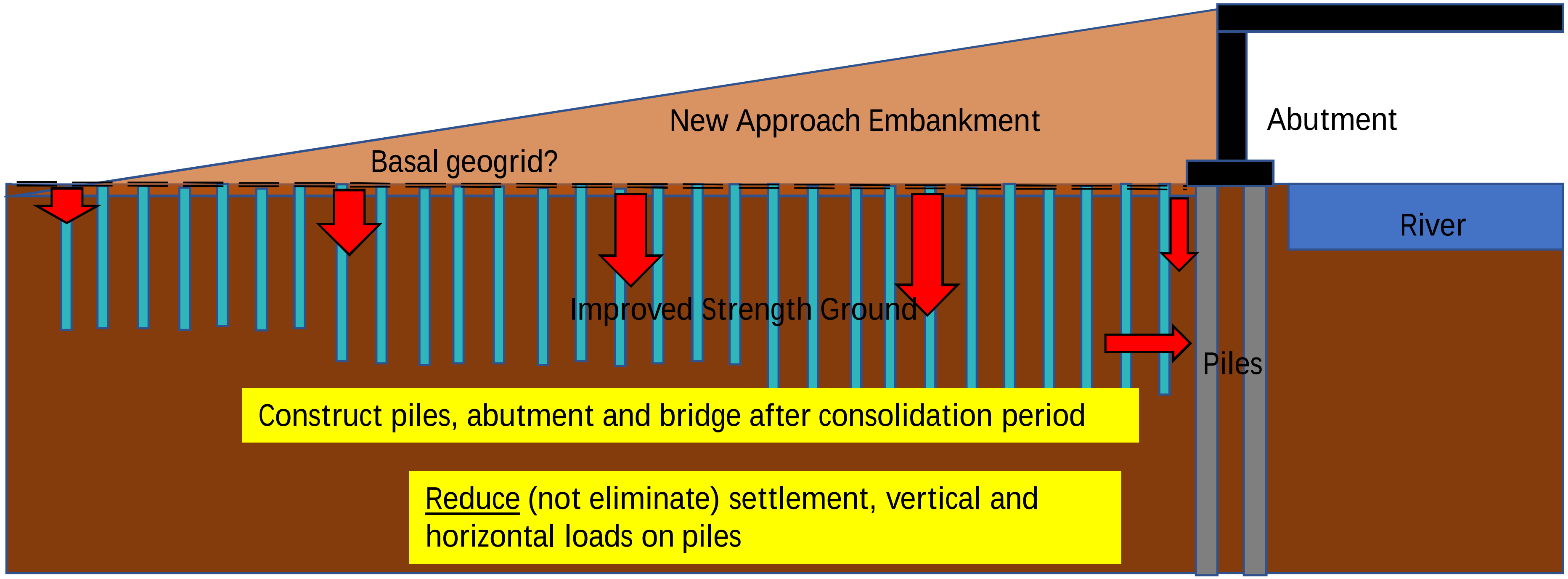
* Drawing not to scale

Build embankment first – with vertical drains (+ surcharge); support abutment with piles



* Drawing not to scale

Build embankment first – with vertical drains (+ surcharge); support abutment with piles



* Drawing not to scale

Approach embankment

New-build – build bridge first

Scenario: -

Rural road

New river crossing

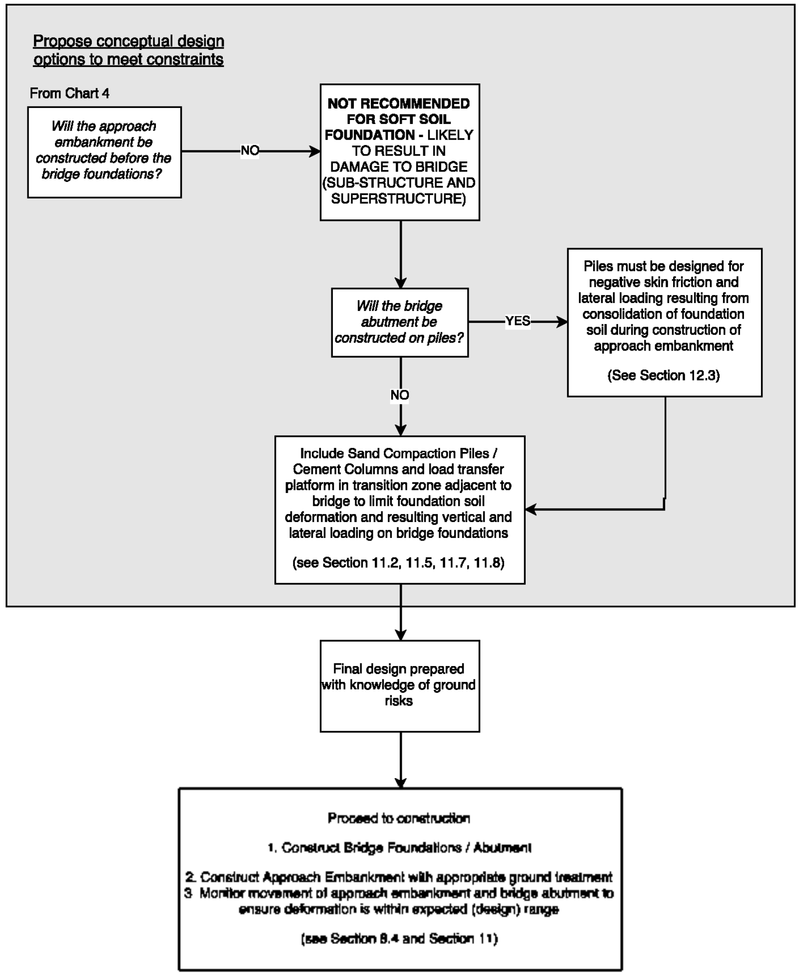
Selected engineering – single span bridge with abutments and earthwork approach ramps

20m + soft ground

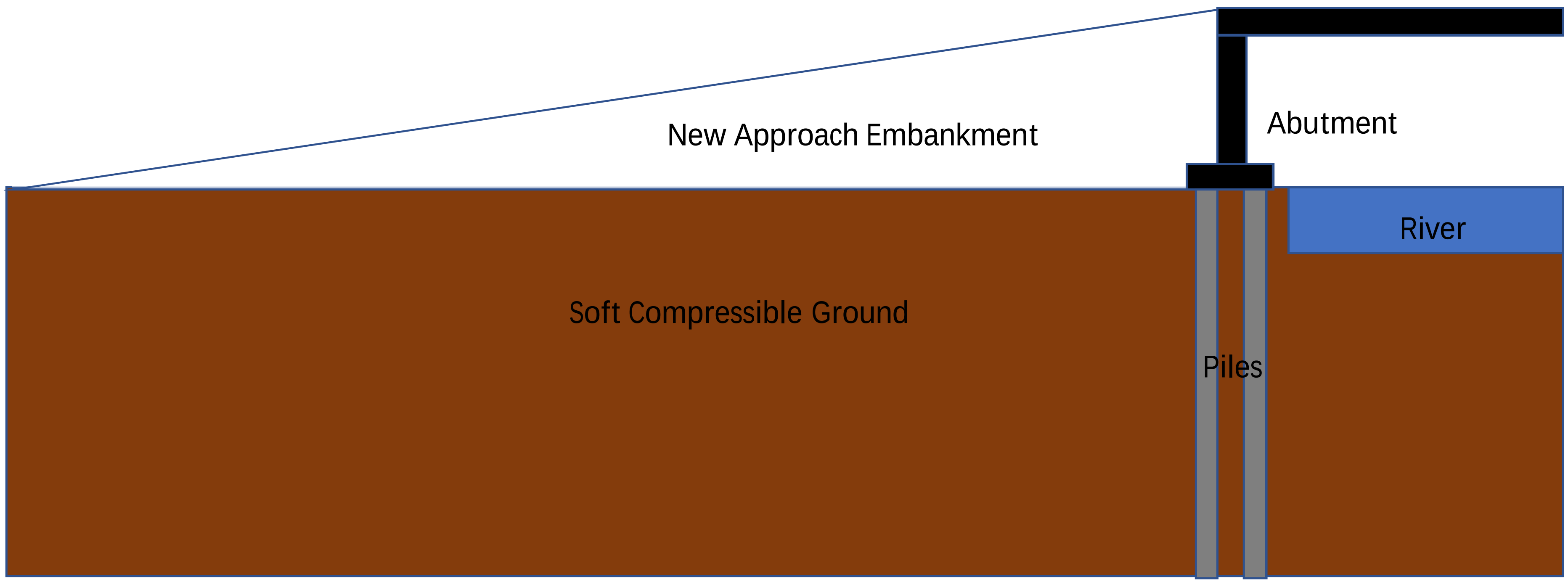
4m high embankments

- Select the appropriate Chart and work through the decision making process
- Embankment constructed before / after bridge (Chart 4 / Chart 4(a))

Chart 4(a): Conceptual Design Options Flow Diagram (Bridge Approach Embankment - approach constructed after bridge)

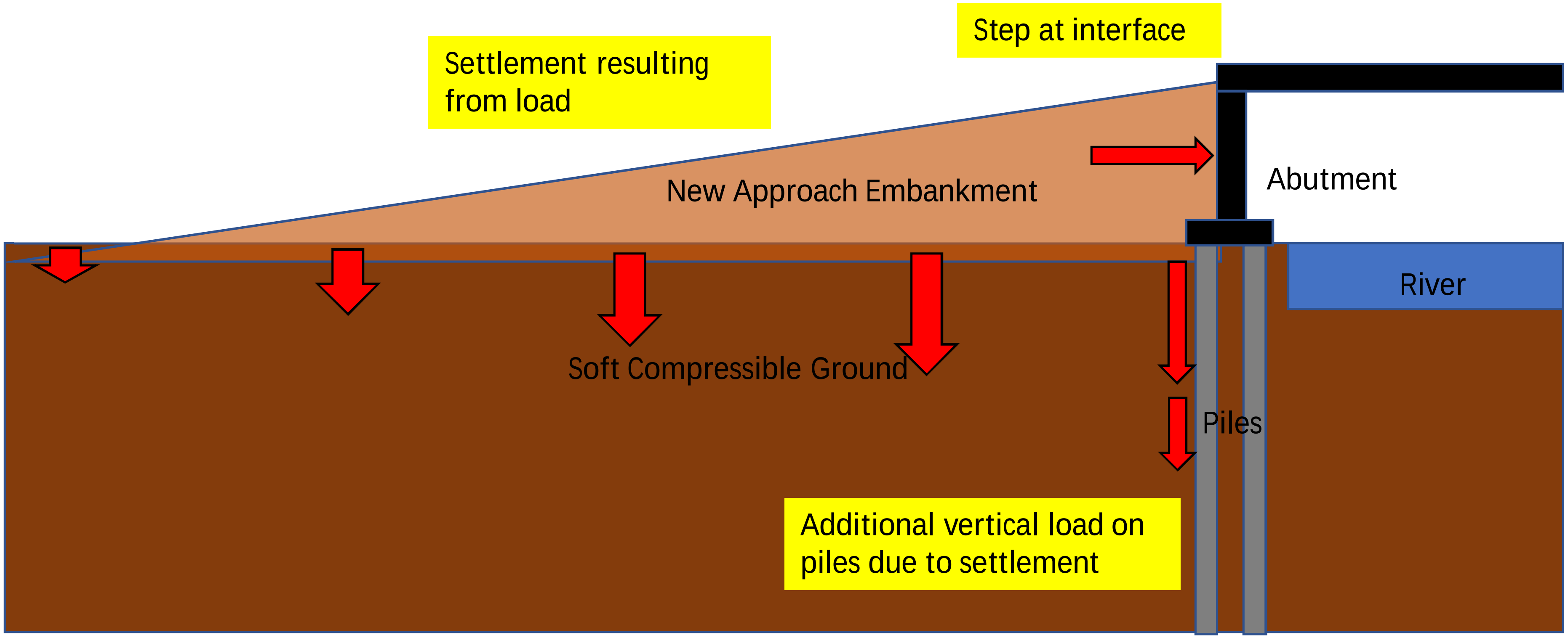


Bridge built first – support abutment with piles



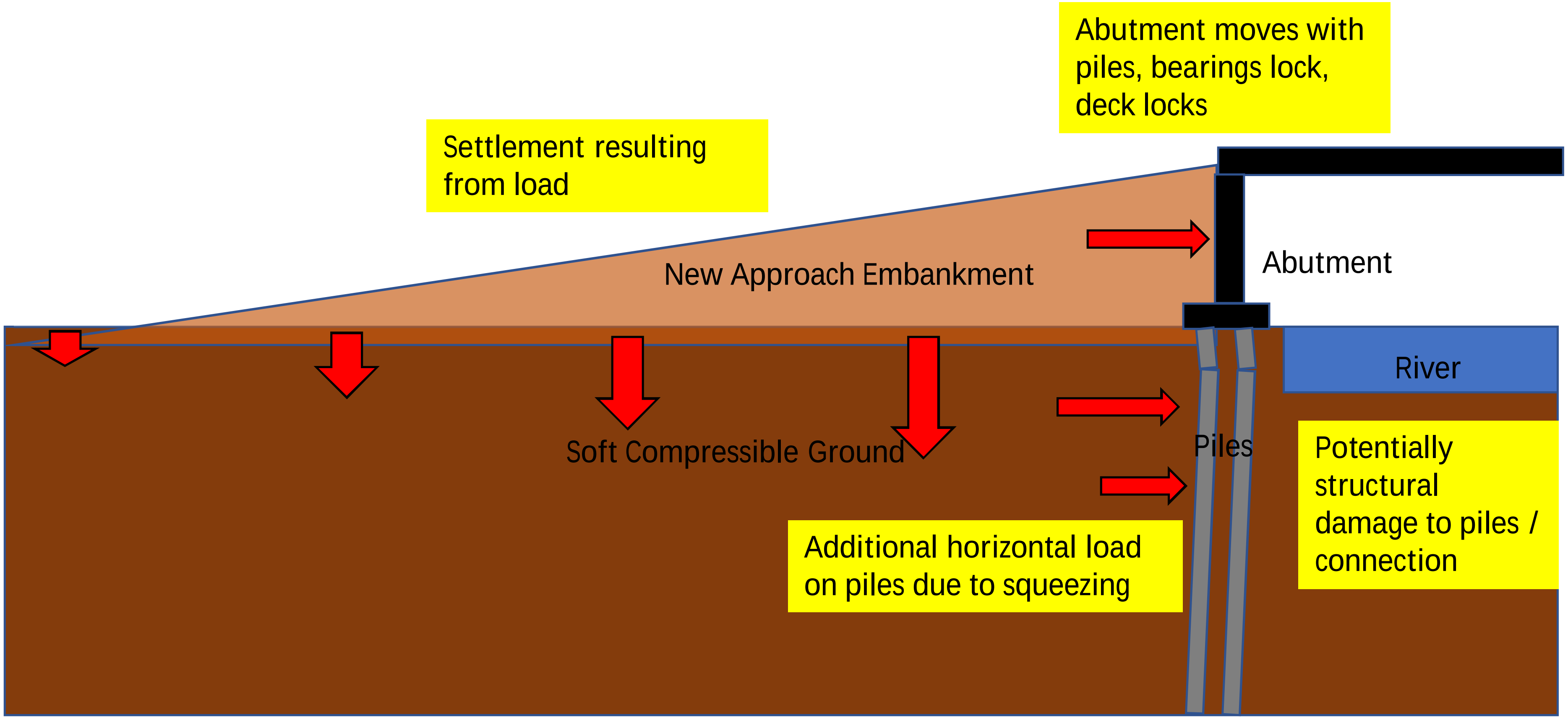
* Drawing not to scale

Bridge built first – support abutment with piles – can result in serious damage



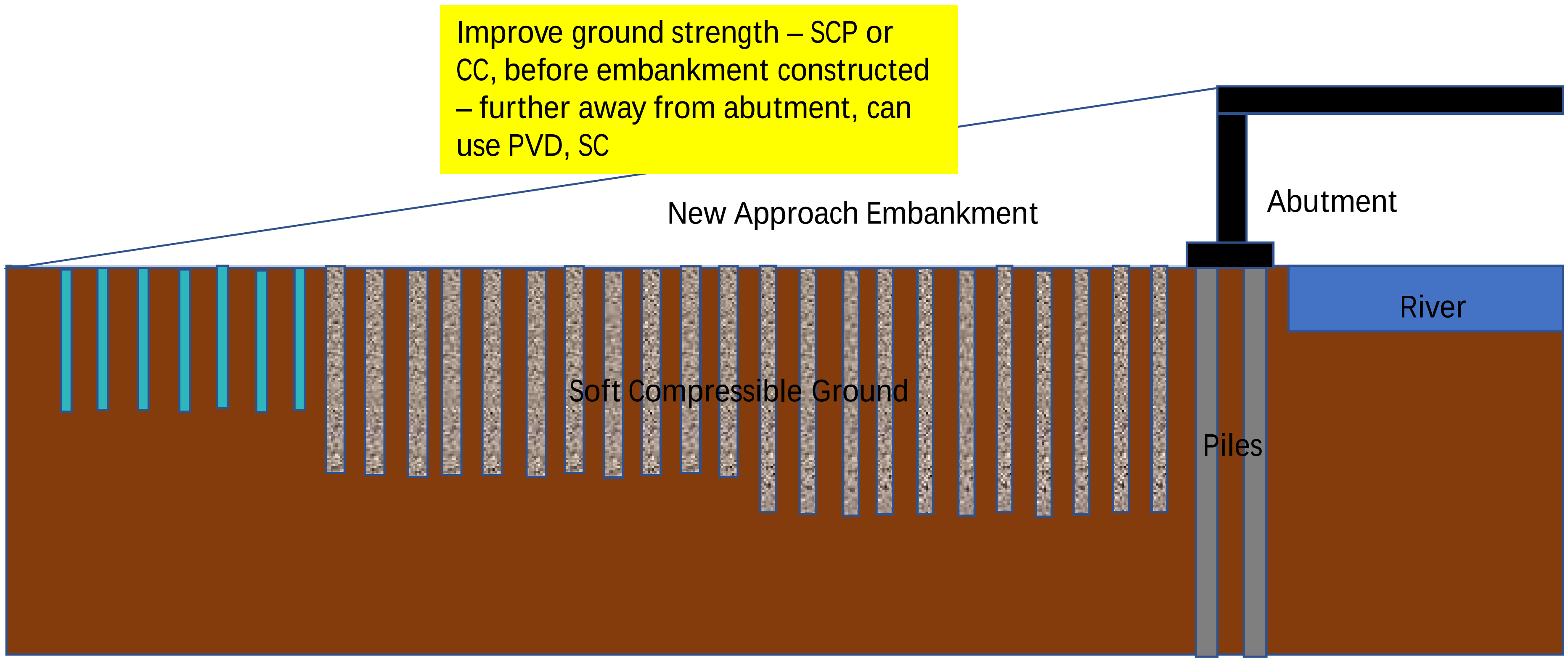
* Drawing not to scale

Bridge built first – support abutment with piles – can result in serious damage



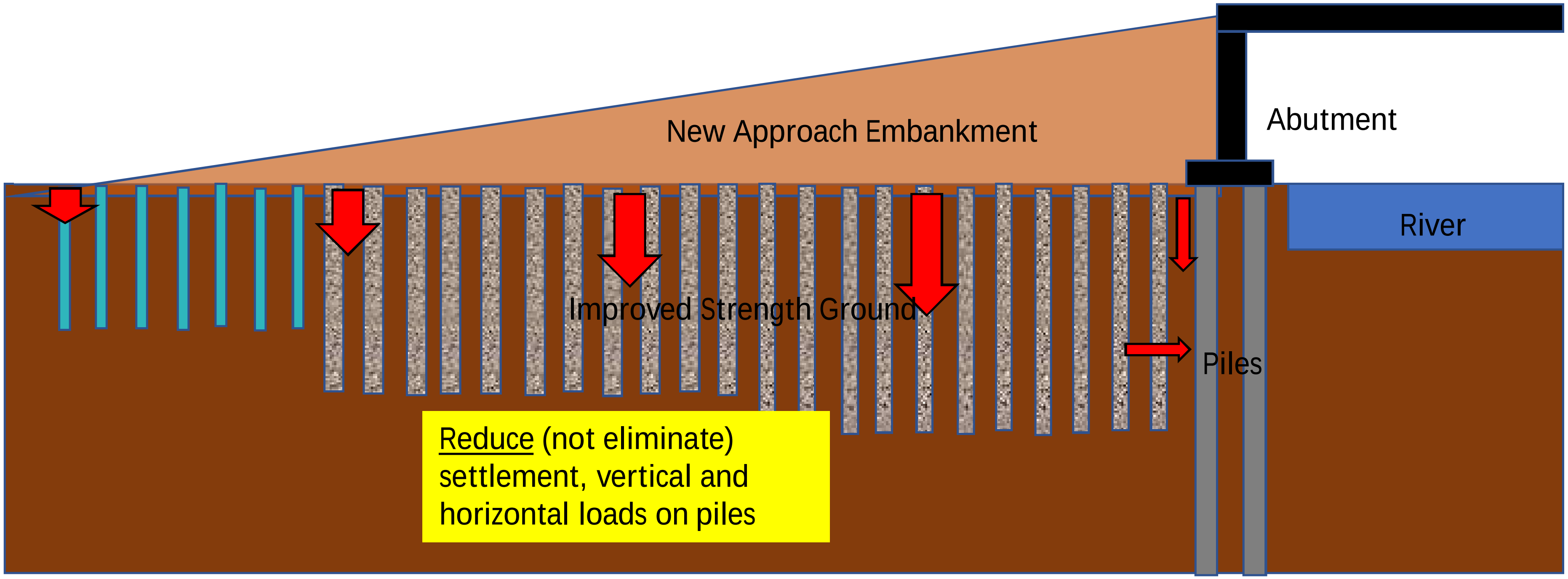
* Drawing not to scale

Build bridge first – support abutment with piles + support embankment with SCP or CC



* Drawing not to scale

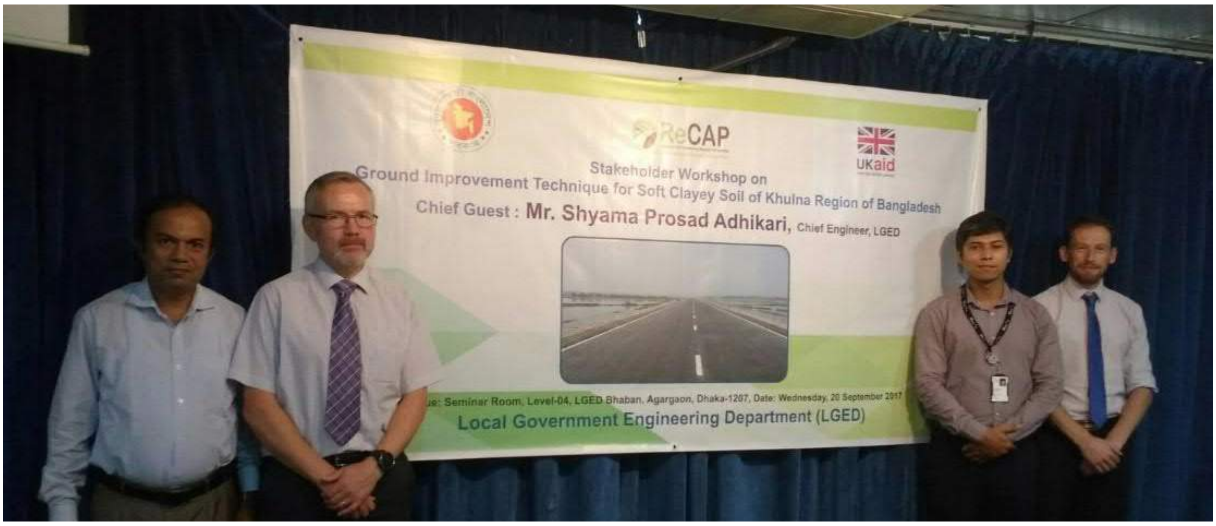
Build bridge first – support abutment with piles + support embankment with SCP or CC



* Drawing not to scale

Photos

A selection of photos from the Stakeholder workshop are shown below.



Feedback

The key issues and comments arising in the open floor discussion are summarised below. In inaugurating the working session, Mr. Shyama Prosad Adhikari, Chief Engineer, LGED, highlighted the effects of climate change and flood damage on many kilometres of rural roads, and requested continued support in developing cost-effective technical solutions. Highlighting the role of LGED's rural roads research fund, Mr. Adhikari emphasised the importance of bridging the gap between research and field-level implementation, and of applied research, where infrastructure investments are more cost-effective if research is applied.

For each comment or topic arising during this working session, a summary of the response provided either during the floor discussion or by inclusion and/or recommendations in the Final report is provided below. Comments from the floor are provided in plain text, and responses from the authors of this report are shown in italics.

- The limitation on comprehensive consolidation testing in rural road budgets.

The consolidation characteristics of the clays / silts / organic material can vary quite considerably based on the nature of the sample tested. The consolidation of the soil below an embankment load affects quite considerable depths, certainly 1-2 times the embankment width and hence covers a large amount of soil. The consolidation test is undertaken on a very small sample that is difficult to extract in an intact state from the ground, even in ideal soil and with high quality plant and experienced operatives. In very soft soil, it is very difficult to obtain (a) a high-quality sample, and (b) transfer the sample to a laboratory without changing the in-situ state. Sufficient number of samples would also need to be undertaken to give a representative number of consolidation test results, such that a statistical approach can be used. The associated budgets are anticipated to be beyond the rural road budget. If only a few tests could be undertaken on samples of dubious quality, it would be more prudent to use conservative 'book' values.

- The availability of potentially useful soil maps.

As part of our review, we identified 'Statistical Evaluation of Bearing Capacity of Khulna Sub-Soil' prepared by KUET. This provide spatial mapping of bearing capacity based on a set of boreholes and test data. This is a good starting place for assessing the likely bearing capacity soils to enable optioneering to take place. Further soil maps based on information that Government and Educational facilities have collated would be very beneficial for assessing likely hazards, and would form part of the Ground Model development and Risk Register presented in the report. Along with Geological maps, aerial photos and other sources of information, these would be invaluable and cost-effective way of establishing engineering solutions before ground investigation takes place.

- The potential cooperation with the Roads & Highways Department in future work.

This is something that the authors of the report would fully support, dependent on the strategy adopted by ReCAP/LGED following the conclusion of the Final Report and adoptions of the recommendations from this project.

- Use of piezocones.

The use of cone penetrometer testing (CPT) or Dutch Cone is very valuable for the rapid assessment of soft foundation soils. CPT is very widely employed in soft ground for profiling and determining geotechnical characteristics and should be used in conjunction with boreholes to calibrate the CPT cone and sleeve resistance measurements. The piezocone variant permits measurement of pore water pressure in the soil and dissipation testing to assess the hydraulic conductivity for use in consolidation assessment. Plant is available in Bangladesh as noted in Appendix C of the Final Report, but is likely to have limited availability, and hence be more costly than other methods. Increased use of the method would bring the cost down.

- Implications of whole life costing for rural road embankments on soft soil.

It was considered from the floor that the solutions recommended under this project would prove more costly than current techniques. However, the responses from around the floor emphasised the importance of considering asset life cycle cost in the design process – i.e. where a greater investment up front would result in longer-lasting infrastructure and reduced costs in maintenance and reconstruction. As outlined in Chapter 7 of the Final Report, whole life-cycle approach is fundamental to the project aims. The consideration of whole life costing is included in the flow charts provided for the decision-making process in designing solutions for embankments on soft soil.

- In the final stages, the floor discussion focussed on the collective identification of the way forward

for application and uptake of the report. These discussions are summarised in the conclusions of the final report (see chapter 12).

In addition to the discourse outlined above, a comments sheet was distributed to all participants, focussing on four key questions surrounding the project aims, objectives and presented results. The questions, comments and responses to these comments are copied below:

Question 1 Which of the ground improvement techniques do you regard as most suitable for application to rural roads and why?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	Summary of 17 No. responses received for Question 1. The attendees agreed that the methods recommended were suitable, but needed to be considered on a site by site basis and on depended on the importance of the road / structure. The familiar Sand Drain is a popular choice with many, as is the Sand Compaction Pile. The use of basal geogrids in combination with Sand Drains or prefabricated vertical drains (PVD) / surcharge is also considered suitable as an economic and simple method. Excavate and replace is mentioned by one attendee, and the use of grouting is suggested in addition to the methods recommended in the report. We consider that grouting is not suitable for rural road applications. Soil stabilisation with cement and sand is noted by one attendee and we consider this an area for future research as detailed in Chapter 11 of the Final Report. Cement columns are not a popular choice, based on cost. This also may be due to knowledge of technique, availability of plant and skills to operate.

Question 1 Which of the ground improvement techniques do you regard as most suitable for application to rural roads and why?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	Individual Responses received for Question 1:
	Basal Reinforcement with surcharge because suitable for construction due to simple and cost effective.
	Geo-Textile basal reinforcement and PVD with surcharge are the most suitable methods for ground improvement techniques. Comparatively they are easy to implement and provide more sustainable benefit etc.
	Sand drain with surcharge. (sand pile/PVD).
	Sand compaction for increasing soil density.
	Removal of soft soil and pouring dense soil.
	Sand compaction pile, easy to construct, cost effective.
	I think Geotextile/ Geogrid should be a economic and suitable solution for ground improvement.
	Other solution like sand drains, cement column are costly as well as time consuming.
	Basal reinforcement can be used in rural road because it is not expensive.
	Sand-drain, basal reinforcement and surcharge.
	Sand compaction pile
	Geo-Textile raft This is assumed to relate to basal geogrid, covered in the Final Report
	Soil stabilisation with cement, sand and clay
	Vertical sand drain (sand column) placing is most suitable. Instruments are available, easy to implement and very suitable for quick settlement.

Question 1 Which of the ground improvement techniques do you regard as most suitable for application to rural roads and why?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	It will depend on the importance of the road. If it is in high economic zone, cement column can be provided. Otherwise sand drain is a widely used method.
	Improvement by geo-grouting because it is easier to implement and may be cost effective. Jet grouting is not suitable for the engineering application on rural roads. Jet grouting is very specialised, expensive and requires high levels of skill and site controls to avoid blow-out at surface due to the very high pressures used with this technique.
	Also improve the bearing capacity of sand piling, it is also low cost and easier technique.

Question 2 Are there any additional constraints to the use of these ground improvement techniques that you consider should be incorporated?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
Are there any additional constraints to the use of these ground improvement techniques that you consider should be incorporated?	Local construction material like wooden pole and bamboo to be used in as basal reinforcement. Use of such materials as basal reinforcement is considered in the Final Report, summarised in Table 9.5. Life expectancy of local construction materials is noted as a concern, with implications on the whole life cost of an asset, and where further research is recommended to monitor the effectiveness of these techniques.
	Quality materials and equipment are not available everywhere in Bangladesh. The range of solutions presented and considered in the Final Report have been prepared in cognisance of this concern – it is not the intention of the report to produce a "one-size-fits-all soft soil solution", but to present research and findings on a range of possible solutions, which, supported by the use of the flow charts provided, will assist LGED engineers in choosing the optimum solution for their particular project or problem. The contributing

Question 2 Are there any additional constraints to the use of these ground improvement techniques that you consider should be incorporated?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	factors to selection of ground improvement techniques are summarised in Chapter 7 of the Final Report.
	Attitude to careful construction at the field level. This is indeed highlighted throughout the final report, and above recommendations for the uptake and embedment of findings, where clear design specifications, and good monitoring and quality control throughout the construction phase will contribute to longer asset life-cycle.
	Cement grouting may be a solution. Jet grouting is not suitable for the engineering application on rural roads. Jet grouting is very specialised, expensive and requires high levels of skill and site controls to avoid blow-out at surface due to the very high pressures used with this technique.
	Examples need to be included in the final report. For a particular soil data please prepare design soil sand drain or sand column or cement column etc. Design examples were included in the presentation and have been attached to the workshop report. In the Final Report, further work is recommended to expand the current project remit to include standard designs.
	Main problem with locally available skilled/ semi-skilled workers. The recommendations provided in the final report include preparation of training materials and provision of training to LGED engineers, who would then become the trainers for local engineers and site workers.
	For proper ground improvement, need to know the soil characteristics by accurate soil testing. Another problem is to improve the existing road because the road was not implemented by proper compaction before. The Final Report highlights a range of site investigation testing techniques available in Bangladesh generally and Khulna specifically (Final Report Appendices C and K), at differing levels of

Question 2 Are there any additional constraints to the use of these ground improvement techniques that you consider should be incorporated?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	cost and practicality, from walkover to intrusive investigation, and for application to different scenarios, and within the design process outlined in Flow Chart 1.
	Different ground improvement techniques are shown. But their cost comparison was not provided. Absolute costs for application of the different ground improvement techniques in Bangladesh are not available within the current time available to the project team, but comparative cost implications have been included in section 9.9.

Question 3 What further work would you like to see undertaken in subsequent stages of this project?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	Pilot soil consolidation through same techniques. Further work has indeed been recommended in the Final Report to pilot the proposed solutions and determine how effective these are in real conditions – this will be fed back into design and implementation guidelines.
	Piloting results (actual results) through piloting of the suggested method. See above.
	Should be easily understandable for implementing personnel. Further work is recommended to pilot the proposed solutions and prepare implementation guidelines.
	Some instances that reflects sinking of roads or bridges due to soft foundation soil as reference. Example ground models and sites where defects were observed are shown in Chapters 4 and 6.

Question 3 What further work would you like to see undertaken in subsequent stages of this project?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	For different type of soil it is necessary to determine the actual cost effective improvement technique. It is not the intention of the report to produce a "one-size-fits-all soft soil solution", but to present research and findings on a range of possible solutions, which, supported by the use of the flow charts provided, will assist LGED engineers in choosing the optimum solution for their particular project or problem. The contributing factors to selection of ground improvement techniques are summarised in Chapter 7 of the Final Report.
	Bearing capacity, settlement should be calculated. For a typical ground model, these are included in Chapter 6. Further work has been recommended to expand the current project remit to include standard designs including the calculation of bearing capacity and settlement.
	Changes of moisture content in subgrade and super fill of the embankment with changing situation. The impact of shrink / swell has been noted in Chapter 5, although this characteristic effect mainly relates to embankment fill rather the subgrade, and hence cannot be readily influenced by ground improvement.
	Step by step causes of failure of village road and structure. This is included in the Final Report Chapters 4 to 6.
	Failure in terms of consolidation. This is understood to refer to a serviceability limit state failure e.g. settlement beyond functional requirements, rather than, for example, a slope failure (ultimate limit state). This is explained in Chapter 6 of the Final Report.
	Soft soil improvement techniques in particular area, that you have implemented can be shared through your website. All reports from this project, covering the soft soil improvement techniques, will be uploaded on final approval to the ReCAP website.

Question 3 What further work would you like to see undertaken in subsequent stages of this project?	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	Different test results like bone log, limit test, unconfined compression test results are done during sub-soil investigation. How these test results are interpreted to select a particular technique can be shared? Further work has been recommended in the Final Report to expand the current project remit to include standard designs, including how to determine parameters from ground data.

Question 4 Additional comments, feedback and questions.	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	Improvement techniques regarding different type of foundation and structure chart should be incorporated for more information. Charts for different engineering applications are included in the Final Report. Further work has been recommended in the Final Report to expand the current project remit to include standard designs
	Need to know reliable and easy methods of sub-soil investigation. Typical ground investigation methods for standard applications are included in Appendix K of the Final Report.
	Presently LGED is using micro pile and toe loading system the region so this system to covered here. This technique has not been mentioned to the author team throughout the research project and is not included. The use of palisade wall has been mentioned, but this is not a ground improvement technique – it is a slope stability measure. The authorship team would welcome details of further methods being trialled by LGED or others in Bangladesh.
	Most of the pictures attached in the presentation are the case of non-maintenance of proper slope in the road or approach embankment. There should be a comparison of cost whether soil treatment or routine maintenance is cost effective or not. The example used in the presentation concentrated on a single ground model – an approach embankment, as there was insufficient time to go through all applications. The Final Report

Question 4 Additional comments, feedback and questions.	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	includes remedial use and new-build applications. The second part of the question is extremely important and identifies the life-cycle cost issue raised in Chapter 7; this is important but requires more information about the methods used and the associated costs. For example, can the cost of 20yrs of maintenance be measured and compared to the capital cost of installing a new solution? The authorship team would need more hard cost data to be made available from LGED to answer this.
	Can the panda probe data be used directly as an assessing bearing capacity? The Panda Probe, like any other cone/probe, can be calibrated against soil data (from an adjacent borehole) to provide engineering parameters that can be used to determine bearing capacity / deformation etc.
	The flow diagrams look critical for our field engineers so it may be more simple solution. These flow diagrams are included in Chapter 10 of the Final Report.
	How bio-engineering method is suitable for controlling lateral movement of soil, especially vertical grass plantation or other plantation in the road side embankment This refers to a slope stability rather than a sub-grade issue, and thus was outside the scope of this research project.
	Jet grouting can be used but in saline zone effectiveness of jet grouting/ cement grouting shall be verified by taking a pilot project. Jet grouting is not suitable for the engineering application on rural roads. Jet grouting is very specialised, expensive and requires high levels of skill and site controls to avoid blow-out at surface due to the very high pressures used with this technique.
	The presentation should recommend the case by case solution of settlement. The example provided in the presentation concentrated on a single ground model – an approach embankment, as there was insufficient time to go through all applications. Further work is recommended to expand the current project remit to include standard designs, including how to assess settlement.
	Proper solution for remedial measures. Chapter 10 of the Final Report includes Text and Tables that

Question 4 Additional comments, feedback and questions.	Comments Submitted comments are shown in plain text, authors' responses in italics where required.
	describe potential remedial measures to address specific defects observed. It is noted in the Report than ground improvement techniques are difficult to implement for remedial applications (Section 10.2.1).
	Does the report consider swelling of soil? The impact of shrink / swell has been noted in Chapter 5, although this characteristic effect mainly relates to embankment fill rather the subgrade, and hence cannot be readily influenced by ground improvement.
	What is the design site of these roads (long term traffic loads) Traffic loading has not been considered in this project. Traffic loading is relevant to the embankment construction and serviceability of the surfacing (e.g wheel rutting), but has little influence on the overall load applied to the sub-grade and improved ground.

Conclusions

The Stakeholder Workshop was well attended, with a high level of engagement, interest, and experience brought to the table from the assembled floor of experts and practitioners. Where technical questions and comments were not directly answered in session, the comments raised typically focussed around the following key areas:

- Requirement for piloting to test the recommended ground improvement techniques;
- Recommendation of alternative techniques: Most of these techniques had been considered as part of the process under project in selecting techniques appropriate to the rural roads environment, and in some cases (such as jet grouting), considered impractical for this context. In others, useful comment has been made on the practicality of techniques, to feed into the recommendations of the final report; and in all cases, the consultation was highly informative in understanding more the practical and operations preferences and experience from LGED engineers;
- Requests for ongoing training and capacity building of design and construction methodologies for ground improvement techniques for the rural roads network;

All received comments have been carefully analysed and understood and relevant content included in the project Final Report. Key themes identified above have been incorporated in the recommendations for ongoing work for the embedment and uptake of the project findings.

Appendix C: Review of GI and lab testing in Bangladesh

Formation of boreholes

The manual percussion drilling method is predominantly used for ground investigation in Bangladesh. The main reasons to use the manual percussion drilling method are the simplicity of the apparatus and availability of cheap labor to conduct the works. Recently due to the requirements of some consultants, rotary percussion is also now being employed by some Bangladesh site investigation companies. The companies that now have rotary drilling facilities are private organizations and comprise:

- Prosoil Foundation Consultant.
- ICON Engineering Services.
- DCL Companies.

During ground investigations conducted in Bangladesh, it is typical for SPT (Standard Penetration Test) data to be recorded and for disturbed and undisturbed samples to be retrieved.

In-situ testing

Some in-situ tests are also available and conducted in Bangladesh and these include:

- Dutch Cone Penetration Test.
- Plate Load Test.
- Screw Plate Load Test.
- Borehole Shear Test.
- Pressuremeter Test.
- Dynamic Cone Penetration (DPL, DPM and DPSH).

Available laboratory testing techniques

Predominantly in accordance with ASTM requirements – NB: There is no accreditation process to ensure that other certification practices are acceptable).

Index & Physical properties:

- Specific Gravity.
- Unit weight (wet & dry).
- Void ratio (Specific Gravity & Unit Weight).
- Moisture content.
- Liquid limit and Plastic limit.
- Linear Shrinkage.
- Shrinkage limit.
- Grain size analysis by wet sieving and Hydrometer.
- Organic matter content - Loss on ignition.
- Sand equivalent test.

Compaction and density tests:

- Maximum and Minimum density of cohesionless soil.
- Standard Proctor Compaction test.
- Modified Proctor Compaction test.

Permeability and seepage characteristics:

- Permeability of cohesive soil by 1-dimensional consolidation.
- Permeability of cohesionless soil (falling head)

- Permeability of cohesionless soil (constant head)

Consolidation and swelling characteristics:
- One dimensional consolidation (e - log p, Cc,Cr,Cv)
- Swelling Index / Swelling Pressure

Strength and deformation characteristics:
- Unconfined compression test
- Lab. California Bearing Ratio (CBR) of soils

Direct shear tests:
- Consolidated Drained test
- Consolidated quick test
- Unconsolidated quick test

Triaxial shear tests:
- Consolidated Drained compression test.
- Consolidated undrained compression test with pore pressure.
- Consolidated undrained compression test without pore pressure.
- Unconsolidated undrained compression test without pore pressure.
- Consolidated undrained extension test without pore pressure.
- Cyclic Triaxial Test.

Testing facilities available in Khulna

Three organizations have laboratory testing facilities available in Khulna and these are:
1. Khulna University of Engineering and technology (KUET)
2. Roads and Highways
3. Local Government Engineering Department (LGED)

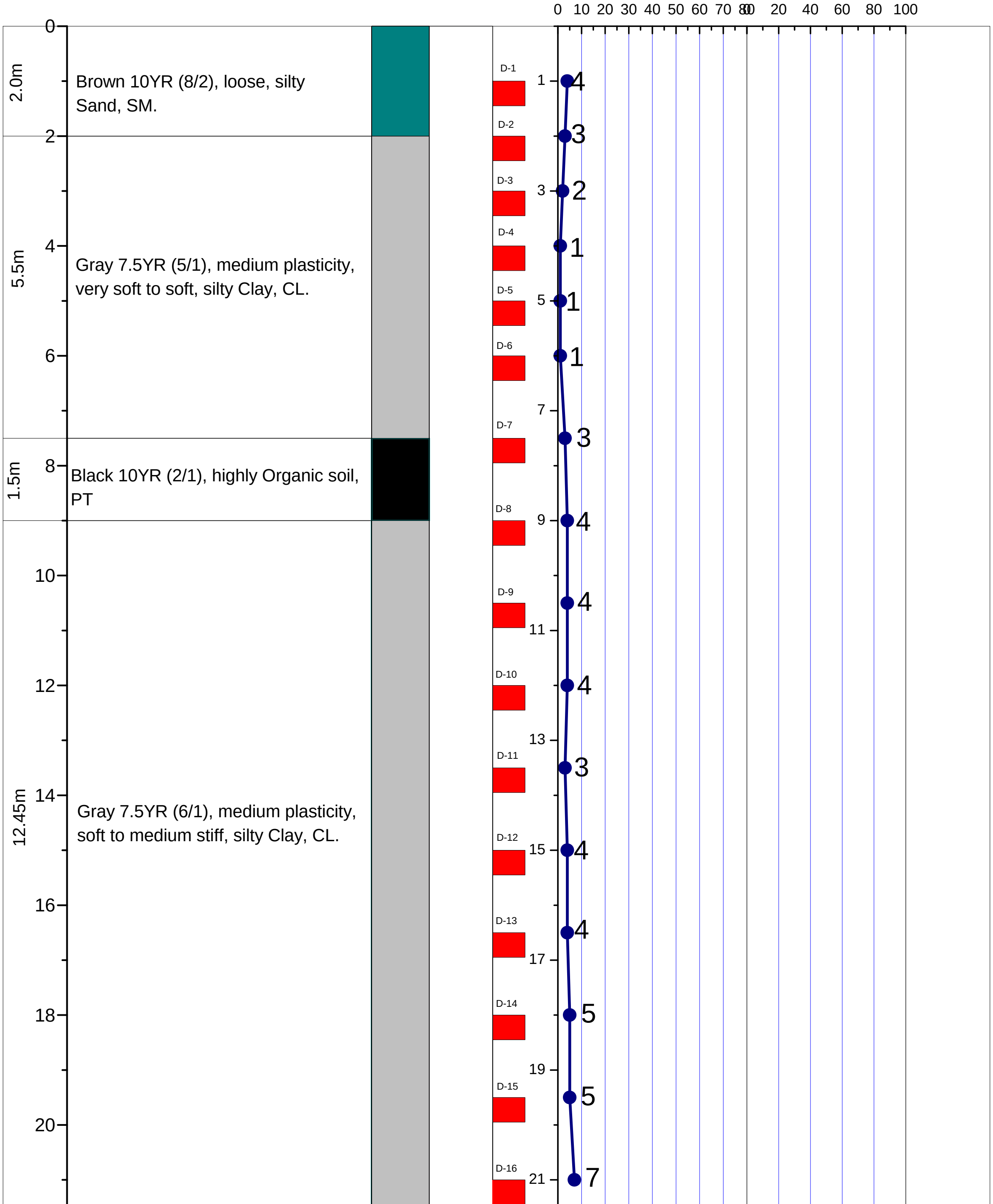
The abovementioned organizations have the facilities to conduct the following laboratory tests:
- Specific gravity.
- Unit weight (wet & dry).
- Void ratio (Specific gravity & Unit Weight).
- Moisture content.
- Liquid limit and Plastic limit.
- Linear Shrinkage.
- Shrinkage limit.
- Grain size analysis by wash sieving.
- Hydrometer, sieve analysis & specific gravity.

Appendix D:Existing Borehole Data

Site Number	Borehole Location	Depth mBGL)	Distance from observational site (m)
3	Kalidaspur,Chandkhali Bazar, Paikgasa, Khulna.	21	~3500
10	Amvita Matri Mondir, Amvita bazar, Dumuria, Khulna	21	~300
11	Banda Bazar, Dumuria, Khulna	21	~5600
12	Kudir Bot-tola, Jabusha, Rupsha, Khulna	51	~4500
13	Terokhada, Khulna	21	~300
15	Terokhada, Khulna (same as 13)	21	~5000
21	Collage Road, Mongla, Bagerhat.	21	~6000

A2	PROSOIL	BOREHOLE NO: BH-1	
		LOCATION: Kalidaspur,Chandkhali Bazar, Paikgasa, Khulna.	
PROJECT : Telecommunication Tower.		DEPTH OF BORING: 21.0m	R. L.: On Road Level.
CLIENT : Orascom Telecom Bangladesh Limited.		DATE OF BORING : 22-09-2013.	G. W. T : 2.0m below from E. G. L.

THICKNESS	SOIL STRATA	BORELOG	UNDISTURBED SAMPLE	DISTURBED SAMPLE	DEPTH (m) SPT VALUES (BLOWS / 300mm)	Grain Size Distribution, % Clay %, Silt %, Sand %	Max^{m} Penetration



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION (BASED ON 'ASTM CLASSIFICATION D-2487')

PLASTIC CLAY	SAND	SILT	SHALE
PEAT/HIGHLY ORGANIC SOIL	MICACEOUS FINE SAND/SILT	UNDISTURBED SAMPLE	DISTURBED SAMPLE

N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

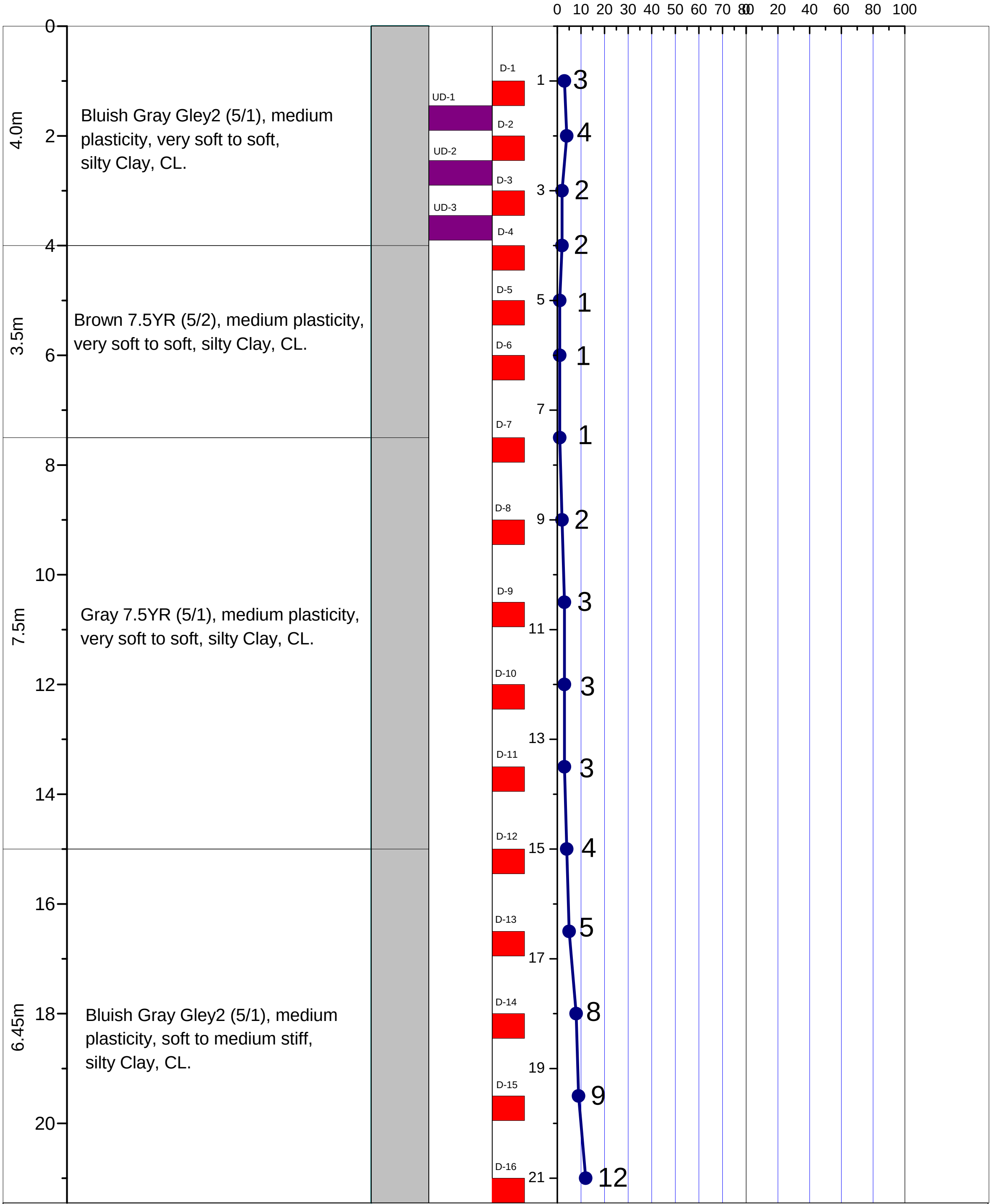
MAX^{m} PENETRATION= 50 BLOWS

AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

A2 PROSOIL

	BOREHOLE NO: BH-1
	LOCATION: Amvita Matri Mondir, Amvita bazar, Dumuria, Khulna.
PROJECT : Telecommunication Tower.	DEPTH OF BORING: 21.0m — R. L.: On Road Level.
CLIENT : Orascom Telecom Bangladesh Limited.	DATE OF BORING : 01-03-2014. — B. W. T : 1.00m below from E. G. L.

THICKNESS	SOIL STRATA	BORELOG	UNDISTURBED SAMPLE	DISTURBED SAMPLE	DEPTH (m) SPT VALUES (BLOWS / 300mm)	Grain Size Distribution, % Clay %, Silt %, Sand %	Max^m Penetration
4.0m	Bluish Gray Gley2 (5/1), medium plasticity, very soft to soft, silty Clay, CL.		UD-1, UD-2, UD-3	D-1, D-2, D-3, D-4	3, 4, 2, 2		
3.5m	Brown 7.5YR (5/2), medium plasticity, very soft to soft, silty Clay, CL.			D-5, D-6, D-7	1, 1, 1		
7.5m	Gray 7.5YR (5/1), medium plasticity, very soft to soft, silty Clay, CL.			D-8, D-9, D-10, D-11, D-12	2, 3, 3, 3, 4		
6.45m	Bluish Gray Gley2 (5/1), medium plasticity, soft to medium stiff, silty Clay, CL.			D-13, D-14, D-15, D-16	5, 8, 9, 12		



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION (BASED ON 'ASTM CLASSIFICATION D-2487')

PLASTIC CLAY — SAND — SILT — SHALE

PEAT/HIGHLY ORGANIC SOIL — MICACEOUS FINE SAND/SILT — UNDISTURBED SAMPLE — DISTURBED SAMPLE

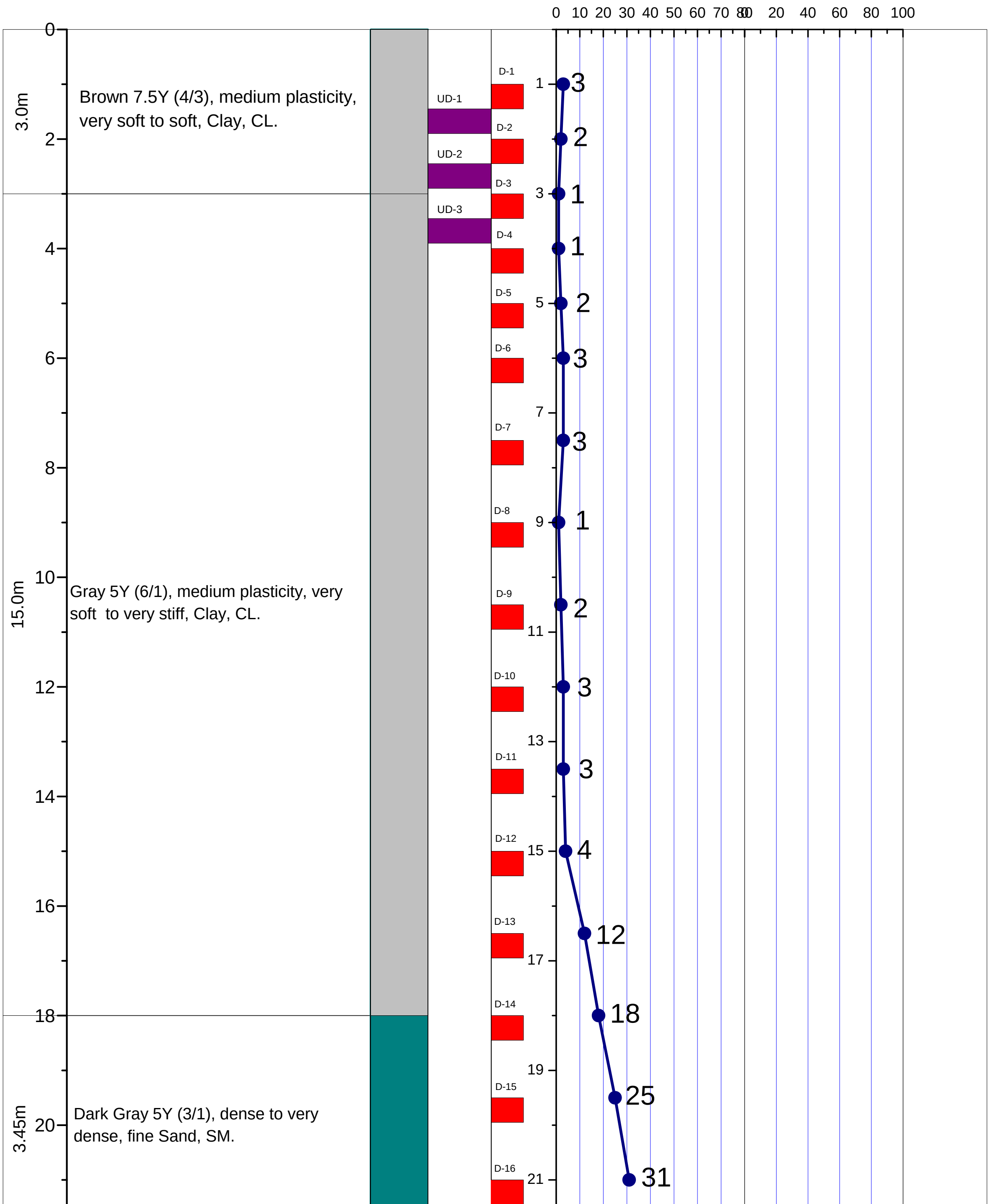
N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

MAX^m PENETRATION= 50 BLOWS

AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

A2	PROSOIL	BOREHOLE NO: BH-1	
		LOCATION: Banda Bazar, Dumuria, Khulna.	
PROJECT : Telecommunication Tower.		DEPTH OF BORING: 21.0m	R. L.: On Road Level.
CLIENT : Orascom Telecom Bangladesh Limited.		DATE OF BORING : 14-03-2014.	B. W. T : 0.30m below from E. G. L.

THICKNESS	SOIL STRATA	BORELOG	UNDISTURBED SAMPLE	DISTURBED SAMPLE	DEPTH (m) SPT VALUES (BLOWS / 300mm)	Grain Size Distribution, % Clay %, Silt %, Sand %	Max^{m} Penetration



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION (BASED ON 'ASTM CLASSIFICATION D-2487')

PLASTIC CLAY | SAND | SILT | SHALE

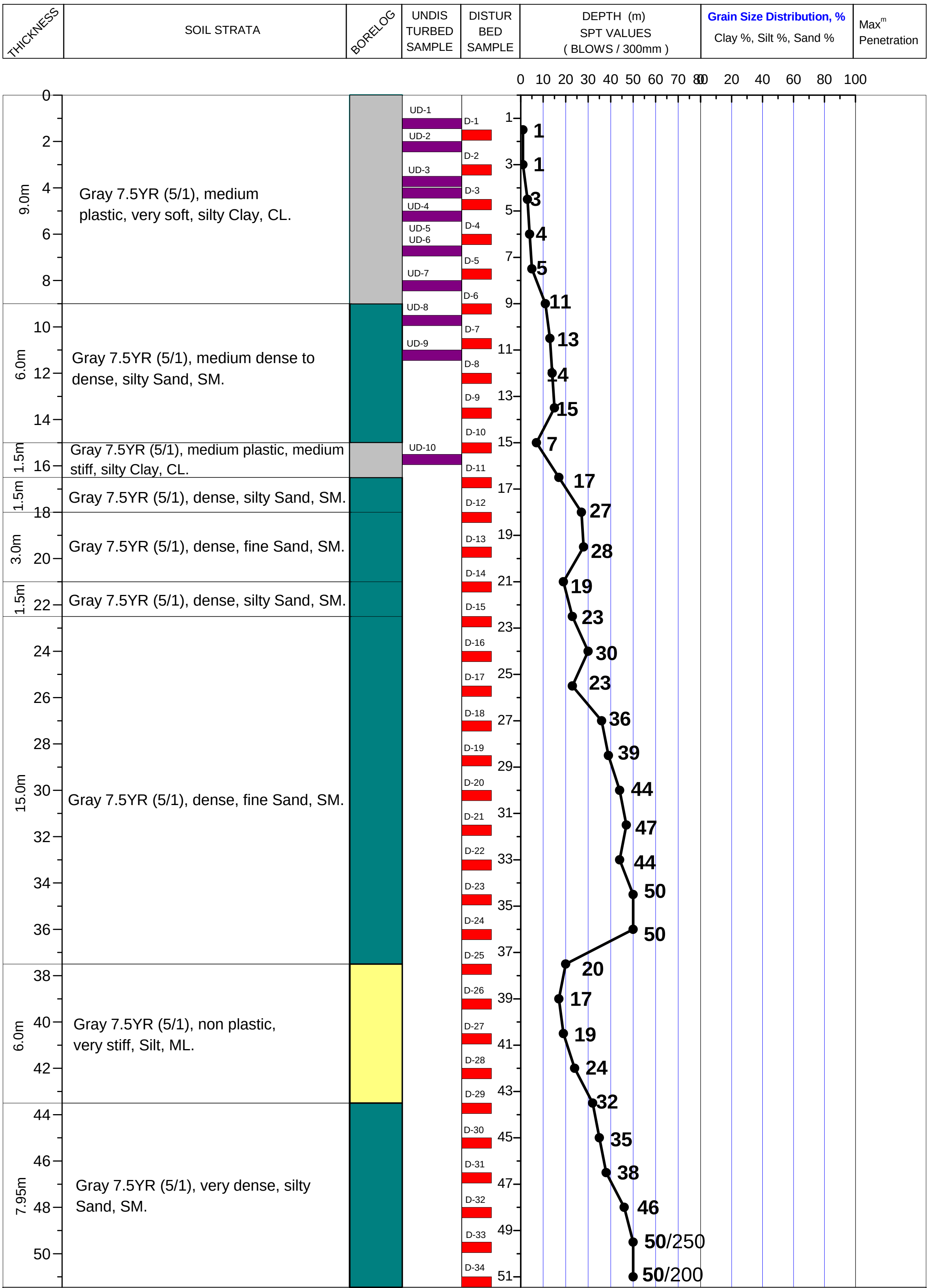
PEAT/HIGHLY ORGANIC SOIL | MICACEOUS FINE SAND/SILT | UNDISTURBED SAMPLE | DISTURBED SAMPLE

N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

MAX^{m} PENETRATION= 50 BLOWS

AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

A2	PROSOIL	BOREHOLE NO: 01	
		LOCATION : Kudir Bot-tola, Jabusha, Rupsha, Khulna.	
PROJECT : Project Popular Detergent, Khulna.		DEPTH OF BORING : 51.0m	**R. L. : 2.51m**
CLIENT : Unilever Bangladesh Limited.		DATE OF BORING : 24-04-2014 to 26-04-2014.	G. W. T : 1.0m below from E.G.L.



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION (BASED ON 'ASTM CLASSIFICATION D-2487')

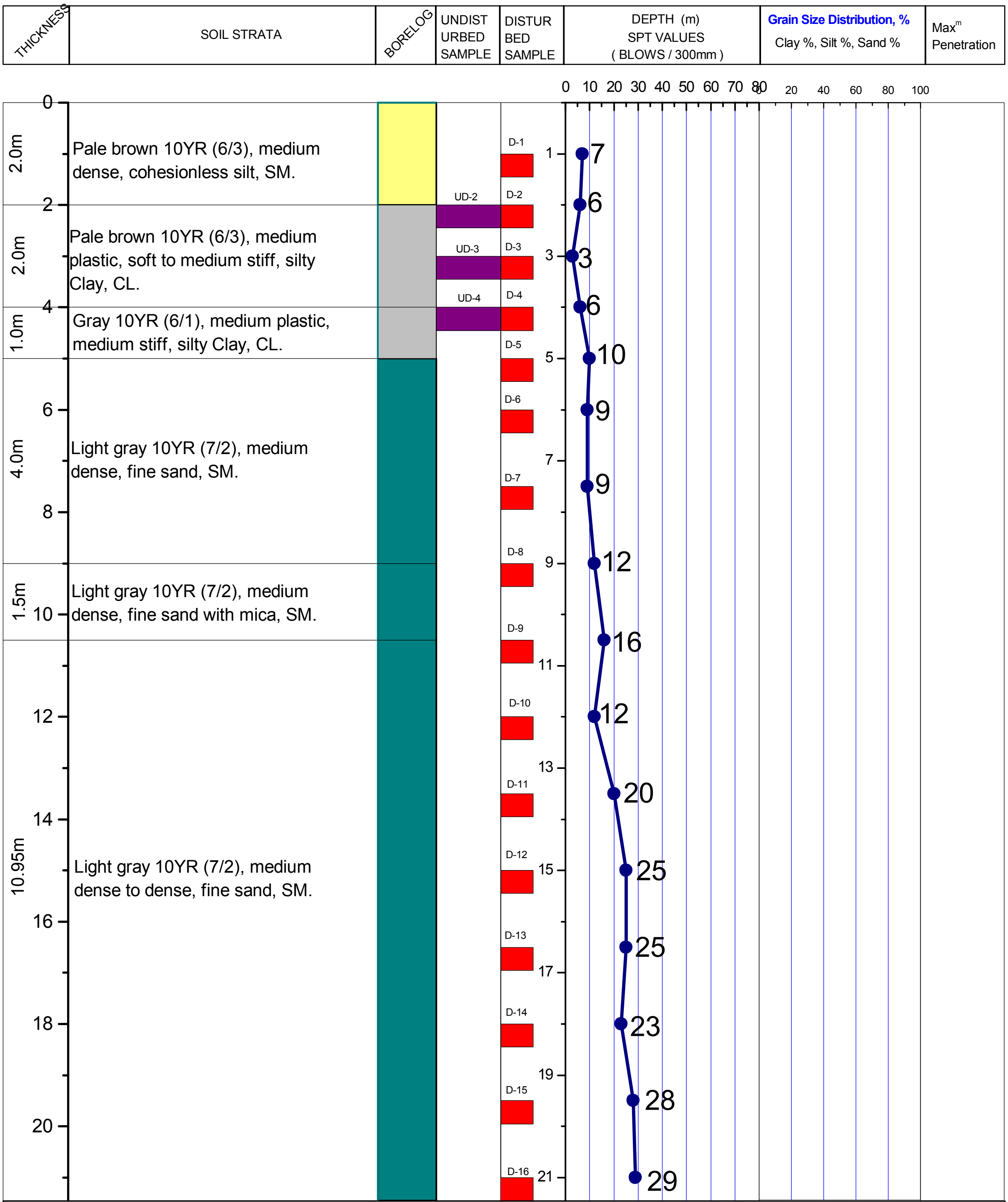
PLASTIC CLAY | SILT | PEAT/HIGHLY ORGANIC SOIL | UNDISTURBED SAMPLE

MICACEOUS FINE SAND/SILT | SAND | SHALE | DISTURBED SAMPLE

N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

MAX^m PENETRATION= 50 BLOWS. AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

A2 PROSOIL	BOREHOLE NO: BH-1	
	LOCATION : Terokhada, Khulna.	
PROJECT : Telecommunication Tower.	DEPTH OF BORING : 21.0m	R. L.: 0.15m below from Road Level.
CLIENT : Sheba Telecom (PVT.) Ltd.	DATE OF BORING : 14-04-2006	G. W. T : 2.9m below from E. G. L.



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION

PLASTIC SILTY CLAY

LOW PLASTIC CLAYEY SILT

NON PLASTIC SILT

SAND

ORGANIC

UNDISTURBED SAMPLE

DISTURBED SAMPLE

SHALE

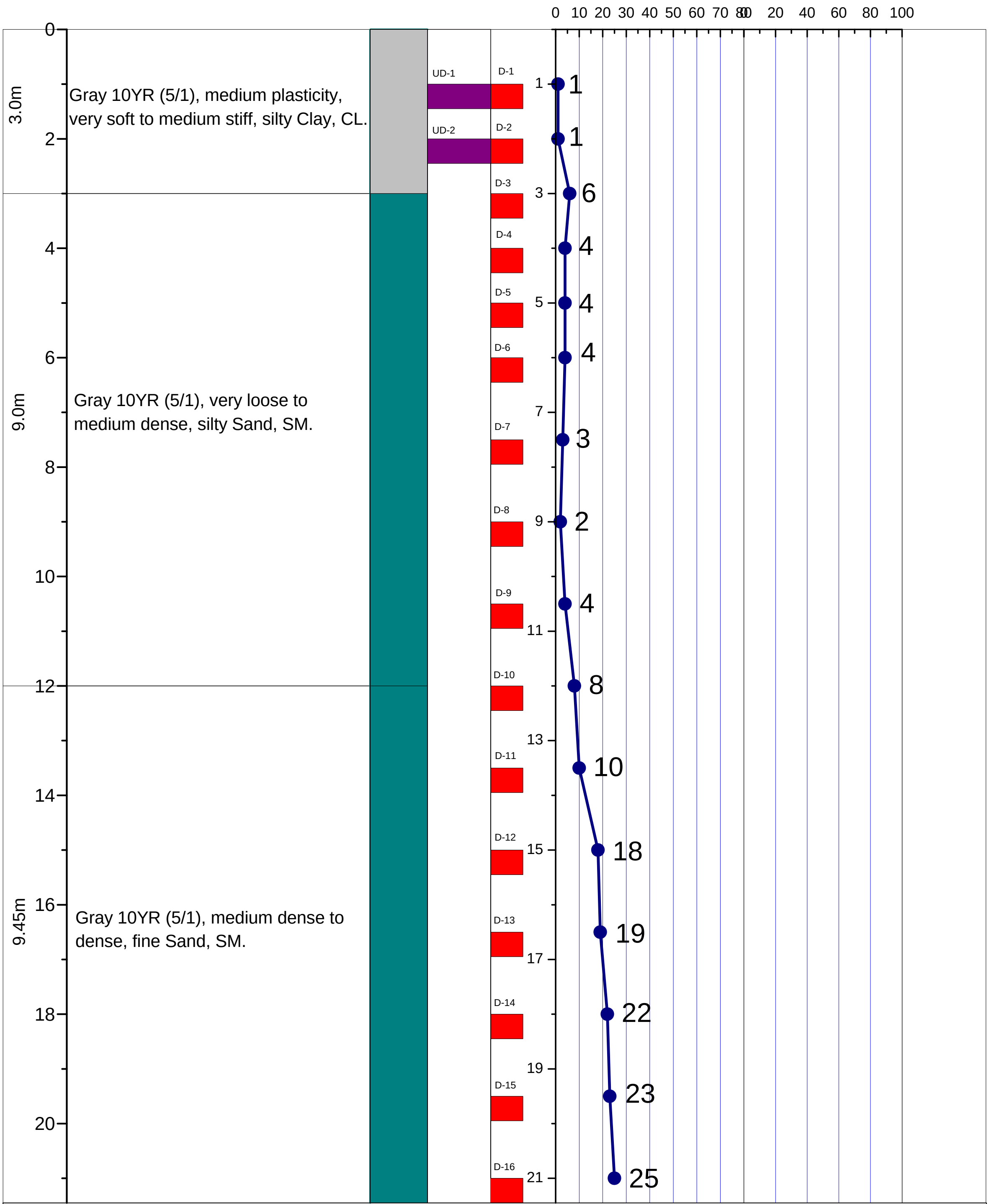
N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

MAX[m] PENETRATION= 50 BLOWS

AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

A2	PROSOIL	BOREHOLE NO: BH-1	
		LOCATION: Collage Road, Mongla, Bagerhat.	
PROJECT : Telecommunication Tower.		DEPTH OF BORING: 21.0m	R. L.: 0.31m below from Road Level.
CLIENT : Orascom Telecom Bangladesh Limited.		DATE OF BORING : 25-09-2013.	G. W. T : On E. G. L.

THICKNESS	SOIL STRATA	BORELOG	UNDISTURBED SAMPLE	DISTURBED SAMPLE	DEPTH (m) SPT VALUES (BLOWS / 300mm)	Grain Size Distribution, % Clay %, Silt %, Sand %	Max^{m} Penetration



LEGEND FOR BORELOG AND GRAIN SIZE DISTRIBUTION (BASED ON 'ASTM CLASSIFICATION D-2487')

PLASTIC CLAY	SAND	SILT	SHALE
PEAT/HIGHLY ORGANIC SOIL	MICACEOUS FINE SAND/SILT	UNDISTURBED SAMPLE	DISTURBED SAMPLE

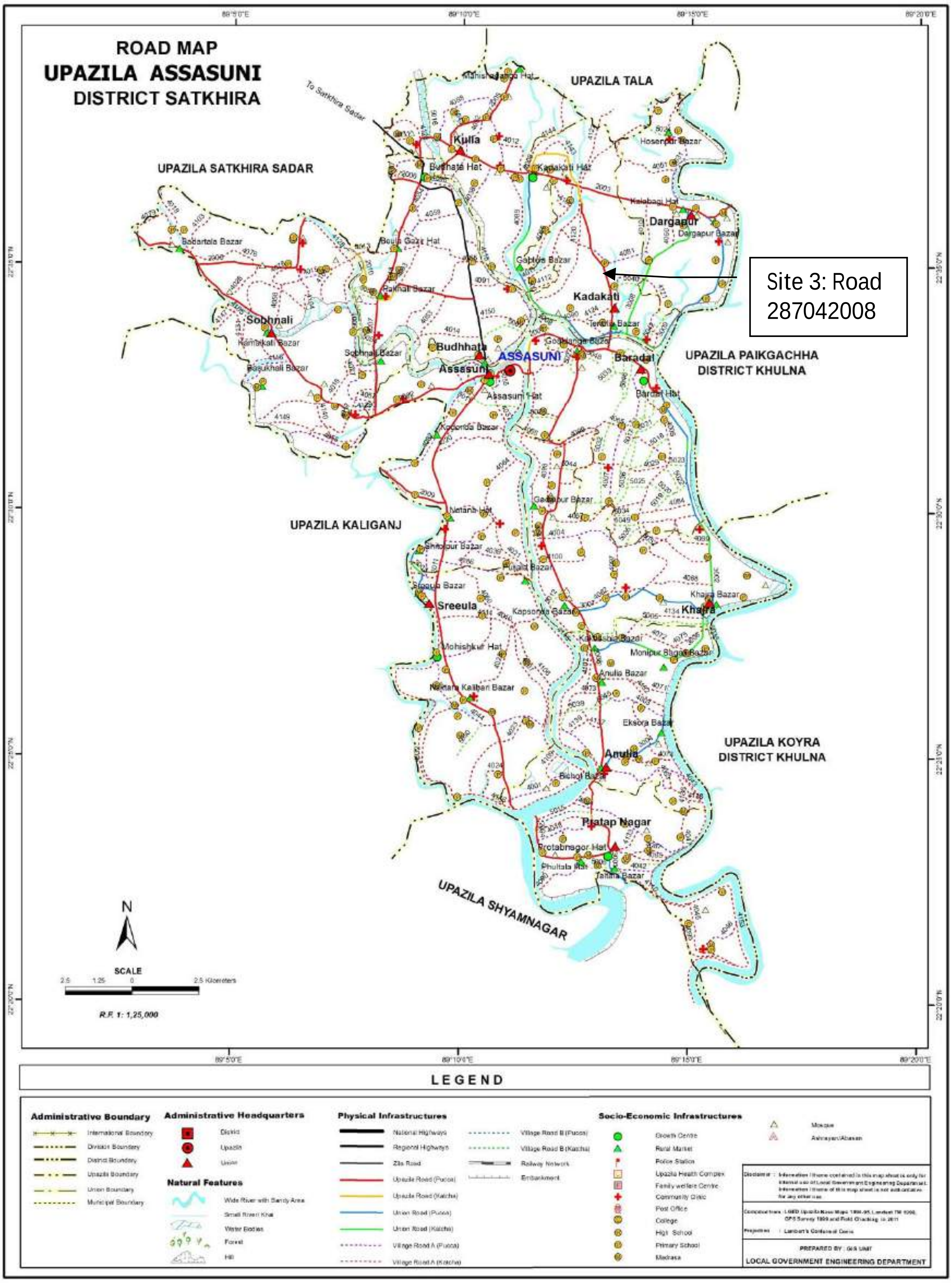
N- BLOWS DELIVERED PER FOOT BY A 140 LB HAMMER FALLING 30 INCHES

MAX^{m} PENETRATION= 50 BLOWS

AFTER 50 BLOWS PENETRATION DEPTH WERE MEASURED

Appendix E: Site Location plans, observations and test data

Site 3 Assasuni



Site 3 Description

Site 3 is a single lane carriageway with a flexible bituminous pavement and brick lined edging / kerb. The embankment is made of soft clay and is planted with planned mixed vegetation. The vegetation was planted 3 years ago at 1.8m centres for slope protection. The vegetation includes Raintree, Koroi and Babla trees, whilst natural grasses and shrubs are also present.

The road is a Upazilla category road connecting to another sub-district. The carriageway is approximately 3 m wide and the embankment is approximately 0.9 m high. Slope angles were observed to vary between 75 and 85 degrees. The slope is at a slacker angle on the RHS of road than LHS. The embankment slope is particularly steep and with minor vegetation in the approach to the wing wall of a bridge / culvert structure. There is a clear loss of soil behind the concrete culvert headwall.

There are heavy vehicles using the road though it was not constructed to accommodate heavy traffic. The passage of heavy vehicles has likely contributed to observed deformation of the carriageway either side of the road centre line and in the run up to a bridge / culvert. Patch repairs and sealed seams are evident in the carriageway surface. The blacktop is extremely badly damaged to the point of being lost entirely over a large stretch of road. Brick kerbing is present where blacktop remains. The resulting surface appears muddy and with tyre tracks and rutting

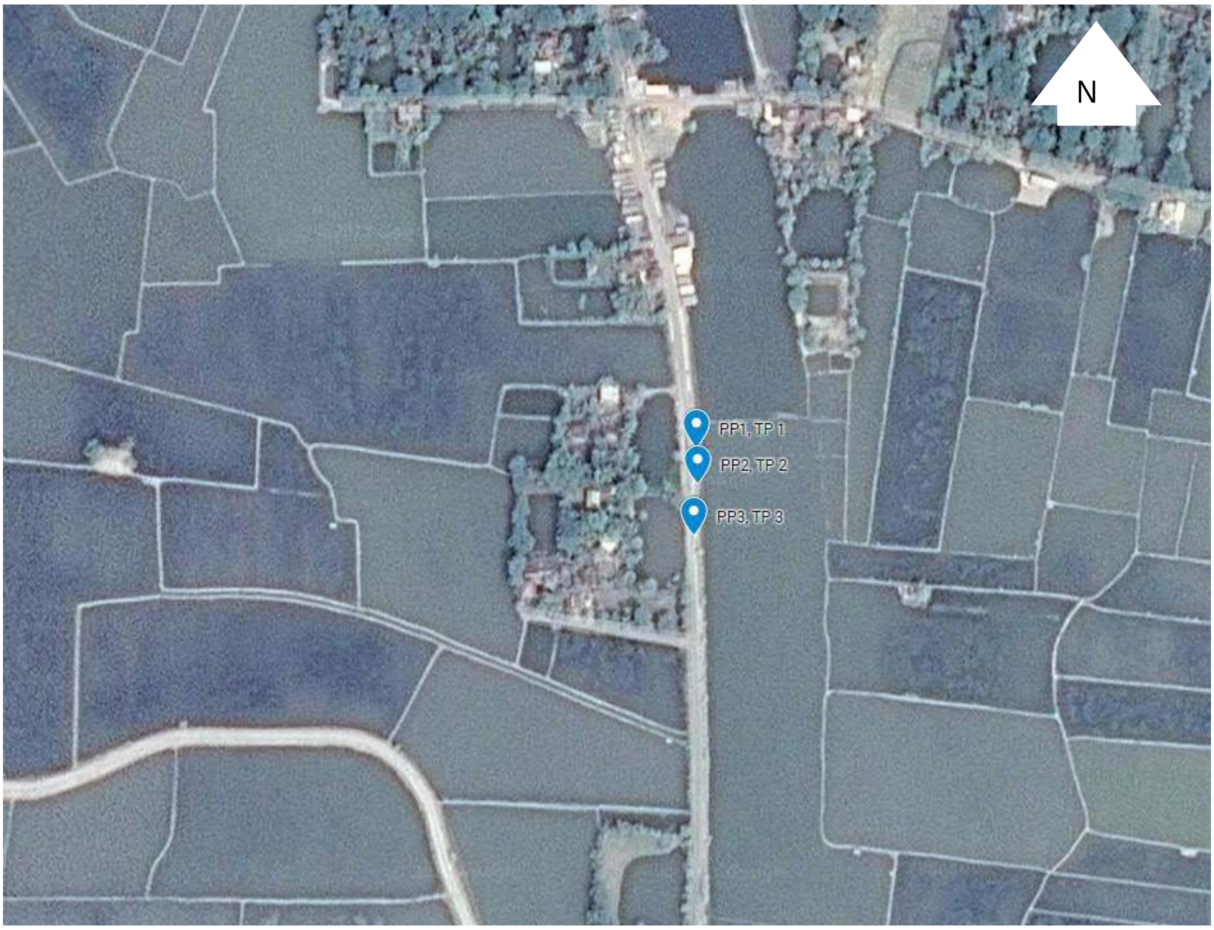
It is clear that vehicles are passing beyond edge of road pavement and brick kerb causing rutting and damage to the soft soils of the verge and slope crest. Reed bales (or similar) have been laid down along the edge of the poor side of the carriageway, in an attempt to temporarily improve the surface conditions and are likely to be masking further problems.

In some places, there is embankment toe protection provided in the form of precast driven piles (~3m long and at 0.9m centres) bolted with vertical concrete slabs in between two piles. This slope toe protection appears to be failing and leaning outwards – seemingly as the embankment spreads.

It is reported that water levels come close to the pavement in the rainy season. Because of fish farming, the water level differs on each side of the road embankment.

Site 3 Investigation and testing

Investigation at the site comprised 3No. trial pits and 3No. Panda Probes. The site investigation locations are detailed below:

Site 3: Assasuni


The factual data obtained from the site investigations is presented below.

Summary of soil sampling for Site 3 Assasuni

Site no. 3: Assasuni (Road: 287042008)

Trial Pit 1

Lithology:

The soil sample contains Very Fine Sand of Reddish Brown colour. The Sand is Loose and contains sufficient amount of Gravels within it. But those gravels are may be mixed from the pavement material because some of them contain coats of Bitumen.

Trial Pit 2

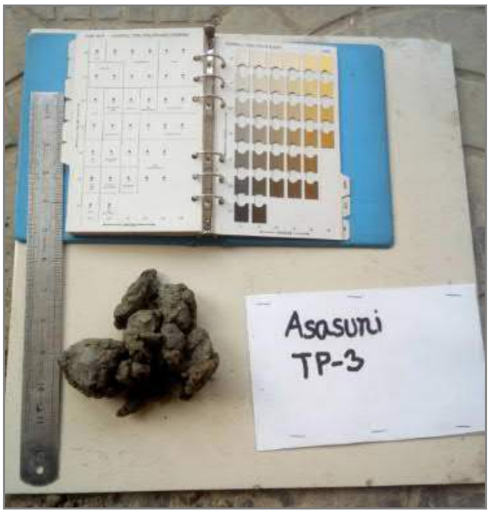
Lithology:

The soil sample is Gray in colour with Moderate Plasticity. It shows Extensive Reddish and Dark materials. The reddish spots are ferruginous materials and dark spots are carbonaceous materials. There are also some straw colour features within the soil which may form by the direct decomposition of organic materials.

Trial Pit 3

Lithology:

The soil sample is Gray in colour with High Plasticity, Occasional lines of ferruginous and spotted carbonaceous materials are present. Small lenses of Grayish Brown soil id also observed within the Gray soil. Soil sample contains plant roots.



Summary of Laboratory Test Results for Site 3 Assasuni

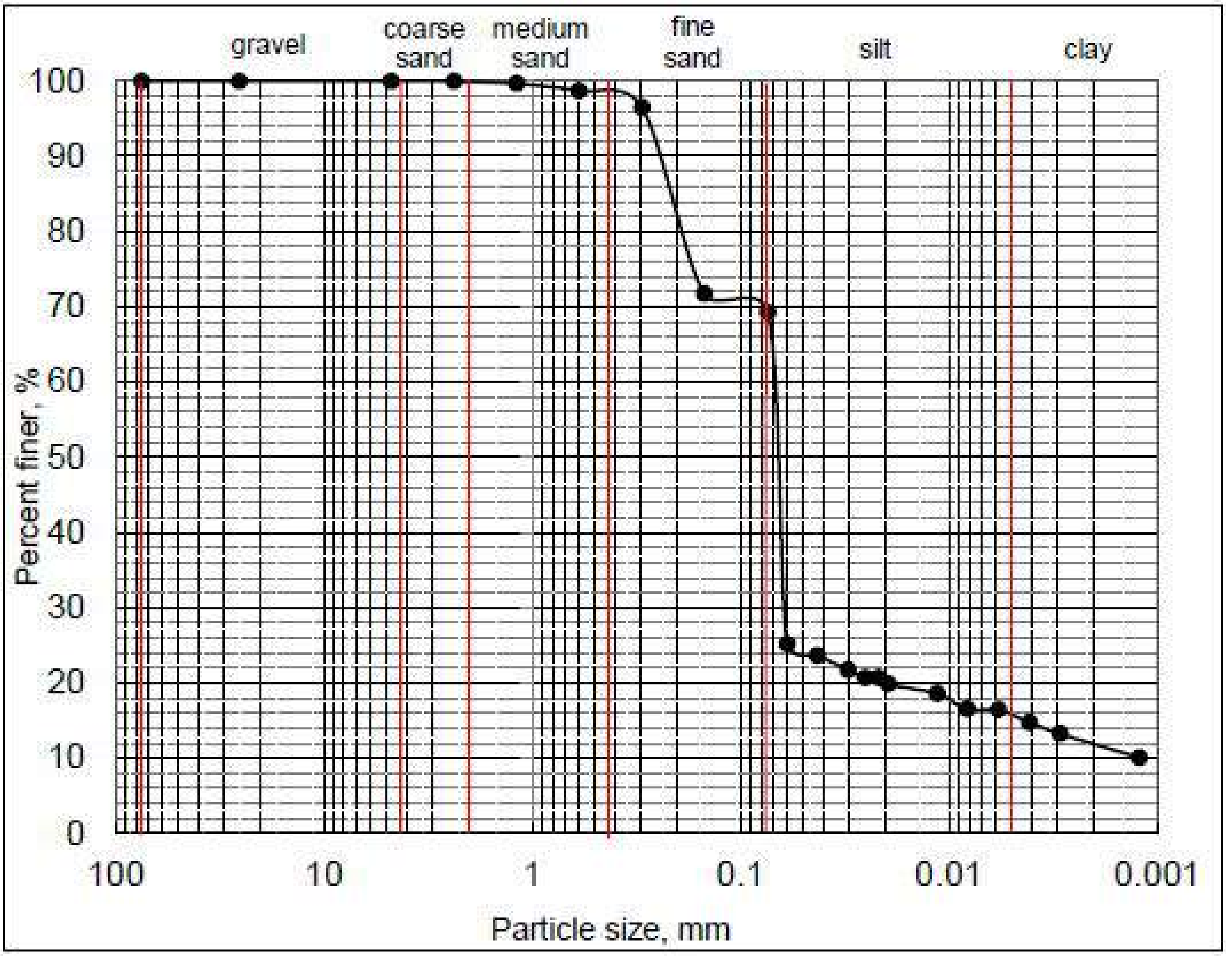
Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
3	1.	0.3	Clay CH - Fat Clay	52	28	-	-	24	8	-
	2.	0.0		NP	NP	-	-	NP	36	-
	2b.	0.0	Silty Sand CL - Lean Clay	27	-	-	-	14	-	-
	3.	0.3		62	19	11	26	43	36	-

*Unified Soil Classification System

The soil used for the embankment fill at the location of Trial Pit No.1 is a Fat Clay.

For the 3 No. cohesive soil samples, the Liquid Limit ranges from 27% to 62%, the Plastic Limit from 19% to 28% and the plasticity index from 14% to 43%. The moisture content obtained for 3 No. samples is 8% and 36%. Linear shrinkage was test for one trial pit (TP3) and returned a value of 26%.

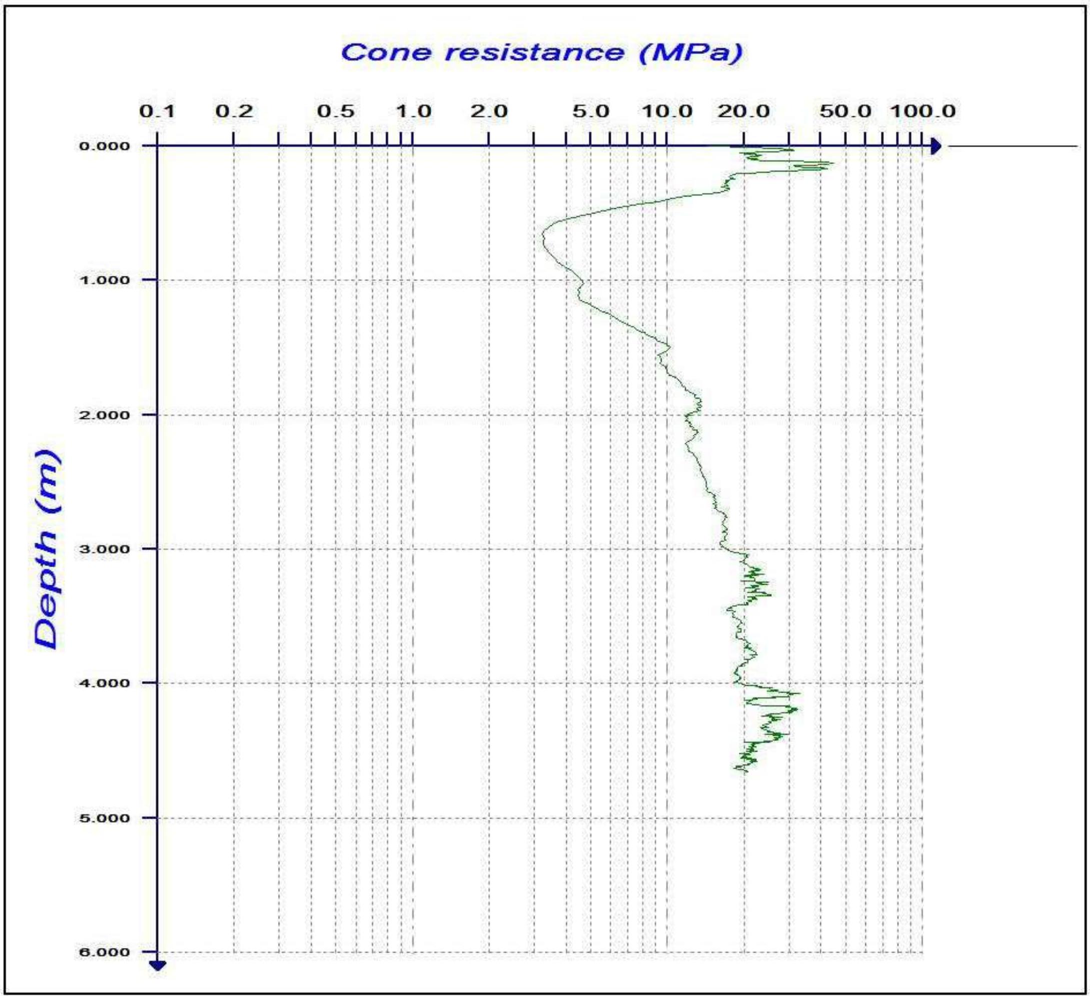
PSD Plot for Sample 2, Site 3



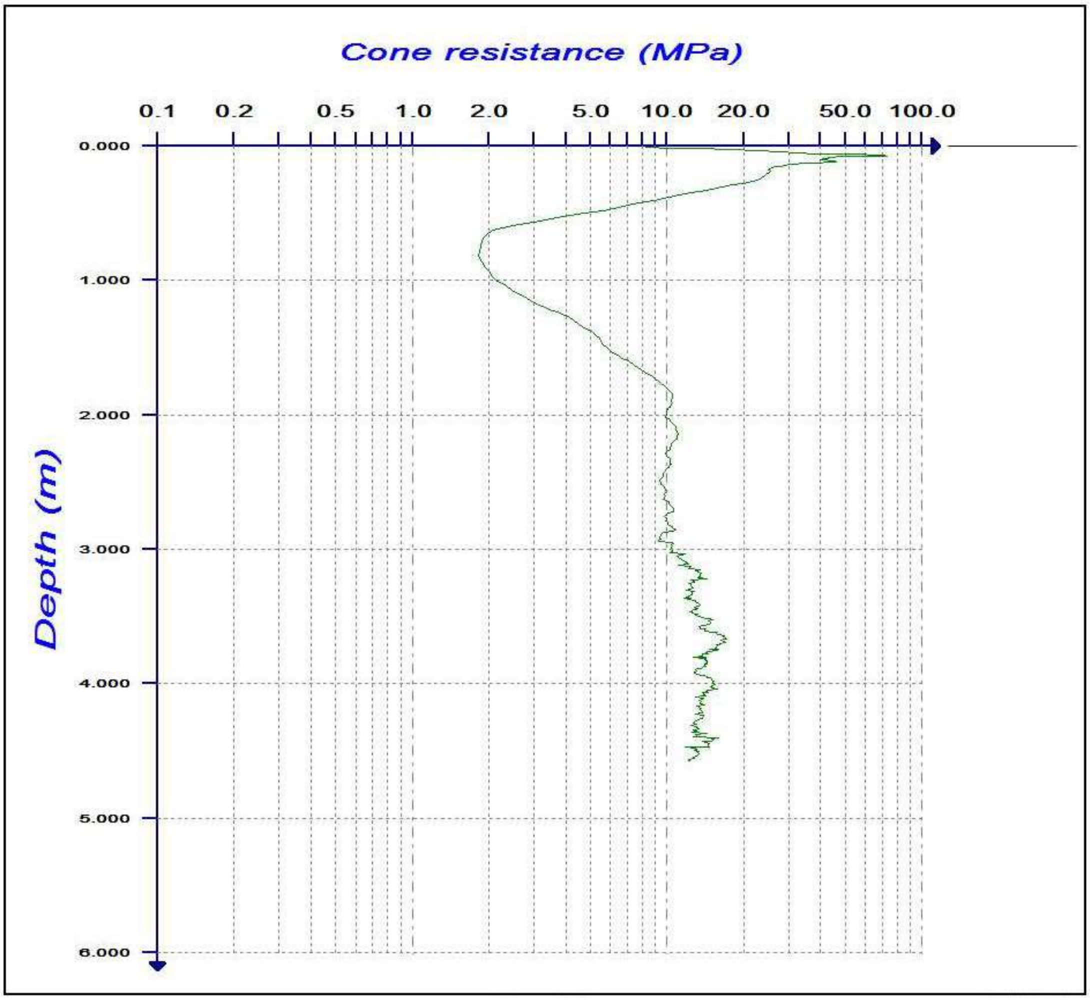
Summary of Panda Probe Results for Site 3 Assasuni

Site Number	Site Name	Probe Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
3	Assasuni	1	4.7	1.2	Reduction in soil strength below road surface from 0 to 0.9 mbgl, before steady increase to end of probe
		2	4.7	1.2	Sharp increase at shallow depth (in road formation) followed by reduction in soil strength below road surface from 0.1 to 0.9 mbgl, before steady increase to 2.0m where strength levels to end of probe
		3	4.5	1.1	General drop in soil strength to 1.5mbgl, before rising to a steady resistance value at 2.0mbgl.

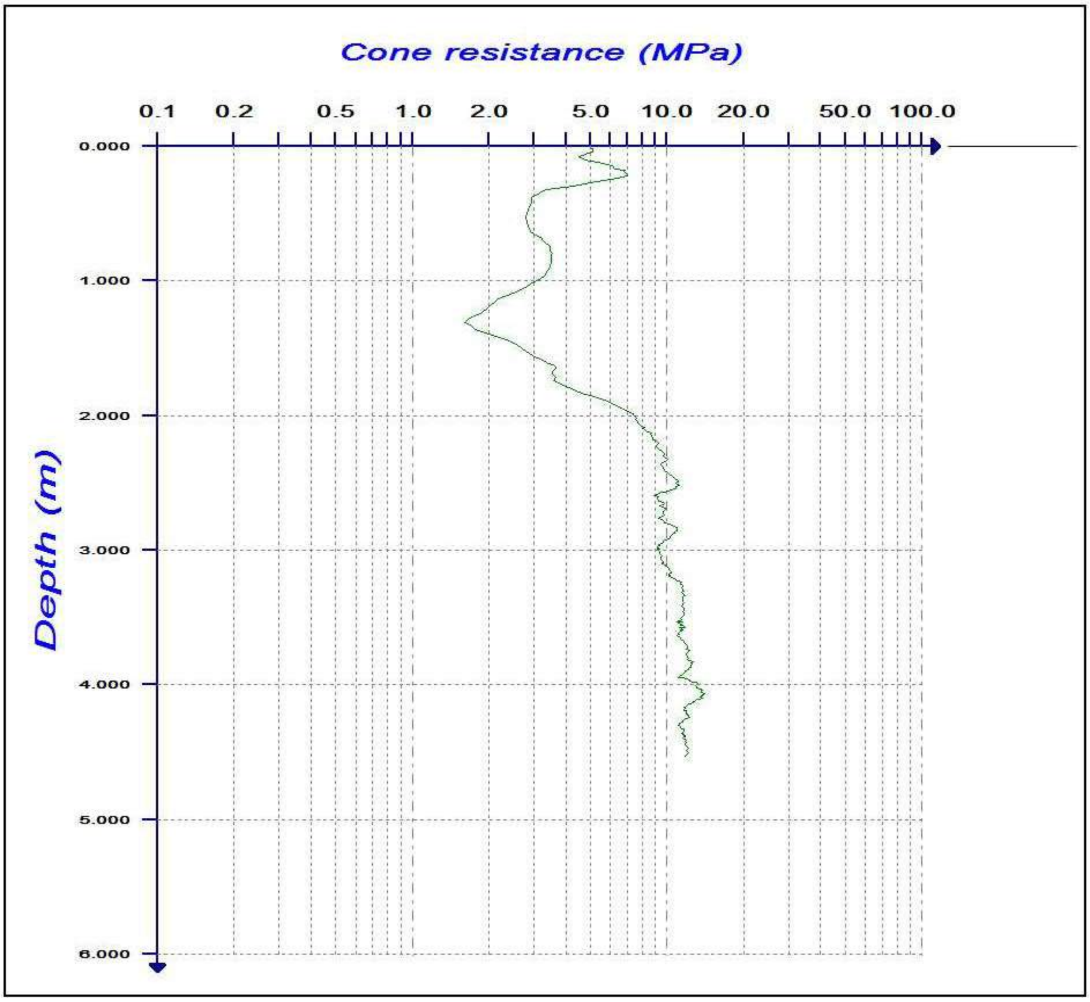
Cone Resistance Plot - Site 3 Assasuni 287042008: Test 1



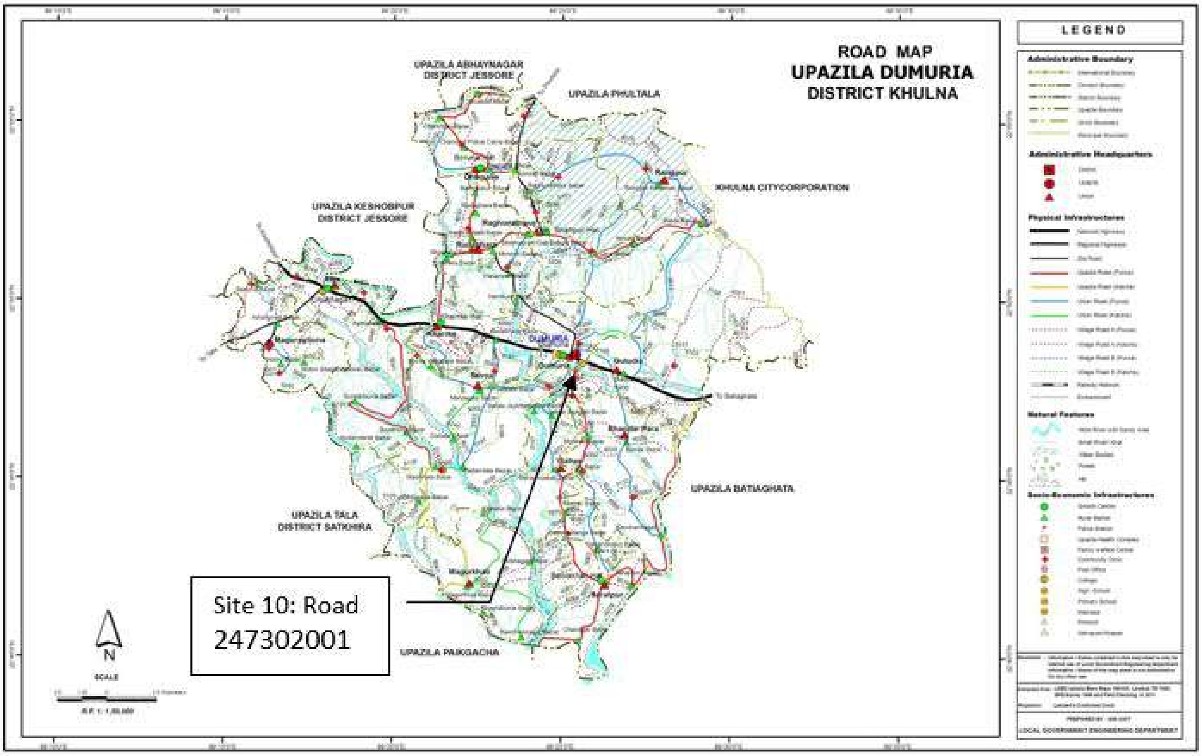
Cone Resistance Plot - Site 3 Assasuni 287042008:Test 2



Cone Resistance Plot - Site 3 Assasuni 287042008: Test 3



Site 10 Dumuria – A



Site 10 Description

Site 10 is a single-track road with a flexible bituminous carriageway. The carriageway is approximately 3.6 m wide. The road embankment is constructed from predominantly organic soil and has uneven slopes that vary in angle between 45 and 60 degrees. The side slopes are vegetated with grasses, herbs and mahogany and Koroi trees and show sign erosion.

The road pavement shows clear signs of distress and cracking. The cracking is most severe on the slope side of the carriageway which displays much longitudinal cracking. There are patches of blacktop loss evident in the road surface.

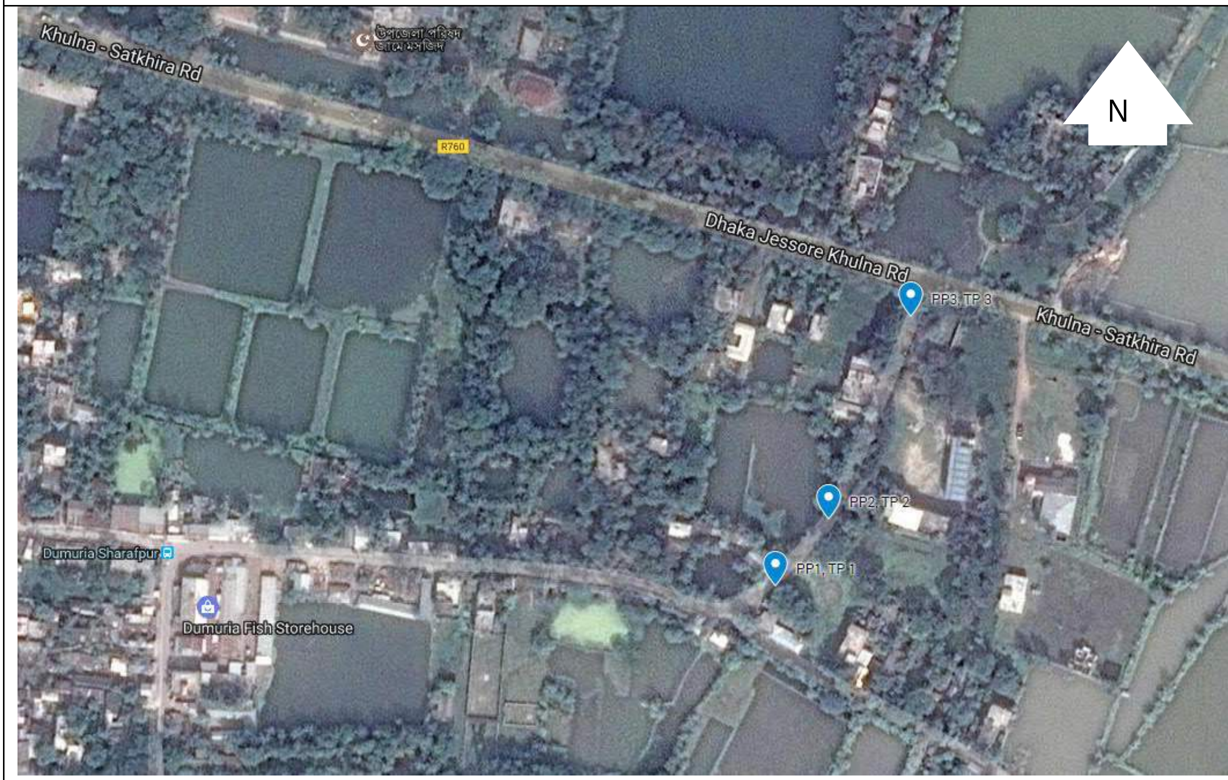
In many places, there is embankment protection provided at the toe of the embankment using 3 m long precast driven pile set out at 0.9 m centres with vertical concrete slabs bolted between a 2 pile arrangement.

The local LGED Engineer has indicated that water levels come close to the pavement surface in the rainy season. Shrimp farming is conducted on either side of the carriageway. There are culverts in the road embankment for cross drainage.

Site 10 Investigation and testing

Investigation at the site comprised 3 No. trial pits and 3 No. Panda Probes. The site investigation locations are detailed below:

Site 10: Dumuria A



The factual data obtained from the site investigations is presented below.

Summary of soil sampling for Site 10 Dumuria A

Site no. 10: Dumuria A (Road 247302001)

Trial Pit 1

Lithology:

bag 1: The soil is Gray colour non-plastic Clayey Silt and it contains brick fragments and plant roots.

bag 2: Grayish Brown Sandy Silt with spotted carbonaceous materials and plant roots.

bag 3: Grayish Brown Sandy Silt.

Trial Pit 2

Lithology:

bag 1: Grayish Brown very fine Silty Clay with plant roots.

bag 2: Grayish Brown Sandy Silt with plant roots.

Bag 3: Brown very fine Sandy Silt with plant roots.

Trial Pit 3

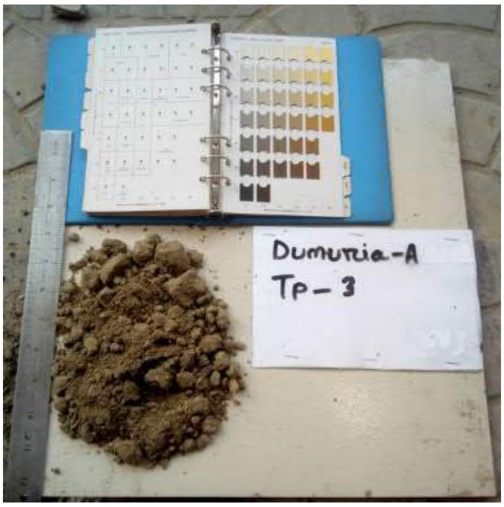
Lithology:

bag 1: Grayish Brown Sandy Silt, which contains plant roots, shells, and partly decomposed wood fragments.

bag 2: Brown very fine Sandy Silt with plant roots and shells.

bag 3: Brown Sandy Silts with plant roots and burrows.

bag 4: Dark Gray plastic Silty Clay with brick fragments, plant roots and shells.



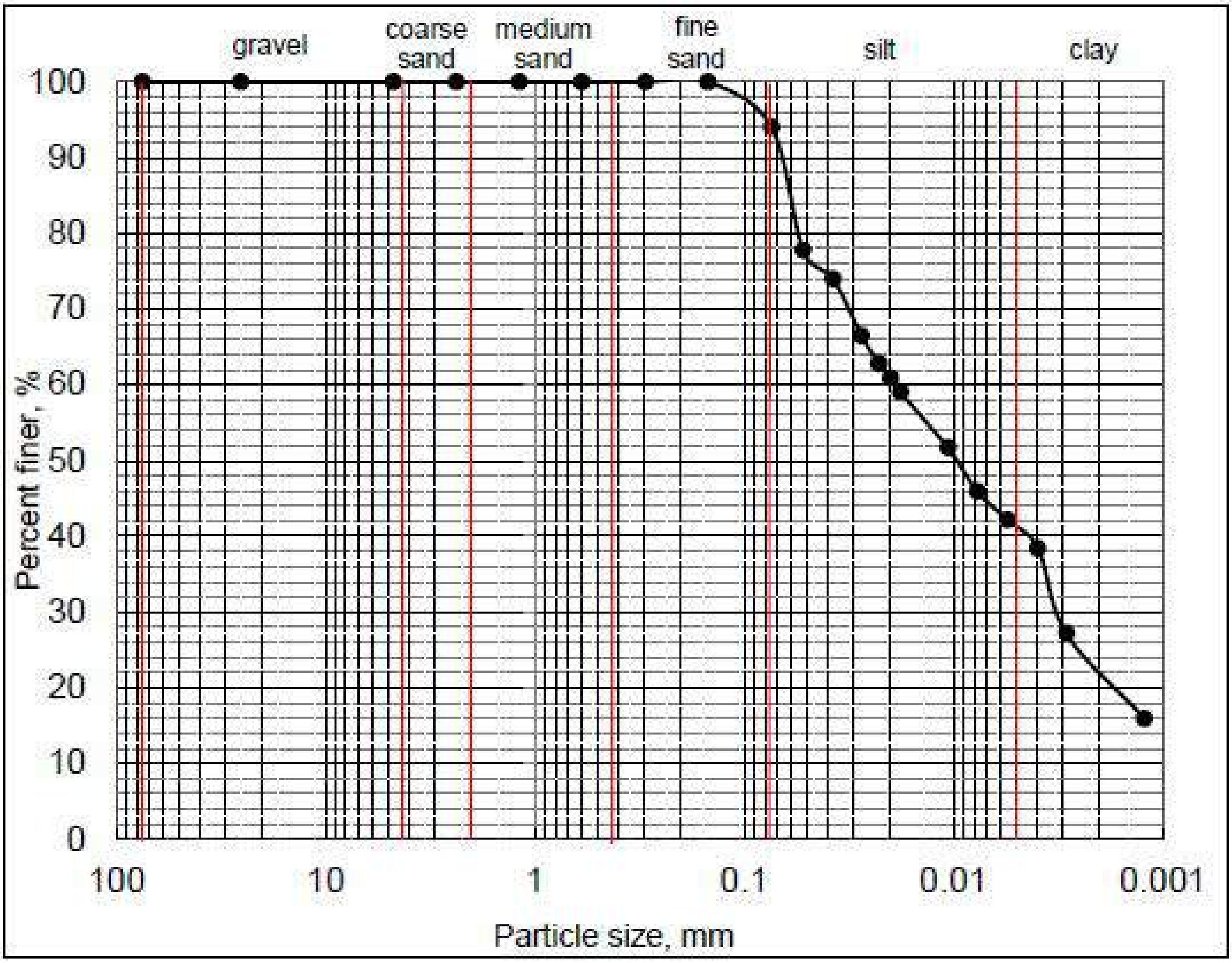
Summary of Laboratory Test Results for Site 10 Dumuria A

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
10	1.	0.25	Clay CL - Lean Clay	47	21	14	18	16	26	6
	1b	0.25	Clay CL – Lean Clay	44	23	-	-	21	-	-
	2.	0.25	Clay CL - Lean Clay	49	21	-	-	28	27	9
	3.	0.3	Clay CL - Lean Clay with Sand	46	20	-	-	26	13	7

*Unified Soil Classification System

The material sampled from the 3 No. trial pits at Site 10 – Dumuria A was very consistent, consisting of a Lean Clay and a Lean Clay with Sand. The Liquid Limit ranged between 44% and 49%, the Plastic Limit ranged between 20% and 21% whilst the Plasticity Index ranged between 16% and 28%. The Moisture Content of the 3 No. samples ranged between 13% and 27% whilst the Organic Content ranged between 7% and 9%. One linear shrinkage test was conducted on sample from TP1 and returned a value of 18%.

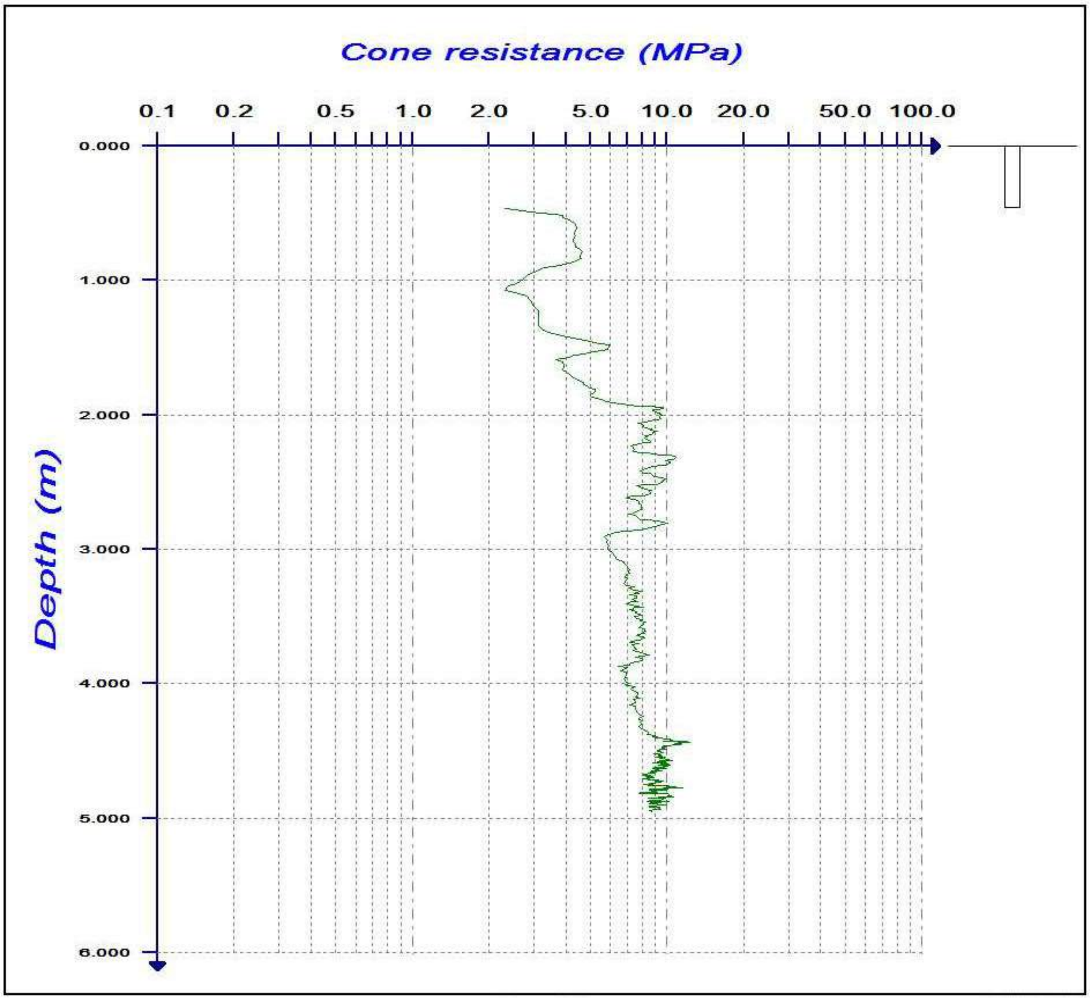
PSD Plot for Sample 1, Site 10



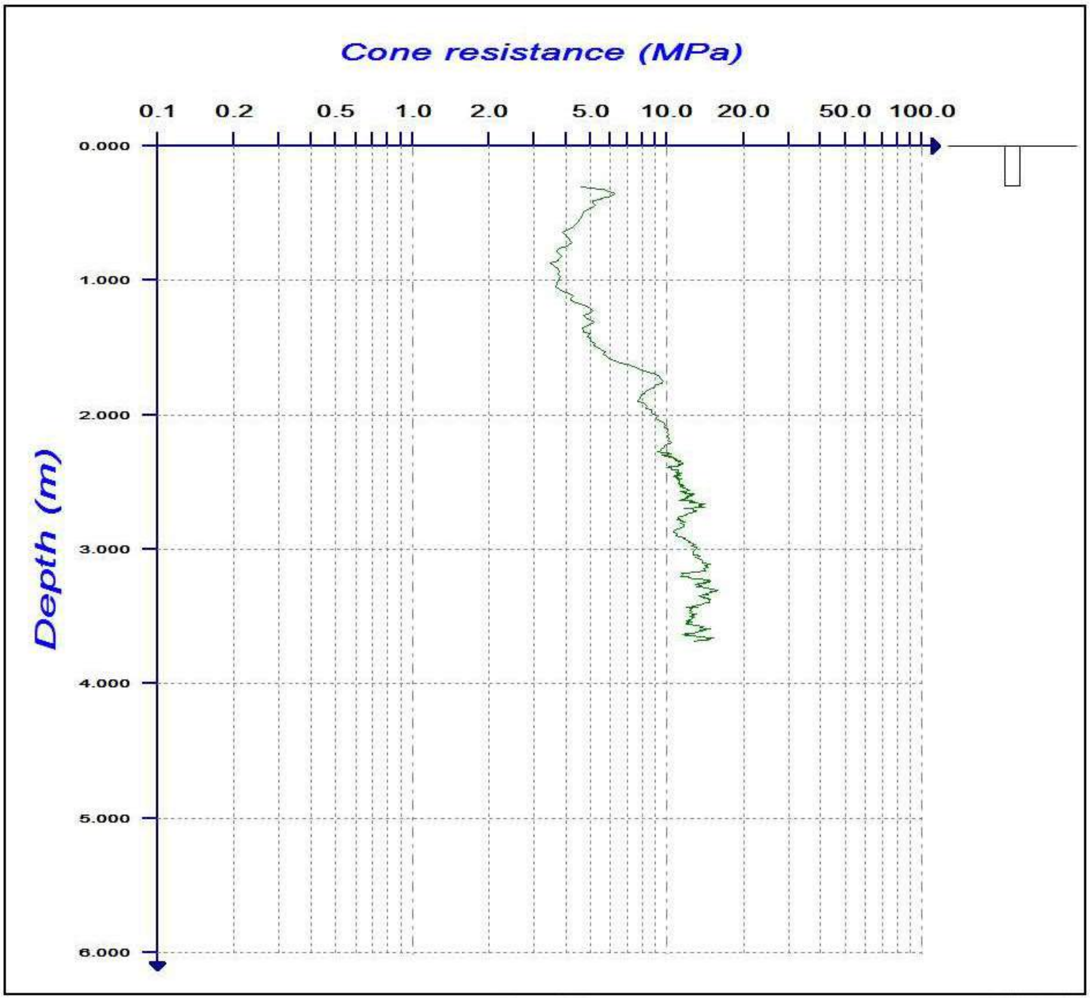
Summary of Panda Probe Results for Site 10 Dumuria A

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
10	Dumuria A	1	4.9	1.8	General slight rise in soil strength to 2.0m then levels off to end of probe.
		2	3.7	2.0	General slight rise in soil strength to 2.0m then levels off to end of probe.
		3	4.5	1.6	Reduction in soil strength below road surface from 0.3 to 0.9 mbgl, before steady increase to end of probe

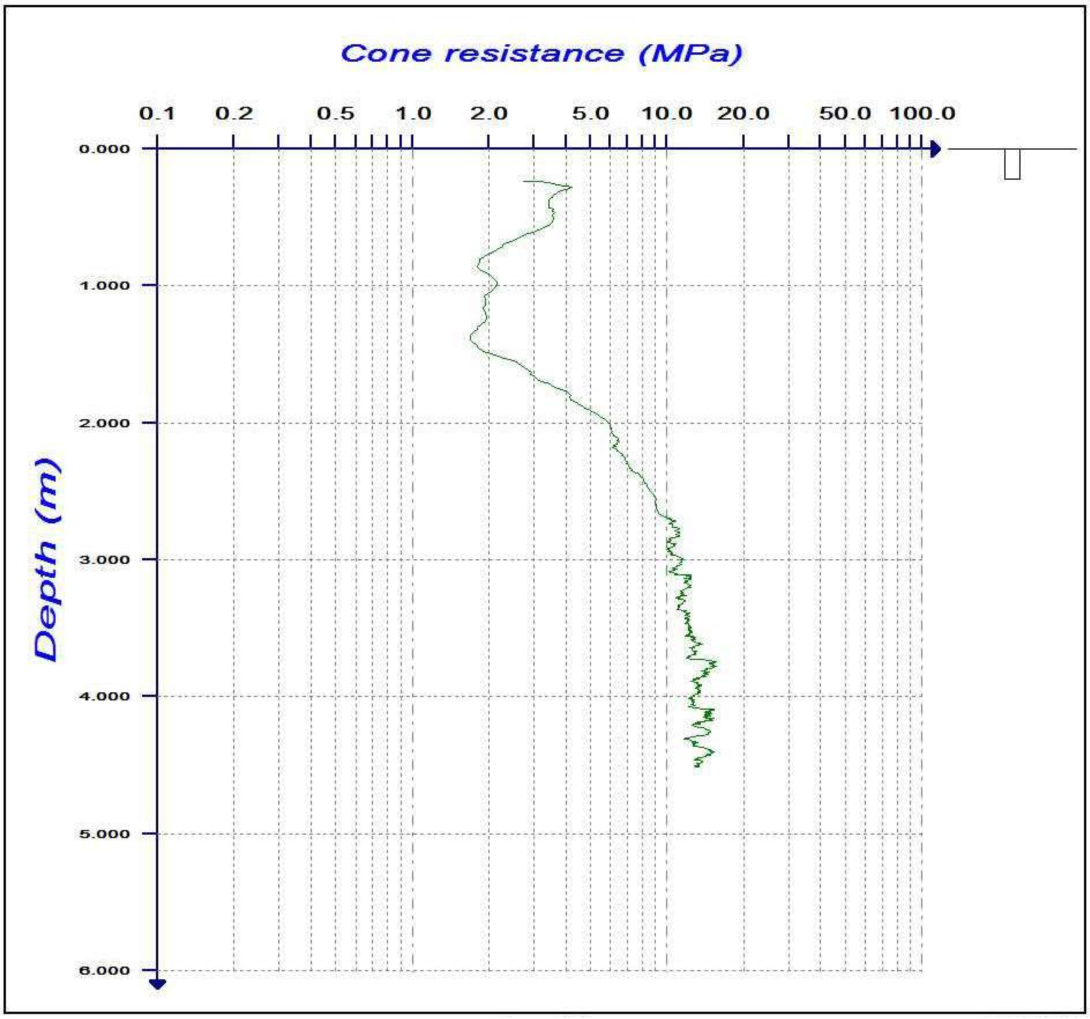
Site 10 Dumuria – Site A 247302001: Test 1



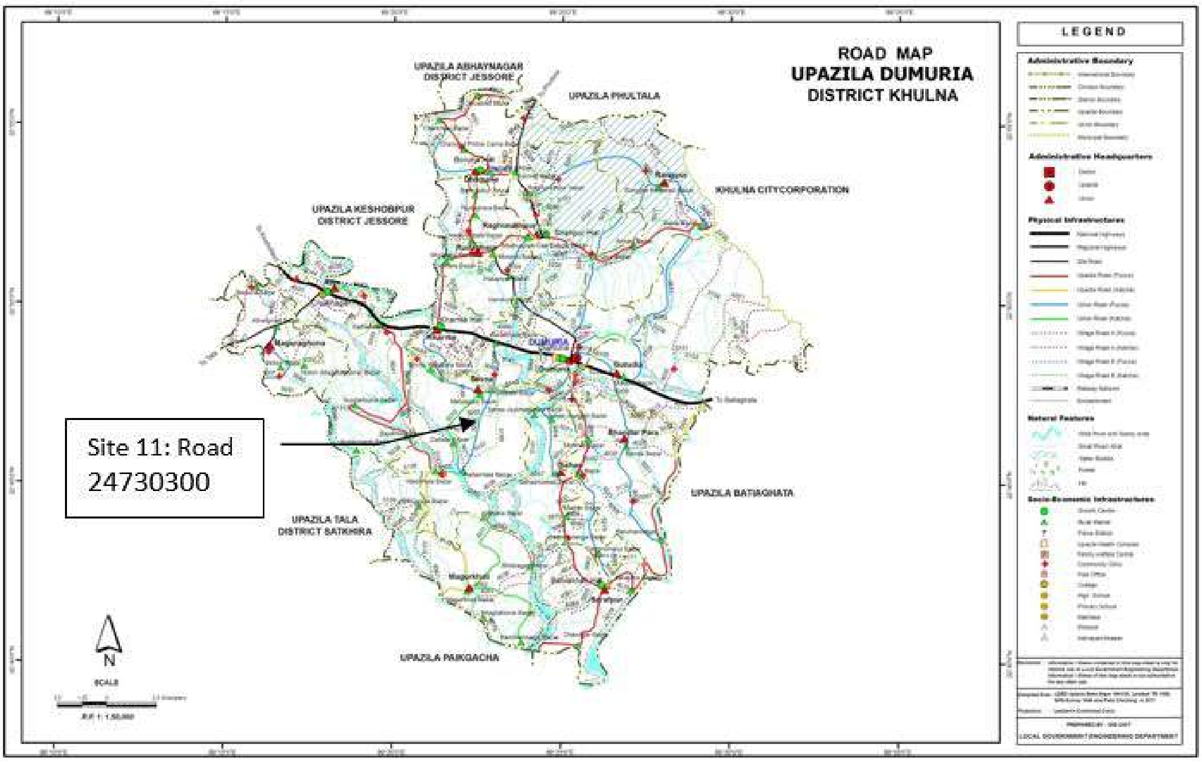
Site 10 Dumuria – Site A 247302001: Test 2



Site 10 Dumuria – Site A 247302001: Test 3



Site 11 Dumuria - B



Site 11 Description

Site 11 is a single lane carriageway with a flexible bituminous pavement. The carriageway is approximately 3m wide.

The embankment is constructed from predominantly organic soil. The slopes were observed to be heavily damaged and are planted with grasses, herbs, and various species of trees. Some trees are leaning outwards from embankment at an angle of approximately 80 degrees. Zones of pavement lost are visible along length of the carriageway. A large zone of pavement failure corresponds with a bulging of the slope toe line – likely due to a relaxation of the embankment material and / or spreading under self-loading and passing vehicle loads.

In many places, there is embankment protection provided at the toe of the embankment in the form of 3m long precast driven piles set out at 0.9m centres with vertical concrete slabs bolted between a 2-pile arrangement.

The local LGED Engineer reports that water levels come very close to the pavement surface in the rainy season. Shrimp farming is conducted on both sides of the embankment. There are culverts in the embankment for cross drainage.

Site 11 Investigation and testing

Investigation at the site comprised 3 No. trial pits and 3 No. Panda Probes. The site investigation locations are detailed below:

1. Lateral spreading of the embankments
2. (Site 11 – 247303003)

Site 11: Dumuria B

Summary of soil sampling for Site 11 Dumuria B

Site no. 11: Dumuria B (Road 247303003)

Trial Pit 1

Lithology:

Bag 1: The soil sample is dark Gray Clay which shows high plasticity. Some reddish Brown to Brown Clayey layers are also present within the soil.

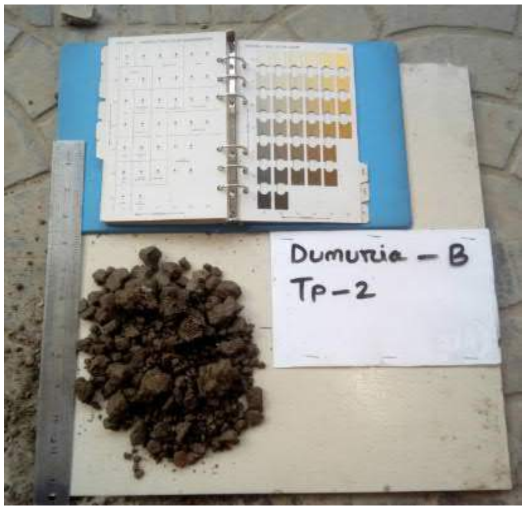
Bag 2: The soil is dark Gray Silty Clay with high plasticity, which contains black layers of soils that may be carbonaceous material rich soil.

Trial Pit 1a (on embankment)

Lithology: Dark Gray Clay that shows high plasticity and contains plant roots.

Trial Pit 2

Lithology: Dark Gray medium plastic Clay with minor Silt with plant roots & shells.



Trial Pit 3

Lithology: Dark Gray medium plastic Silty Clay which contains plant roots.

Summary of Laboratory Test Results for Site 11 Dumuria B

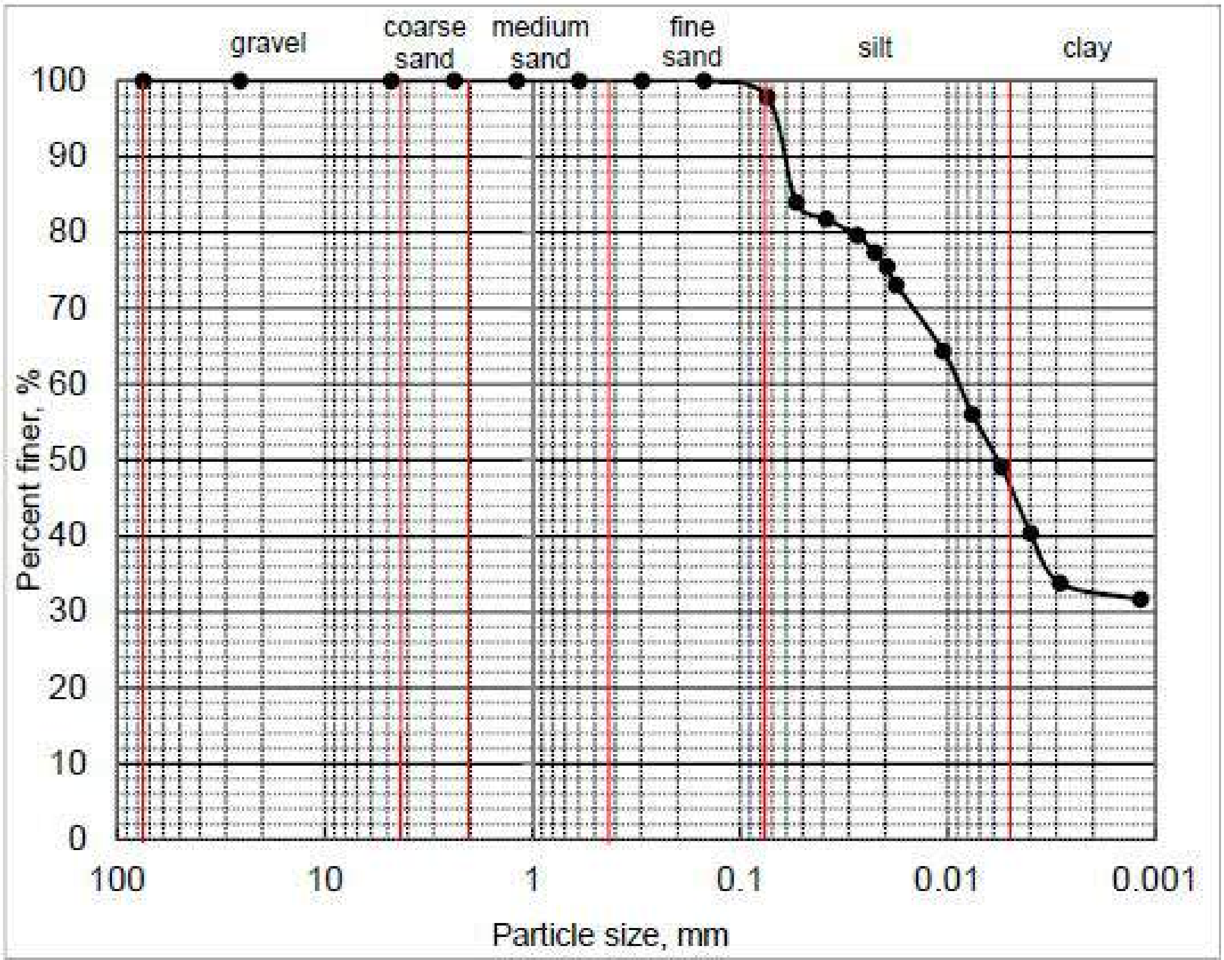
Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
11	1.	0.2	Clay CH – Fat Clay with Sand	49	25	-	-	27	28	2
	2.	0.3	Clay CH - Fat Clay	58	25	-	-	33	30	5
	3.	0.3	Clay CH - Fat Clay	59	25	13	22	34	33	7
	3b.	0.3	Clay CH – Fat Clay	79	-	-	-	52	-	-

*Unified Soil Classification System

The material sampled from the 3 No. trial pits at Site 11 – Dumuria B was very consistent, being classified as a Fat Clay and a Fat Clay with Sand.

The Liquid Limit ranged from 49% to 79%, the Plastic Limit was consistent at 25% and the Plasticity Index ranged between 27% and 52%. Moisture Content ranged between 28% and 33%. The Organic Content ranged between 2% and 7%. One linear shrinkage test was conducted on the sample for TP3 and returned a value of 22%.

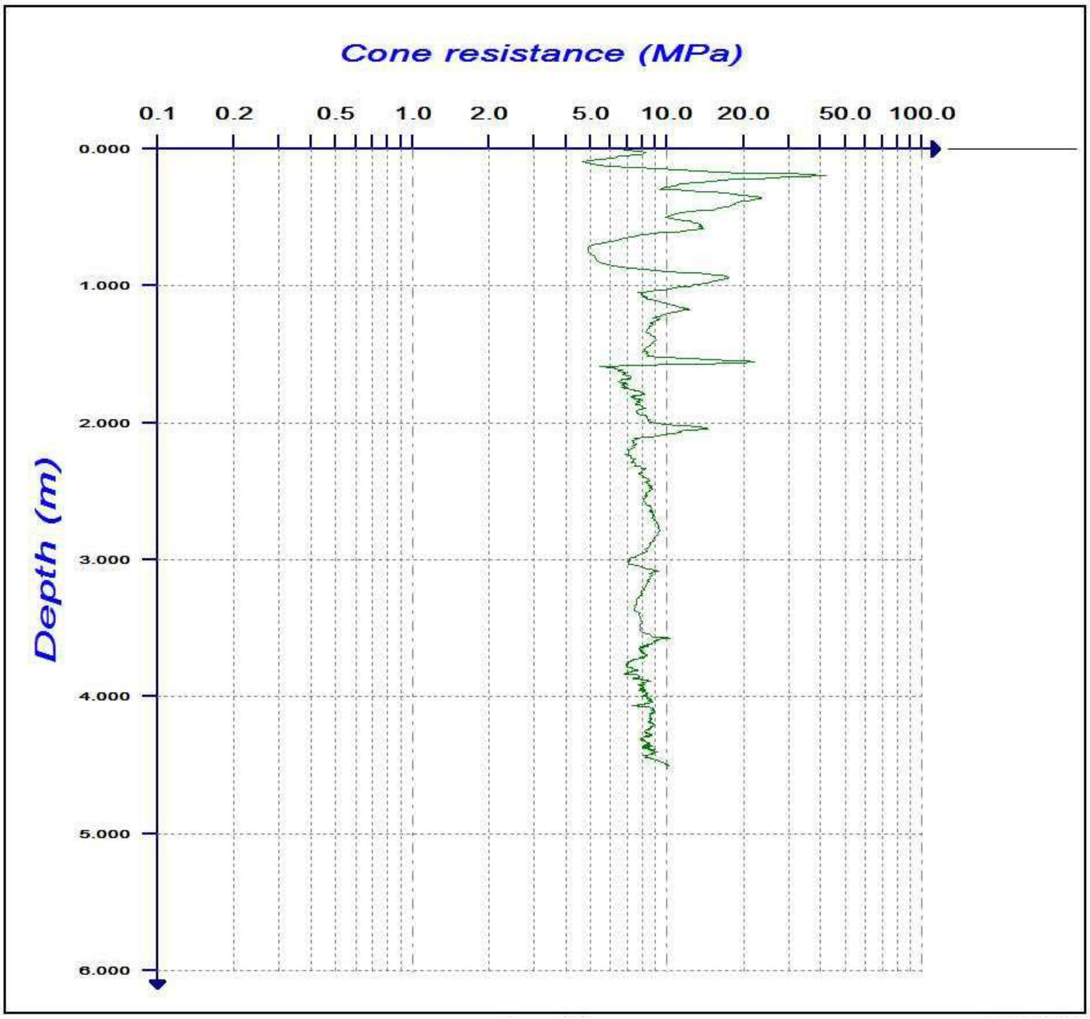
PSD Plot for Sample 3, Site 11



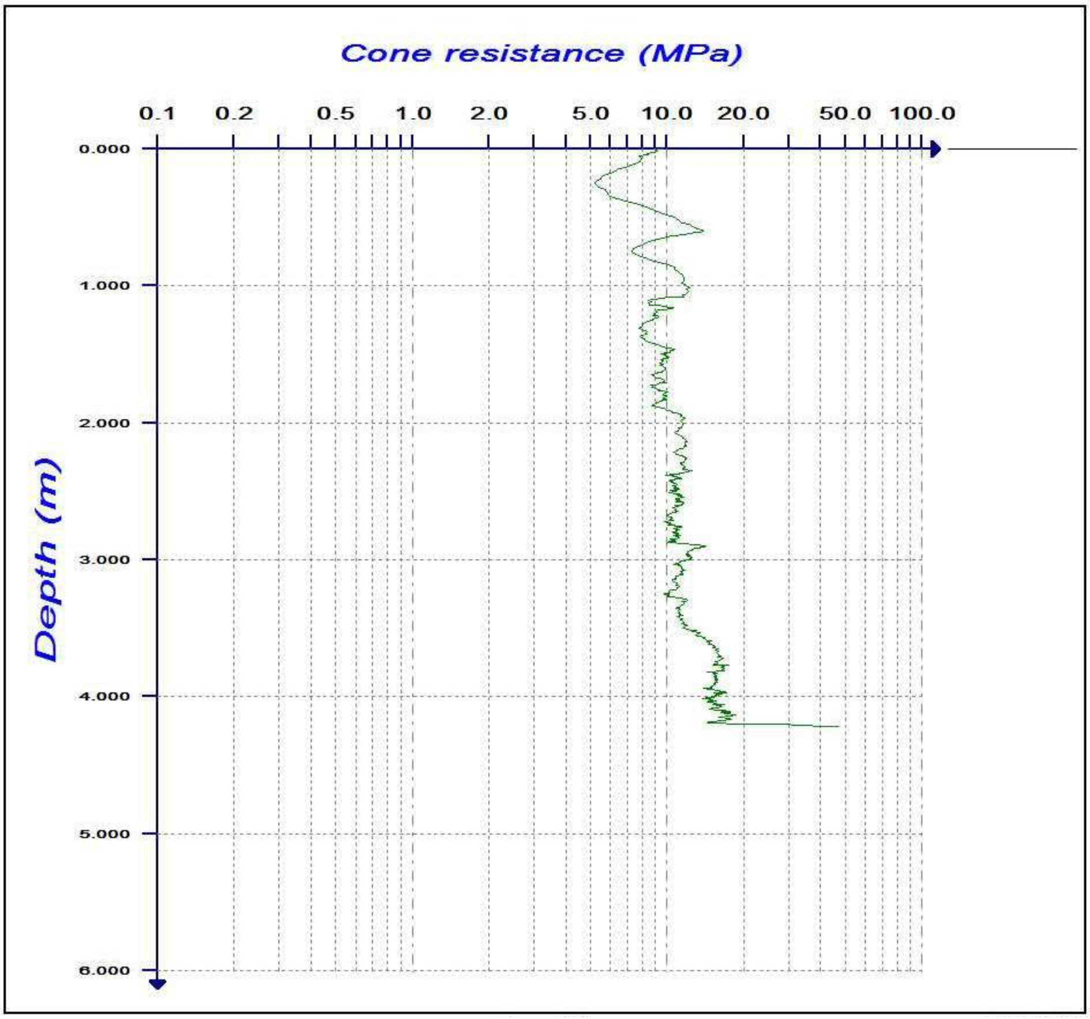
Summary of Panda Probe Results for Site 11 Dumuria B

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
11	Dumuria B	1	4.5	1.6	Variable strength within the embankment, levelling off to a consistent value from 1.0mbgl to the end of the probe.
		2	4.2	1.6	General slight rise in soil strength to end of probe.
		3	4.8	0.9	Reduction in soil strength below road surface from 0 to 0.7 mbgl, before steady increase to 2.0m then levels off to end of probe.

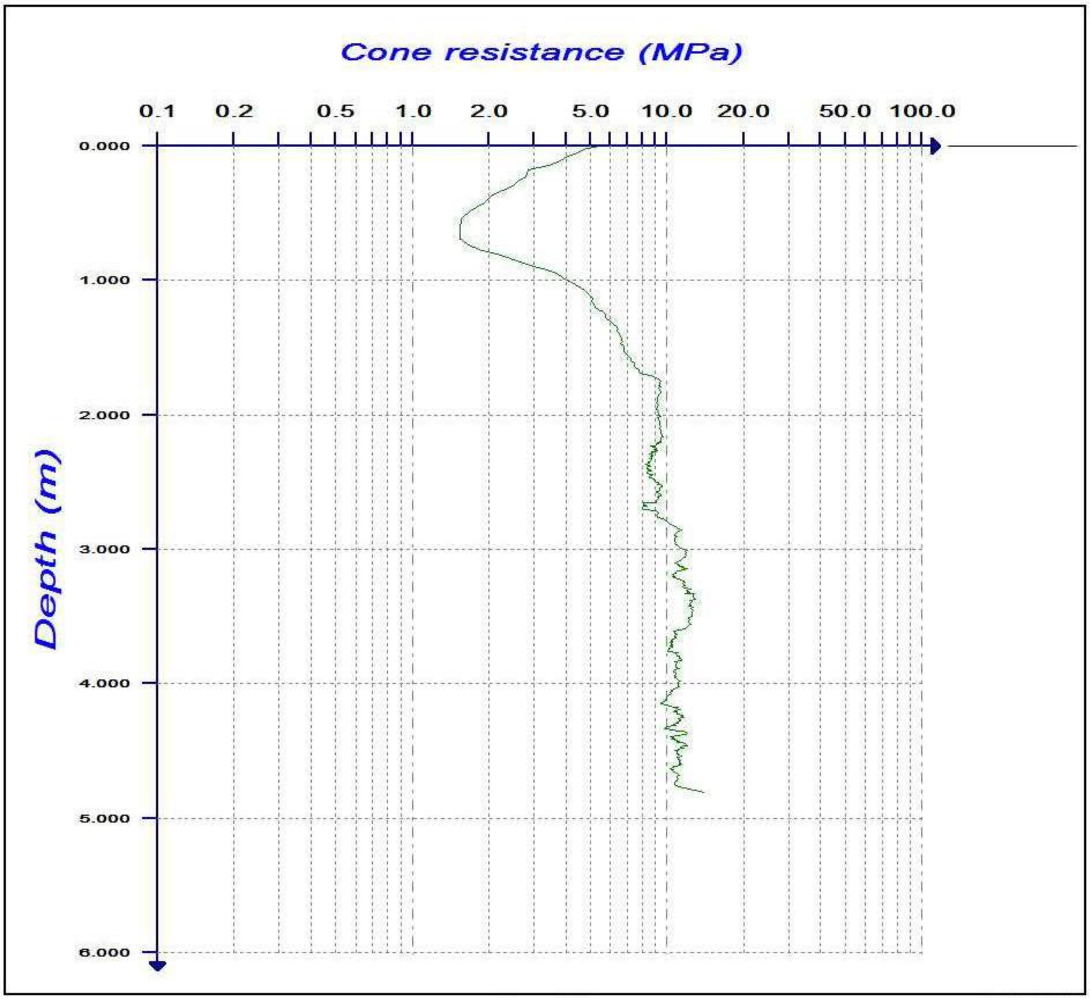
Site 11 Dumuria – Site B 247303003: Test 1



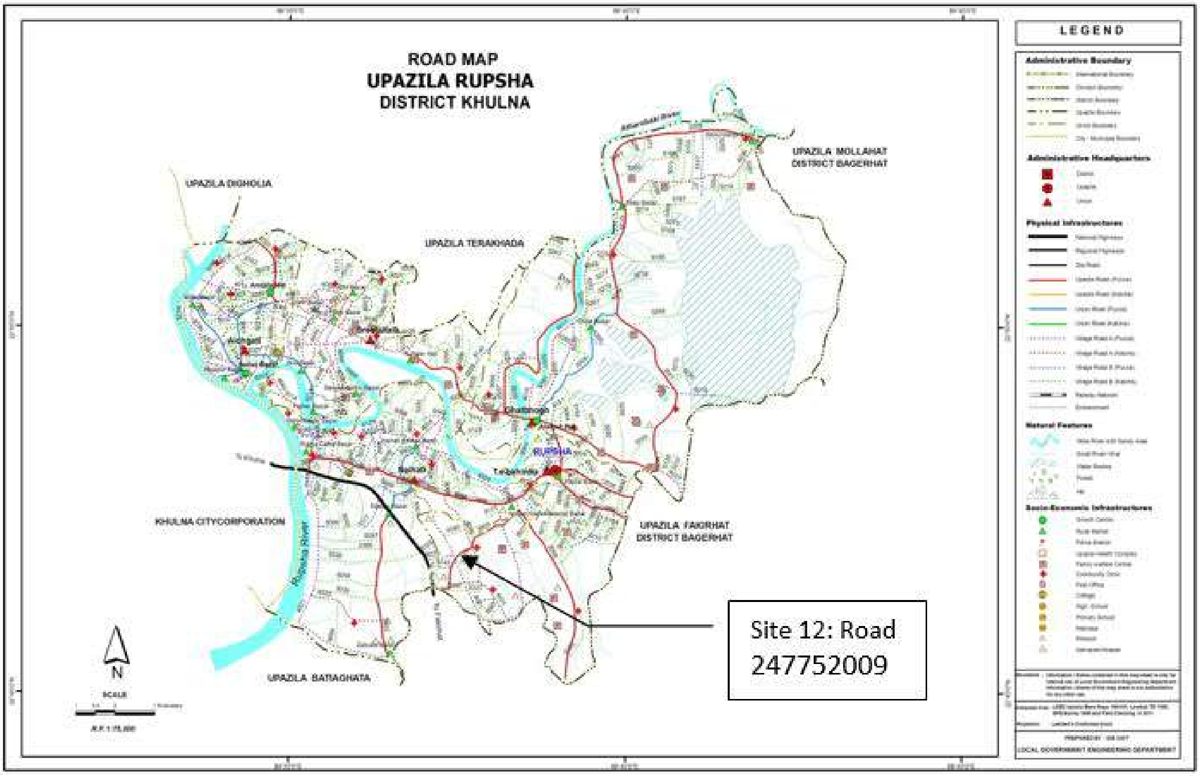
Site 11 Dumuria – Site B 247303003: Test 2



Site 11 Dumuria – Site B 247303003: Test 3



Site 12 Rupsa



Site 12 Description

Site 12 is a single-track carriageway with a flexible bituminous pavement over a herringbone brick subbase. The carriageway is approximately 3.6 m wide with soft shoulders.

The embankment is constructed from organic clay at the bottom and silty clay at the top. The embankment slopes were assessed to be extensively damaged. The side slopes are vegetated with grasses, herbs and mahogany and Shihu trees.

The carriageway shows significant deformation in wheel tracks with associated degradation of the blacktop. There is a large zone where the blacktop appears to be destroyed. Water is collecting in wheel ruts and causing localised softening that is exacerbating the degradation of the running surface. Heavily loaded wagons use the road and are evidently causing considerable damage. Longitudinal cracking is present in pavement surface associated with settlement of the wheel tracks. The road meets a bridge / culvert structure; there is no significant differential settlement between the approach ramp earthwork and the structure itself.

In many places, there is embankment protection provided at the toe of the embankment in the form of 3 m long precast driven piles set out at 0.9 m centres with vertical concrete slabs bolted between a 2-pile arrangement. This embankment toe protection has failed. A large zone of pavement degradation corresponds with a zone where the toe reinforcement exhibits major serviceability failure. A zone of pavement degradation corresponds with a section of embankment slope on which no trees are present with a predominantly grass cover.

The local LGED Engineer reports that the water level comes very close to the pavement surface in the rainy season. Shrimp farming is conducted on both sides of the road embankment. There are culverts in the embankment for cross drainage.

Site 12 Investigation and testing

Investigation at the site comprised 3 No. trial pits and 3 No. Panda Probes. The site investigation locations are detailed below:

Site 12: Rupsa

Summary of soil sampling for Site 12 Rupsa

<table>
<tr><td colspan="2">Site no. 12: Rupsa (Road 247752009)</td></tr>
<tr><td>Trial Pit 1

Lithology:

Bag 1: The soil sample is dark Gray Clayey Silt and it contains plant roots, brick fragments and shells of different organisms.

Bag 2: The soil sample is Gray medium plastic Silty Clay. Plant roots and shells of organisms are present.

Bag 3: The soil sample is dark Gray non-plastic Clayey Silt which contains plant roots and shells of organisms.</td><td>
</td></tr>
<tr><td>Trial Pit 2

Lithology: The soil sample is dark Gray Clay with minor amount of fine Sand. The Clay shows moderate plasticity and contain plant roots, brick fragments.</td><td>
</td></tr>
</table>

Test Pit No. 2a (on embankment)

Lithology: The soil sample is dark Gray Silty Clay. Clay shows medium plasticity, plant roots and brick fragments.

Trial Pit 3

Lithology: The soil sample is dark Gray Clay. The Clay shows high plasticity. Some reddish layers of Clayey soil ids present within the Gray Clay, which can be caused by the oxidation of ferruginous materials. Soil contains plant roots and Sand size brick fragments.

Summary of Laboratory Test Results for Site 12 Rupsa

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
12	1.	0.3	Clay CL – Lean Clay	44	23	-	-	21	25	7
	2.	0.3	Clay CL – Lean Clay	48	20	-	-	28	28	8
	3.	0.3	Clay CH - Fat Clay	54	26	10	36	29	32	8

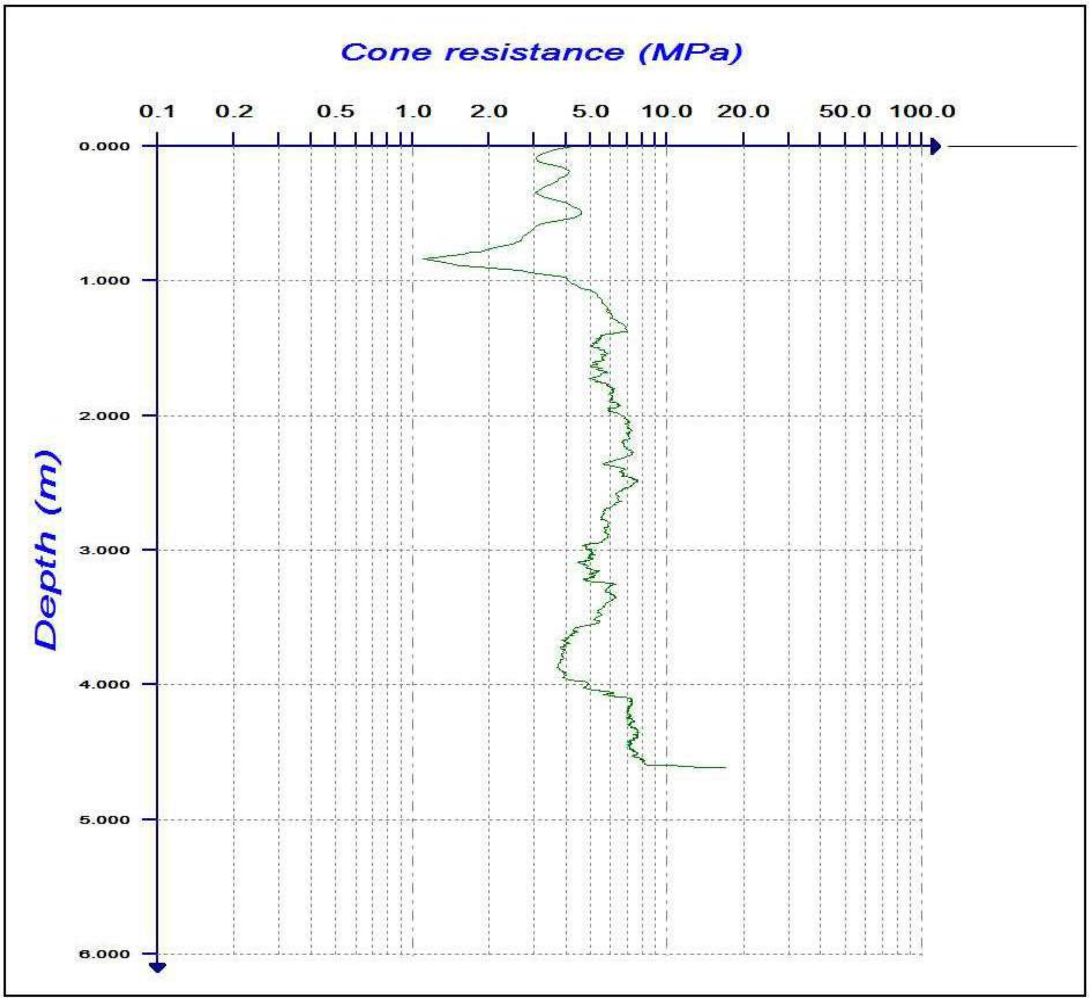
*Unified Soil Classification System

The material sampled at the 3 No. trial pits at Site 12 – Rupsa varied between a Lean Clay and a Fat Clay. The Liquid Limit ranged from 44% to 54%, the Plastic Limit ranged from 20% to 26% and the Plasticity Index ranged from 21% to 29%. The Moisture Content ranged from 25% to 32% whilst the Organic Content ranged from 7% to 8%. One linear shrinkage test was conducted on sample from TP 3, the result returned was 36%.

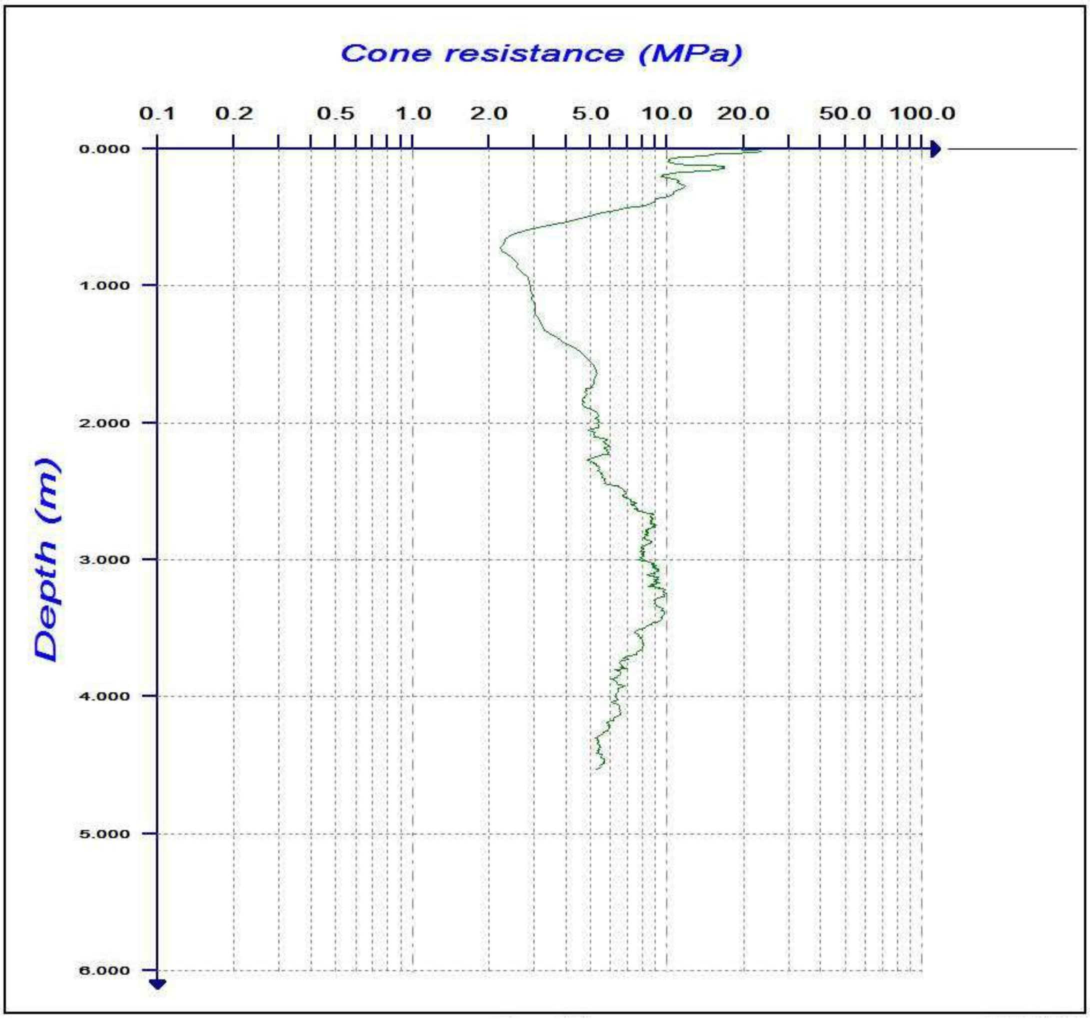
Summary of Panda Probe Results for Site 12 Rupsa

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
12	Rupsa	1	4.6	0.4	General slight rise in soil strength to end of probe. Zone of reduced strength from 0.6 to 0.9mbgl
		2	4.5	1.5	Reduction in soil strength below road surface from 0 to 0.7 mbgl, before steady increase to 3.0m then slight reduction to end of probe.
		3	4.8	1.8	Reduction in soil strength below road surface from 0.1 to 0.9 mbgl, before steady increase to 1.8 m then levels off to end of probe.

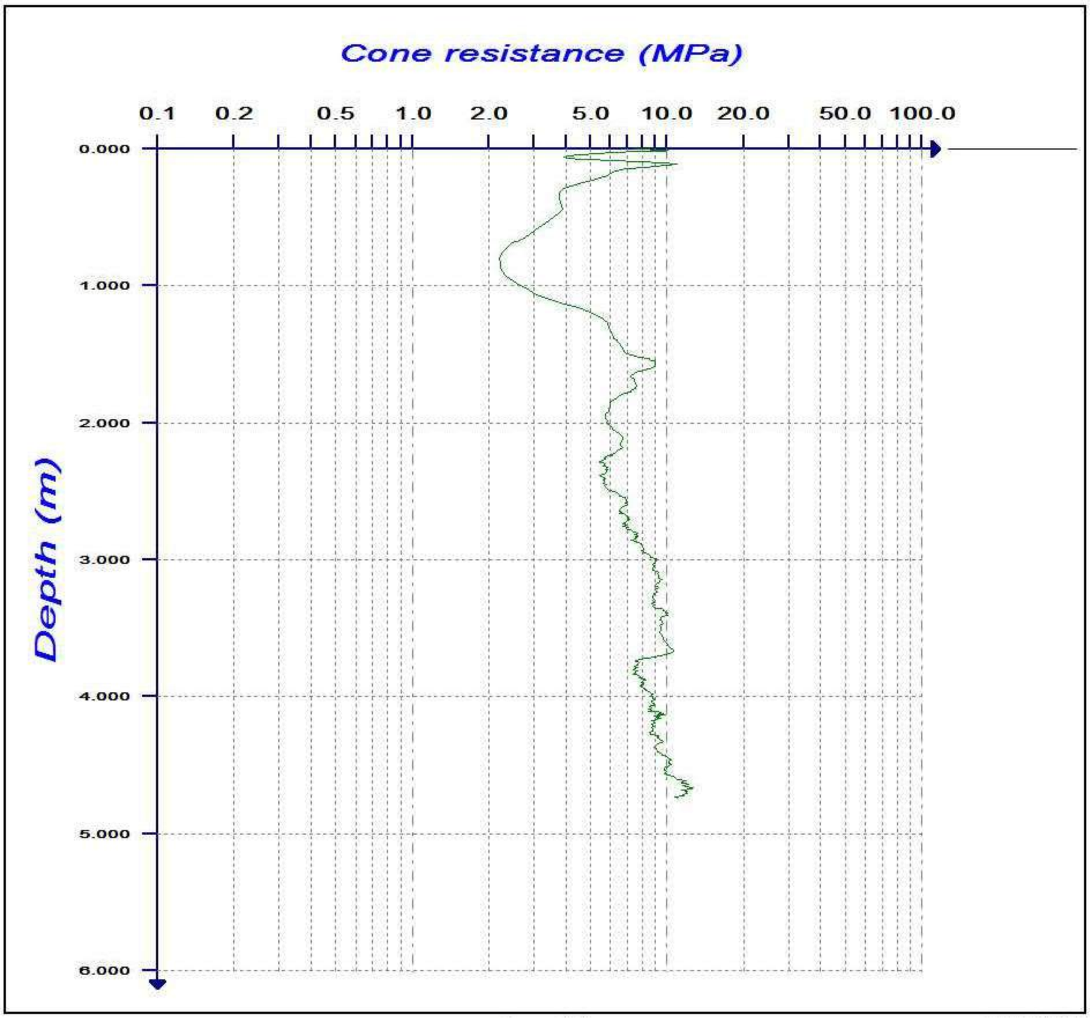
Site 12 Rupsa 47752009:– Test 1



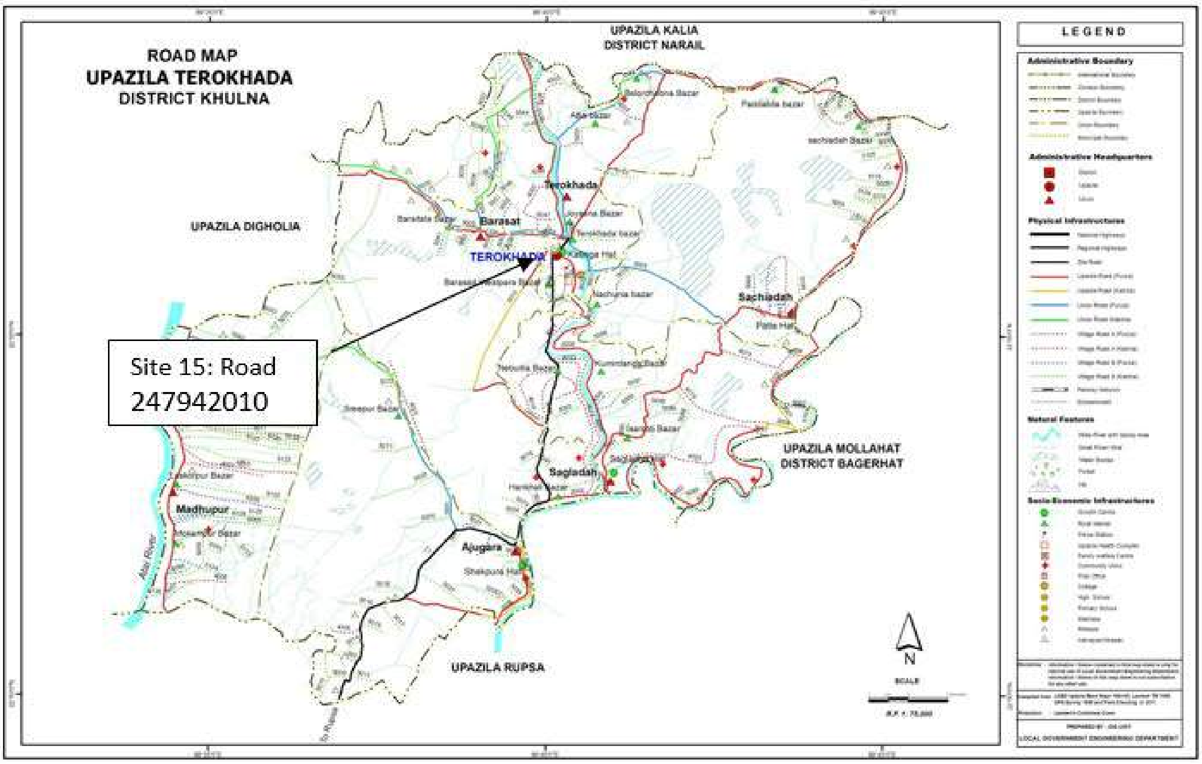
Site 12 Rupsa 247752009: Test 2



Site 12 Rupsa 247752009: Test 3



Site 15 Terokhada - A



Site 15 Description

Site 15 is a single-track carriageway with a flexible bituminous pavement. The carriageway is approximately 3 m with a soft shoulder. The embankment is constructed from organic clay at the bottom and silty clay at the top. The side slopes of the embankment are vegetated with grasses, herbs and trees of various species.

The road has significantly settled on the slope / water side of the carriageway and a large area of pavement has been lost. The road is being trafficked by wagons and deformation of the road along the wheel tracks is evident. Vehicles are also pulling onto the verge to allow other vehicles to pass which is damaging the edge of the pavement construction and the crest of the slope. There is significant loss of pavement on a section of embankment where there are no trees present on the LHS slope. The road is muddy with deep wheel ruts which hold moisture and cause localised softening, exacerbating the issues.

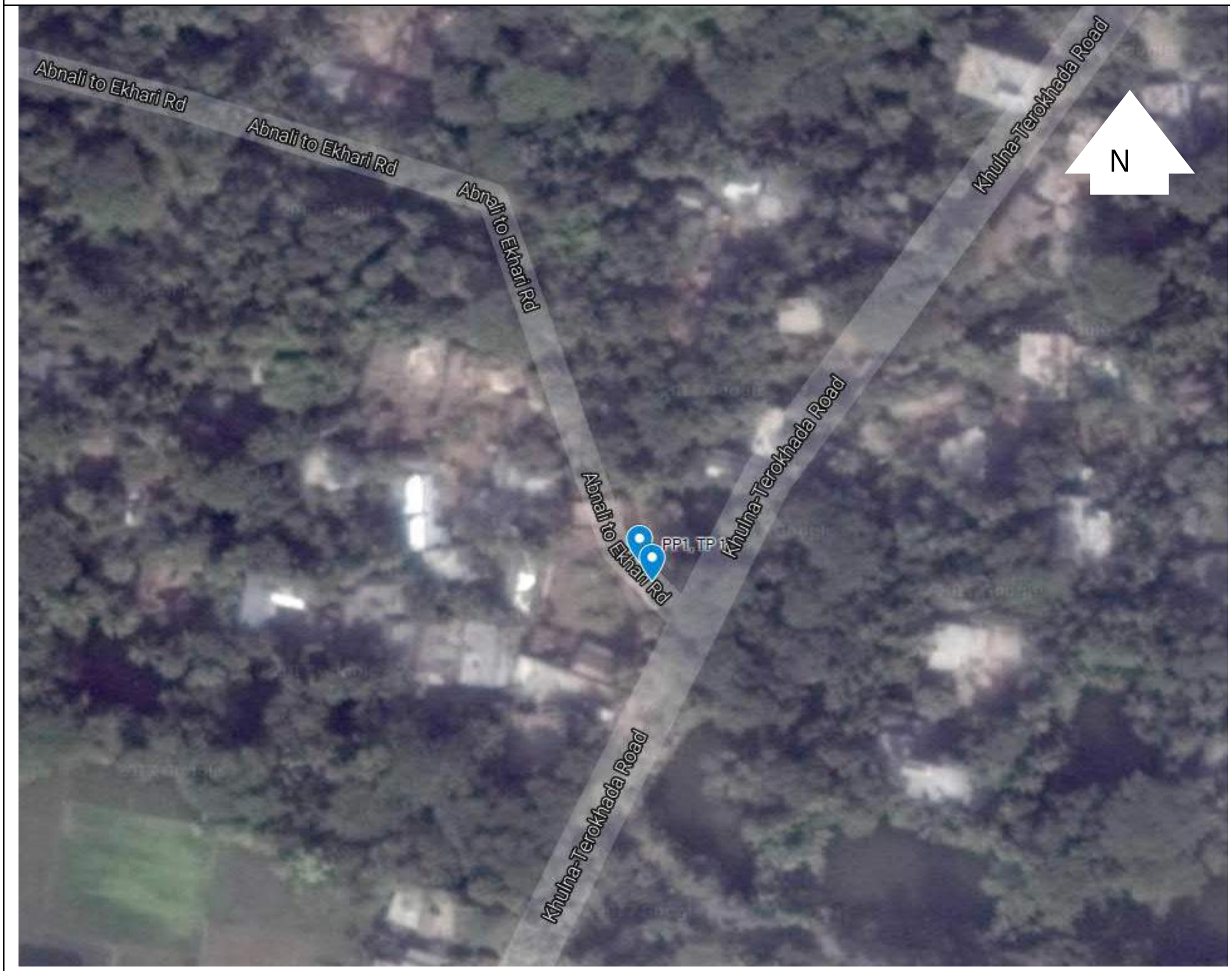
The toe of the slope has the typical driven pile and board support arrangement with again is leaning out towards the water.

The local LGED Engineer reports that water levels come close to the pavement surface in the rainy season. There are culverts in the embankment for cross drainage.

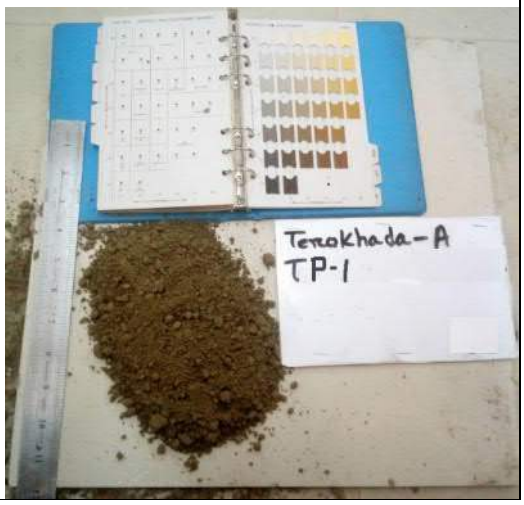
Site 15 Investigation and testing

Investigation at the site comprised 2 No. trial pits and 2 No. Panda Probes. The site investigation locations are detailed below:

Site 15: Terokhada A



Summary of soil sampling for Site 15 Terokhada A

Site no. 15: Terokhada A (Road 247942010)	
Trial Pit 1 Lithology: Brown Sandy Silt with plant root partly decomposed wood fragments.	

Trial Pit 2

Lithology: The soil is Grayish Brown very fine Sandy Silt with plastic Clayey soil and plant roots.

Summary of Laboratory Test Results for Site 15 Terokhada A

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
15	1.	0.3	Clay CL – Lean Clay	41	21	-	-	20	19	-
	2.	0.3	Clay CL – Lean Clay	42	21	-	-	21	14	6

*Unified Soil Classification System

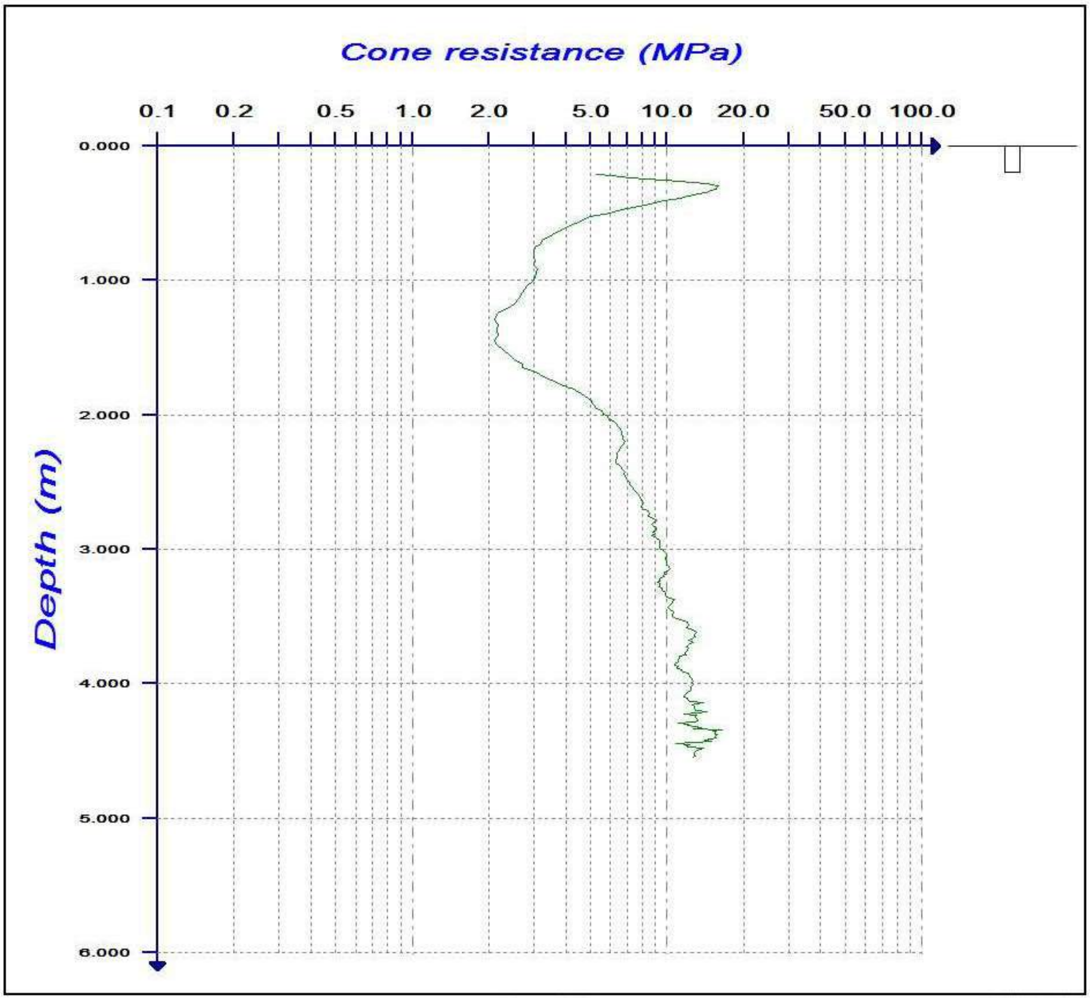
The samples retrieved from the 2 No. trial pits at Site 15 – Terokhada A were extremely consistent. Both samples were classified as Lean Clay.

The Liquid Limit ranged from 41% to 42%, the Plastic Limit was consistent at 21% and the Plasticity Index ranged from 20% to 21%. The Moisture Content ranged from 14% to 19%.

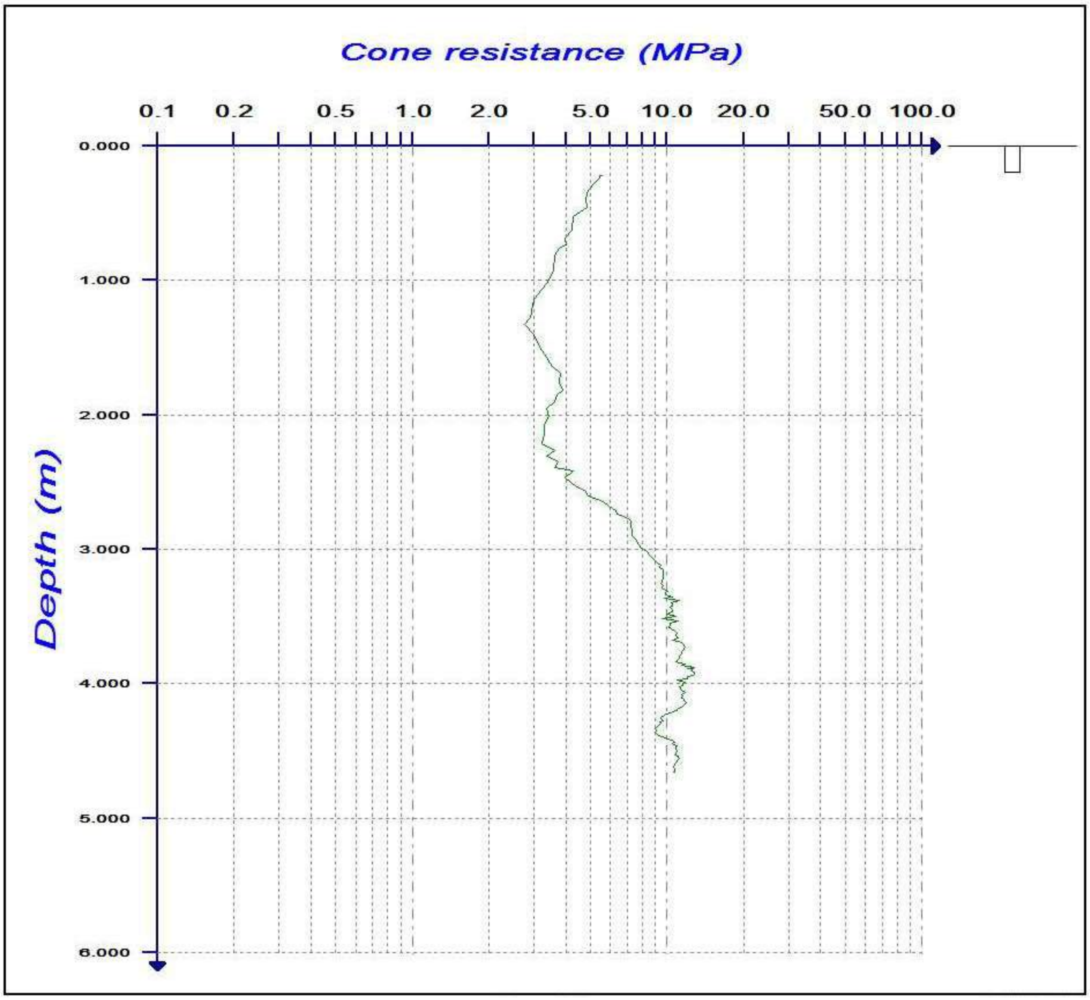
Summary of Panda Probe Results for Site 15 Terokhada A

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
15	Terokhada A	1	4.5	2.2	Reduction in soil strength below road surface from 0.3 to 1.5 mbgl, before steady increase to end of probe.
		2	4.6	2.1	Slight reduction in soil strength below road surface from 0.2 to 1.3 mbgl, before steady increase to end of probe.

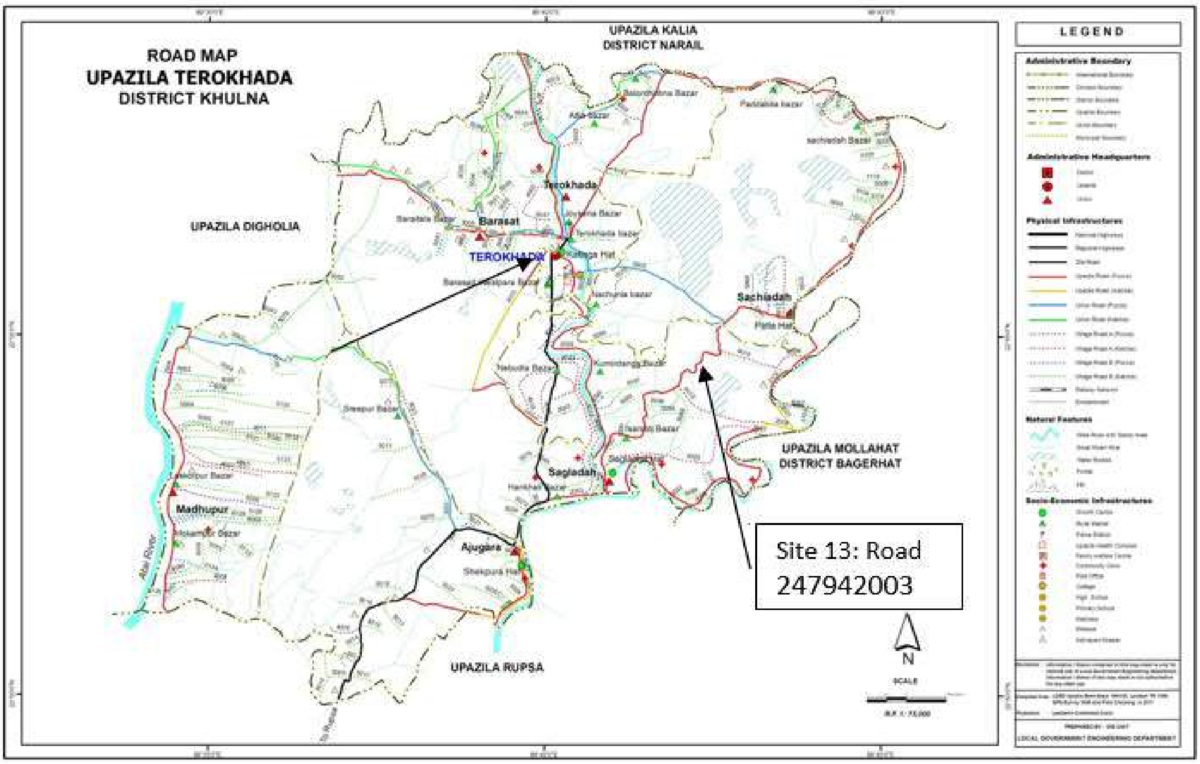
Site 15 Terokhada – Site A 247942010: – Test 1



Site 15 Terokhada – Site A 247942010: Test 2



Site 13 Terokhada - B



Site 13 Description

Site 13 is a single lane carriageway with a flexible bituminous pavement. The carriageway is approximately 3.6 m wide with a soft shoulder.

The embankment is constructed from organic clay at the bottom and silty clay at the top. The slopes were recorded as significantly damaged and are vegetated with grasses, herbs and various species of tree. Some trees are leaning out from the slope whilst others are vertical. It is possible that the trees are tilting under their own self weight as the soft soil cannot support the root ball.

A canal has been created on the RHS of the road. Major erosion and damage has occurred to the highway slope that runs into the canal. There is significant slope regression where material is falling into the canal and this regression has extended into the pavement at 2 locations forming two concave areas of pavement loss with each extending approximately 1m into road from the edge.

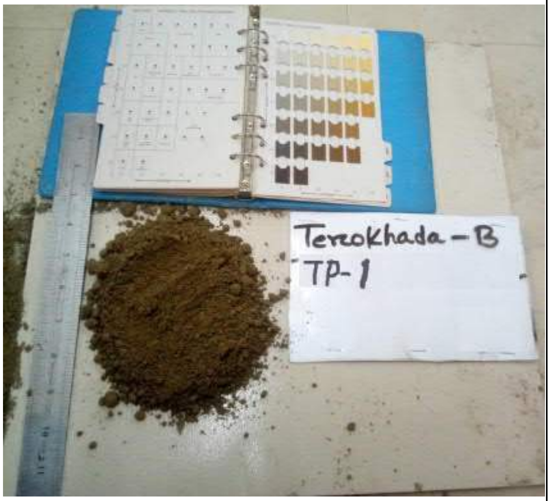
The local LGED Engineer reports that water levels come close to the pavement surface in the rainy season.

Site 13 Investigation and testing

Investigation at the site comprised 3 No. trial pits and 3 No. Panda Probes. The site investigation locations are detailed below:

Site 13: Terokhada B

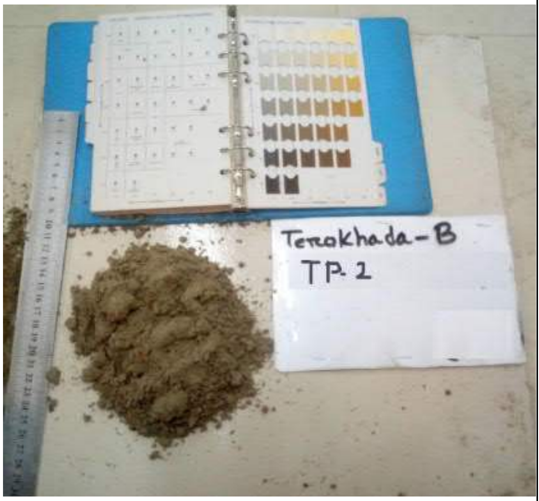

Summary of soil sampling for Site 13 Terokhada B

Site no. 13: Terokhada B (Road 247942003)	
Trial Pit 1 Lithology: Brown Silty Clay and plant roots.	

Trial Pit 2

Lithology:

Gray Silt with lenses of Clayey soil, plant roots and brick fragments.



Trial Pit 3

Lithology: Brown Sandy Silt with plant root partly decomposed wood fragments.

Summary of Laboratory Test Results for Site 13 Terokhada B

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
13	1.	0.3	Clay CL – Lean Clay	43	28	-	-	17	17	-
	2.	0.3	Clay CH - Fat Clay	47	21	11	28	26	26	4.5
	3.	0.3	Fine Sand SM – Silty Sand	-	-	-	-	-	-	-

*Unified Soil Classification System

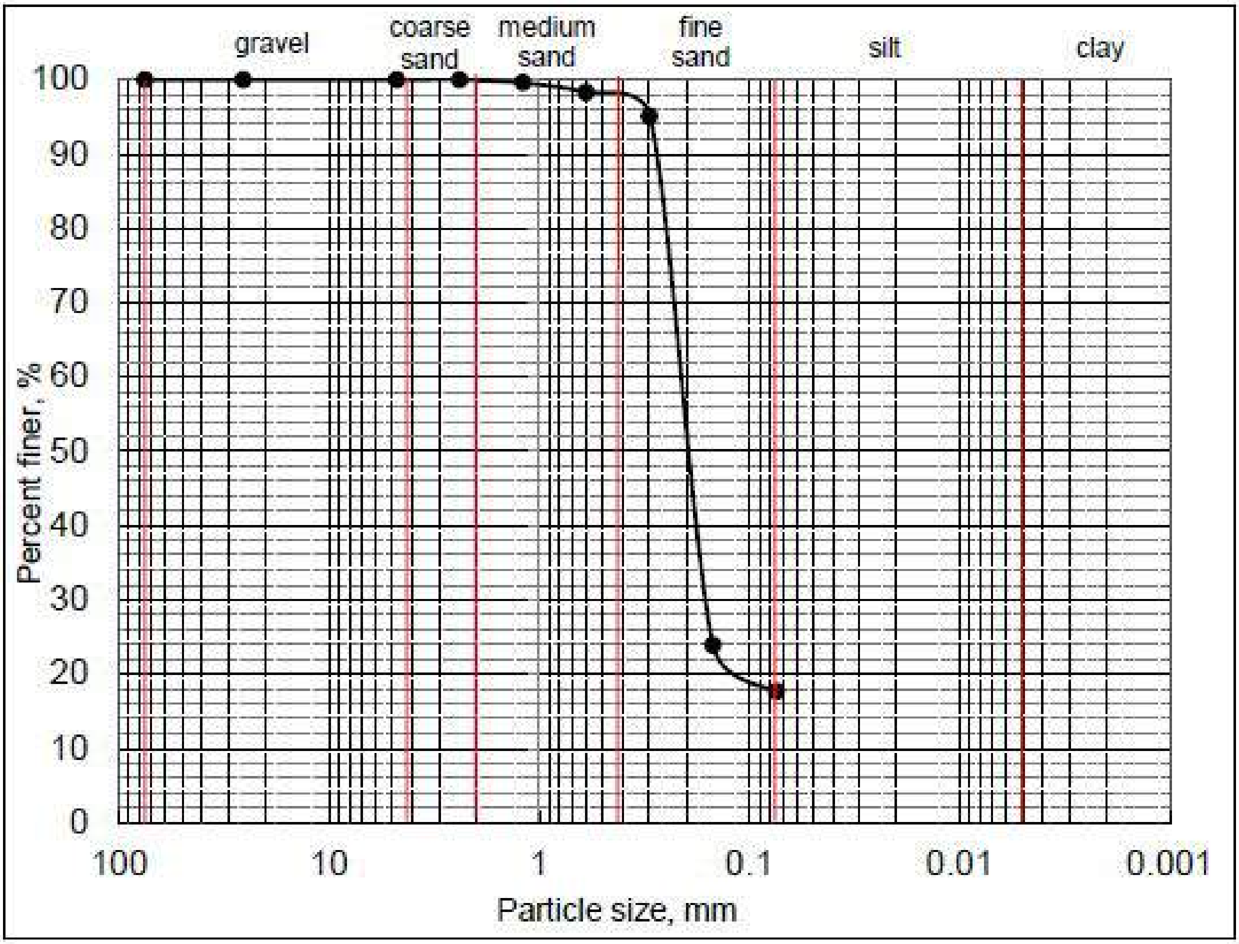
The samples retrieved from the 3 No. trial pits at Site 13 – Terokhada B, all had a different classification: Trial Pit 1 was a Lean Clay; Trial Pit 2 was a Fat Clay and Trial Pit 3 was a Silty Sand.

For the cohesive material, Liquid Limit ranged from 43% to 47%, Plastic Limit ranged from 21% to 28% and Plasticity Index ranged from 17% to 26%. The Moisture Content ranged from 17% to 26%. One linear shrinkage test was conducted on the sample from TP2 and returned a value of 28%.

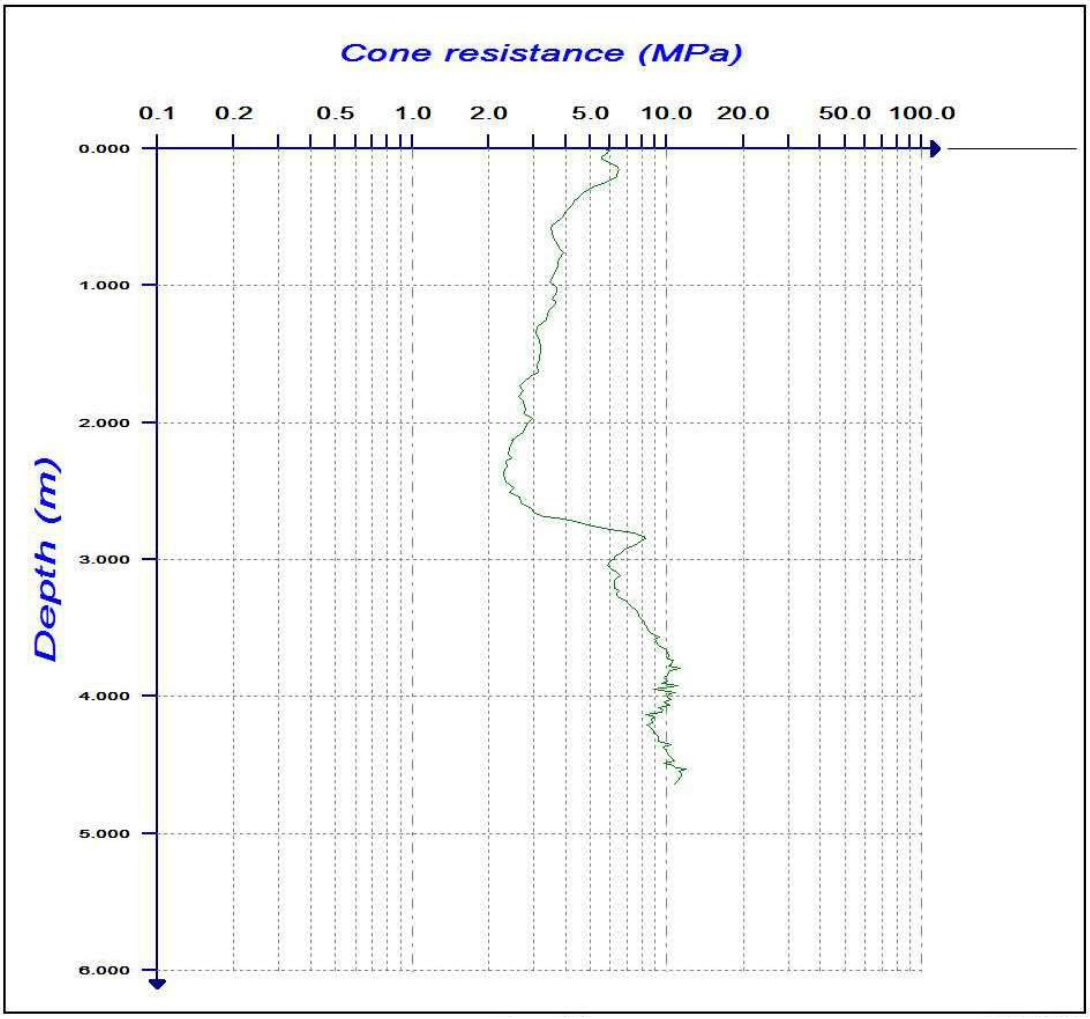
Summary of Panda Probe Results for Site 13 Terokhada B

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
13	Terokhada B	1	4.6	5.0	Slight reduction in soil strength below road surface from 0.0 to 2.4 mbgl, before sharp, then steady increase to end of probe.
		2	4.6	5.0	Reduction in soil strength below road surface from 0.0 to 0.9 mbgl, before levelling off to 2.0m, then steady increase to end of probe.
		3	4.6	5.0	General slight rise in soil strength to end of probe.

PSD result for Sample 3, Site 13

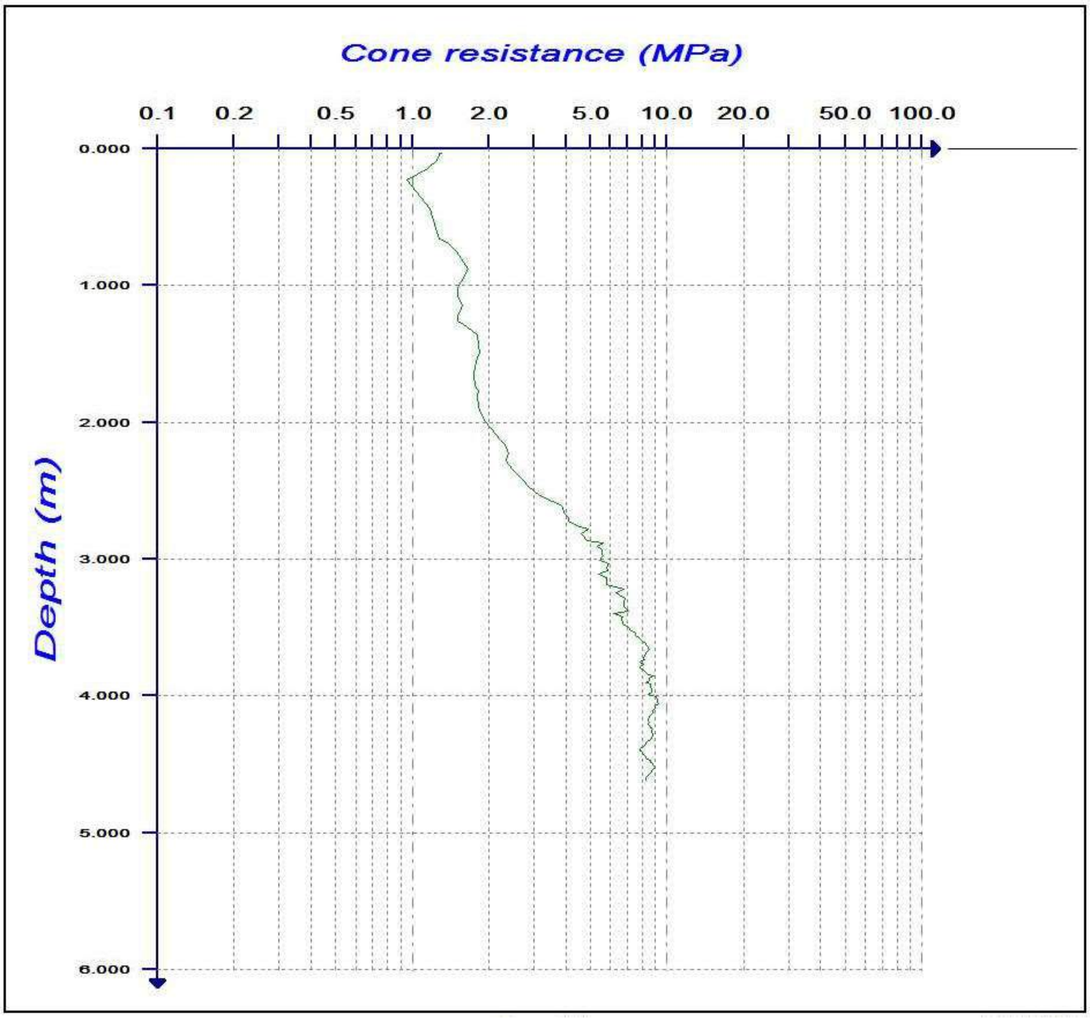


Site 13 Terokhada – Site B 247942003: Test 1

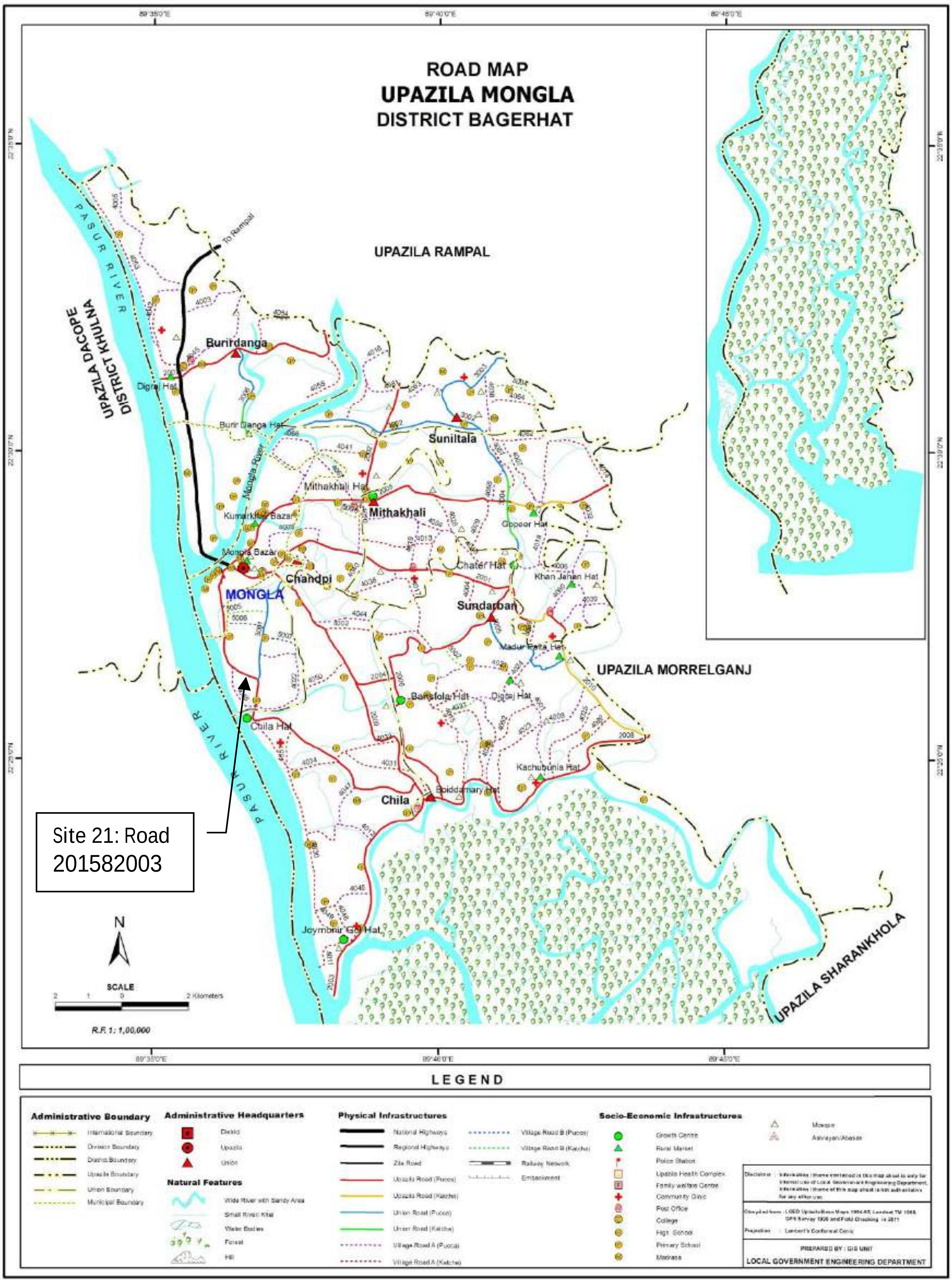


Site 13 Terokhada – Site B 247942003: Test 2

Site 13 Terokhada – Site B 247942003: Test 3



Site 21 Mongla A & B



Site 21 Description

Site 21 is a single lane carriageway with a flexible bituminous pavement. The carriageway is approximately 3.6 m wide with soft shoulder. The road embankment is constructed from silty clay. The slopes were recorded during the inspection as being damaged. The slopes are vegetated with grasses, herbs and trees of various species. There is minimal vegetation on water side of the road and thick vegetation on landward side of the road.

The road surface is deformed along wheel tracks but more so on the water side of the carriageway. Associated with the deformation is the typical longitudinal cracking seen elsewhere. Patches of blacktop loss are evident with the typical red colour that indicates the exposure and degradation of the underlying brick subbase.

The slope line is bulging out at a point that corresponds with a large patch of pavement loss. The unsupported road side has slumped into the water, impacting on up to 0.3m of adjacent pavement. There is minimal vegetation along the water side.

There is a marked difference in level between the earthwork and bridge, indicative of differential settlement. There are significant cavities on the RHS of the single-track road where it meets the bridge, and material that has been lost under the bridge wing wall, where the wall has been undermined. The earthwork slope and pile/board retaining structure is failing and moving towards water, undermining the wing wall.

There is loss of slope material at the waterline; material has likely been washed-out by the drainage pipe that emanates from the embankment side. The slope has been cleared of trees and possibly dug into to facilitate the construction of stilted huts by locals

In many places there is embankment protection at the toe of the embankment in the form of 3m long precast driven piles spaced at 0.9m centres, with vertical concrete boards bolted in a 2 pile arrangement. The piles are leaning out to differing degrees and the boards are misaligned.

The local LGED Engineer reports that water levels come close to the pavement surface in the rainy season. There are culverts in the road embankment for cross drainage.

Site 21 Investigation and testing

Investigation at the site comprised 4 No. trial pits and 4 No. Panda Probes. The site investigation locations are detailed below:

Site 21: Mongla A & B

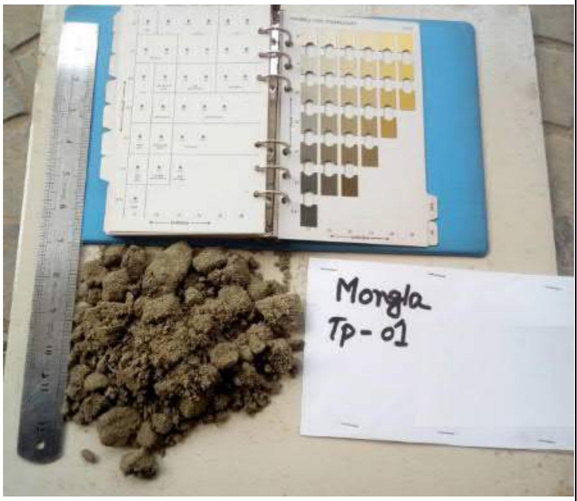
Summary of soil sampling for Site 21 Mongla A & B

Site no. 21: Mongla (Road 201582003)

Trial Pit 1

Lithology:

The soil sample composed of Sandy Clay where the Sand is Gray in colour, loosely compact. The Clay is also Gray in colour and shows moderate plasticity. The soil sample contains extensive plant roots and reddish spots which may form by the oxidation of soil.

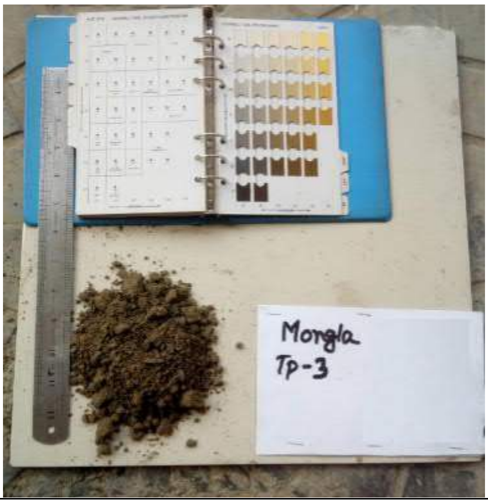


Trial Pit 2

Lithology: The soil is light Brownish Gray fine size Sand with small fragments of non-plastic Clayey soil. The Clayey soil is reddish Brown in colour. Soil sample contains plants roots and brick fragments

Trial Pit 3

Lithology: The soil sample is Gray Sandy Silt with minor Clay in it. The Sand is very fine size and Clay shows moderate plasticity. Soil sample contain plant roots and semi decomposed wood fragments.



Summary of Laboratory Test Results for Site 21 Mongla A & B

Site No.	Trial Pit No.	Sample Depth (mbgl)	Visual Classification USCS* Classification	LL (%)	PL (%)	SL (%)	L_s (%)	PI (%)	Moisture Content (%)	Organic Content (%)
21	1.	0.25	Clay CH – Fat Clay with Sand	54	21	-	-	33	27	-
	2.	0.25		-	-	-	-	-	-	-
	3.	0.3	Clay CL – Lean Clay	41	24	-	-	17	-	-
	4.	0.3	Fine Sand SM - Silty Sand	-	-	-	-	-	-	-

* Unified Soil Classification System

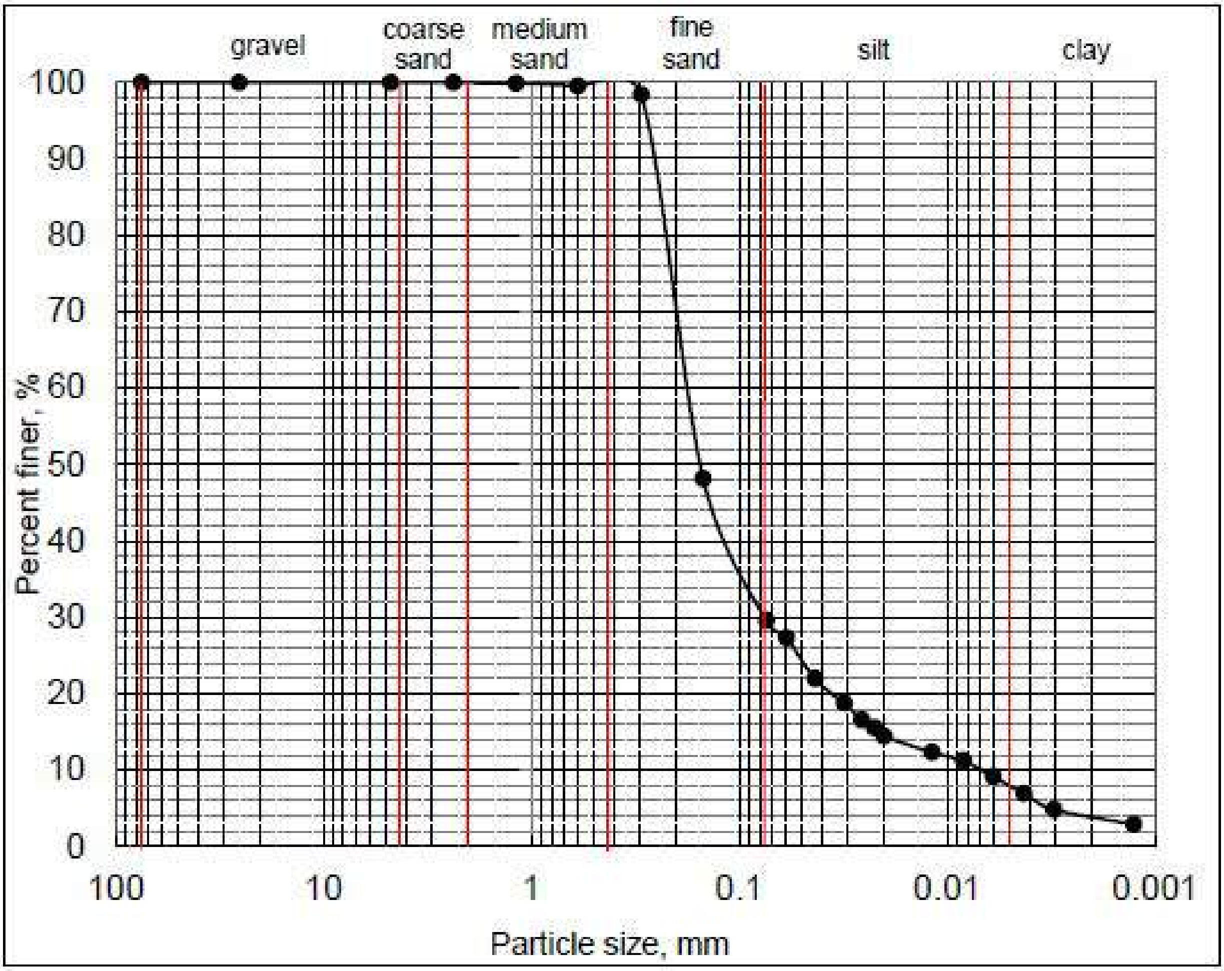
There was variation in the material received from Mongla. The material sampled from site 'Mongla A' was a Fat Clay with Sand. The material sampled from 'Mongla B' was a Lean Clay and a Silty Sand.

For the cohesive soil taken from Trial Pit 1 and Trial Pit 3 the Liquid Limit ranged from 41% to 54%, the Plastic Limit ranged from 21% to 24% and the Plasticity Index ranged from 17% to 33%. The Moisture Content for the sample from Trial Pit 1 was 27%.

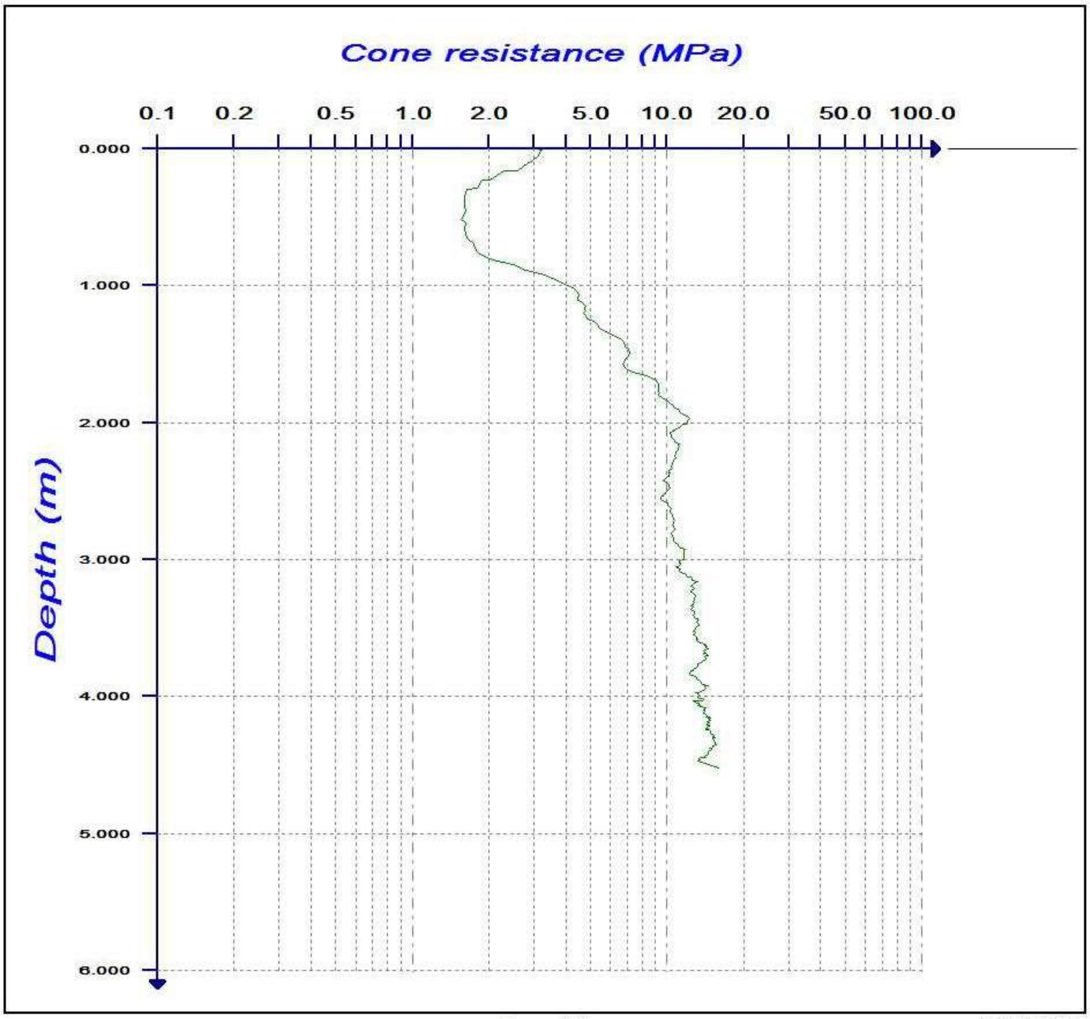
Summary of Panda Probe Results for Site 21 Mongla A & B

Site Number	Site Name	Test Number	Final Depth (mbgl)	Embankment Height (m)	Summary of Penetration
21	Mongla A & B	1	4.5	0.5	Reduction in soil strength below road surface from 0.1 to 0.4 mbgl, before steady increase to 2.0 m then levels off to end of probe.
		2	4.5	0.5	General slight rise in soil strength to end of probe.
		3	4.5	0.48	Reduction in soil strength below road surface from 0.2 to 0.7 mbgl, before increasing to 2.0m, then levelling off to end of probe
		4	4.5	0.48	Reduction in soil strength below road surface from 0.2 to 0.8 mbgl, before slight increase to end of probe

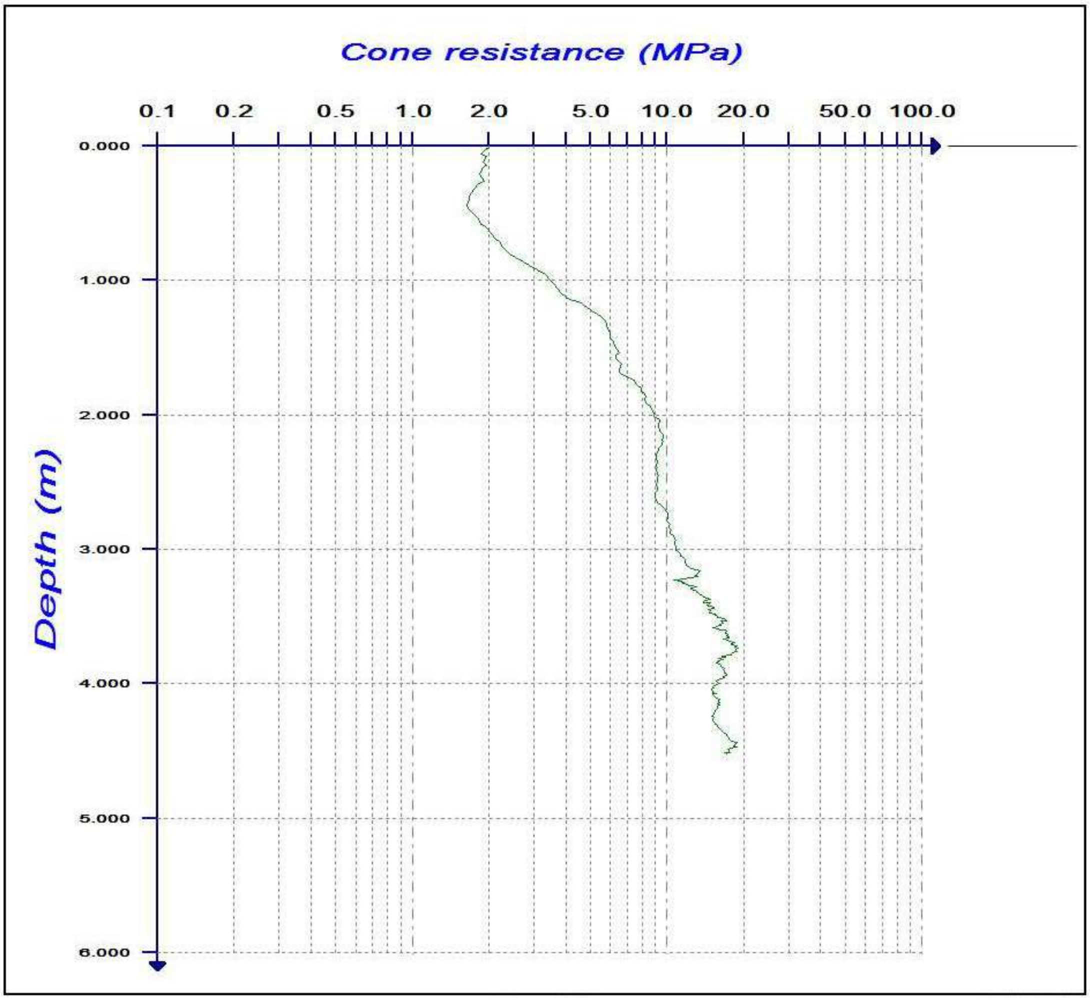
PSD Result for Sample 4, Site 21



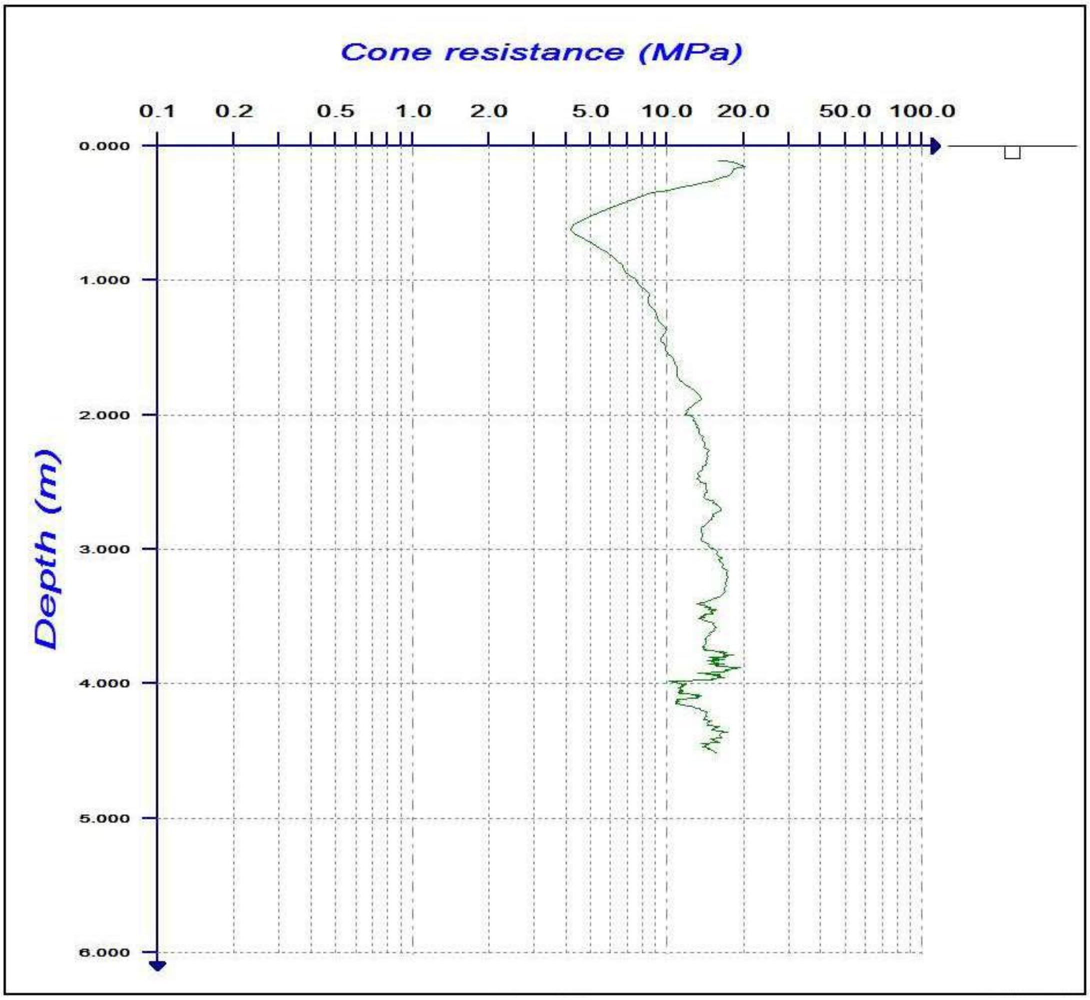
Site 21 Mongla 201582003: Test 1



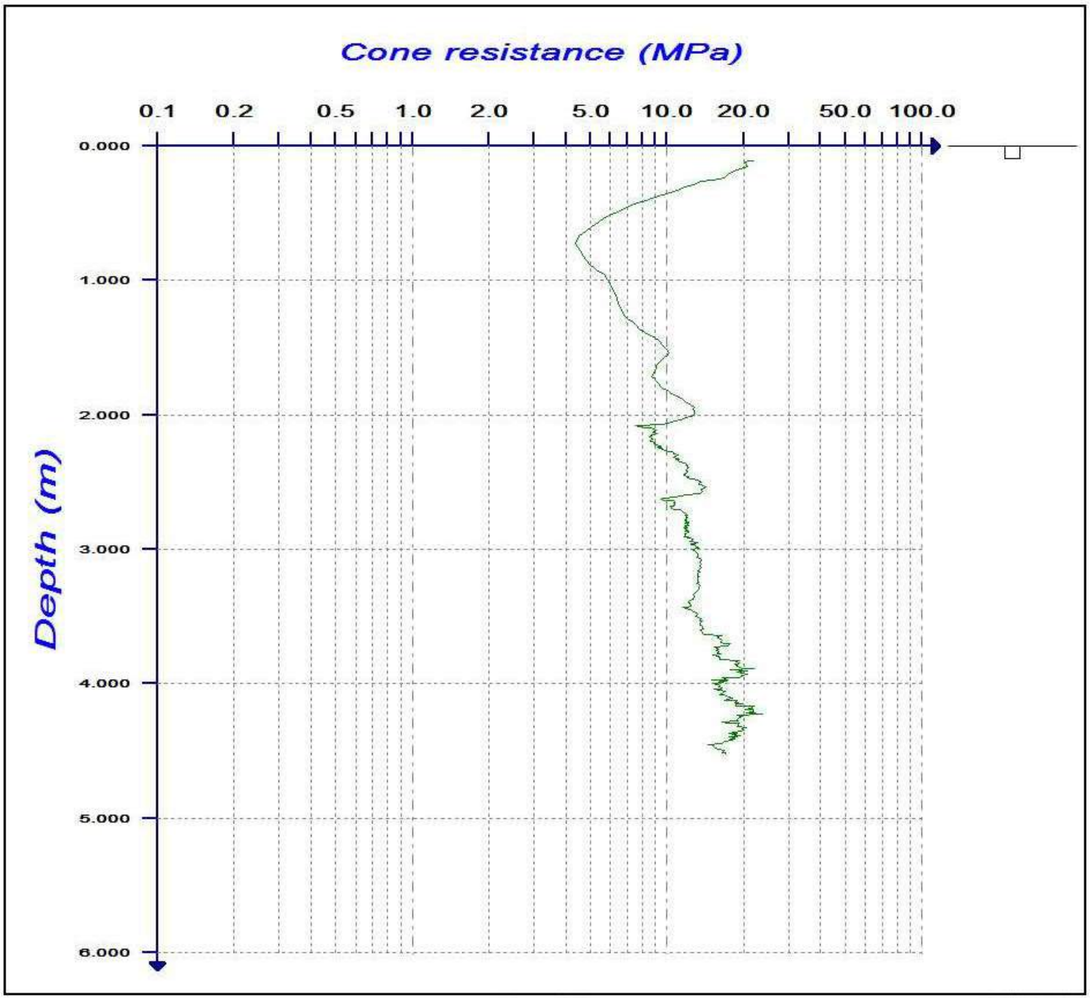
Site 21 Mongla 201582003: Test 2



Site 21 Mongla 201582003– Test 3



Site 21 Mongla 201582003 – Test 4



Appendix F: Ground Models

Site 3 Assasuni Observational Ground Model

The site demonstrates loss of edge support, poor surface conditions, and over-steep slopes / erosion.

Key features	Site photograph
Erosion, over-steepening of the embankment slopes at Site 3	

The embankment fill consists of Fat Clay material and generally has a cone resistance that reduces from the near surface layers to the base of the embankment illustrating probable poor compaction and consequently higher water content, leading to low strength and support characteristics. Little self-weight compaction is likely due to the low height of the embankment, with only the near surface pavement layers (comprising higher quality material than the general embankment fill) demonstrating significant resistance to cone penetration.

The foundation soil sampled underlying the embankment consists of a Fat Clay. The cone resistance does show some local variability in resistance that is likely to the result of sandier or more organic material. The foundation soil has been subject to loading from the embankment above and an increase in resistance between 5 and 10 MPa from 1.0 to 2.5 mbgl may be the result of consolidation over time (although this cannot be confirmed). Beyond 3mbgl, there seems to be little increase in cone resistance with depth. Notwithstanding the low load (1.2-1.5m high embankment), the anticipated deformation resulting is estimated as ~100mm of vertical deformation (see Section 6.3 Analytical Ground Model) with the inherent impact on surface conditions.

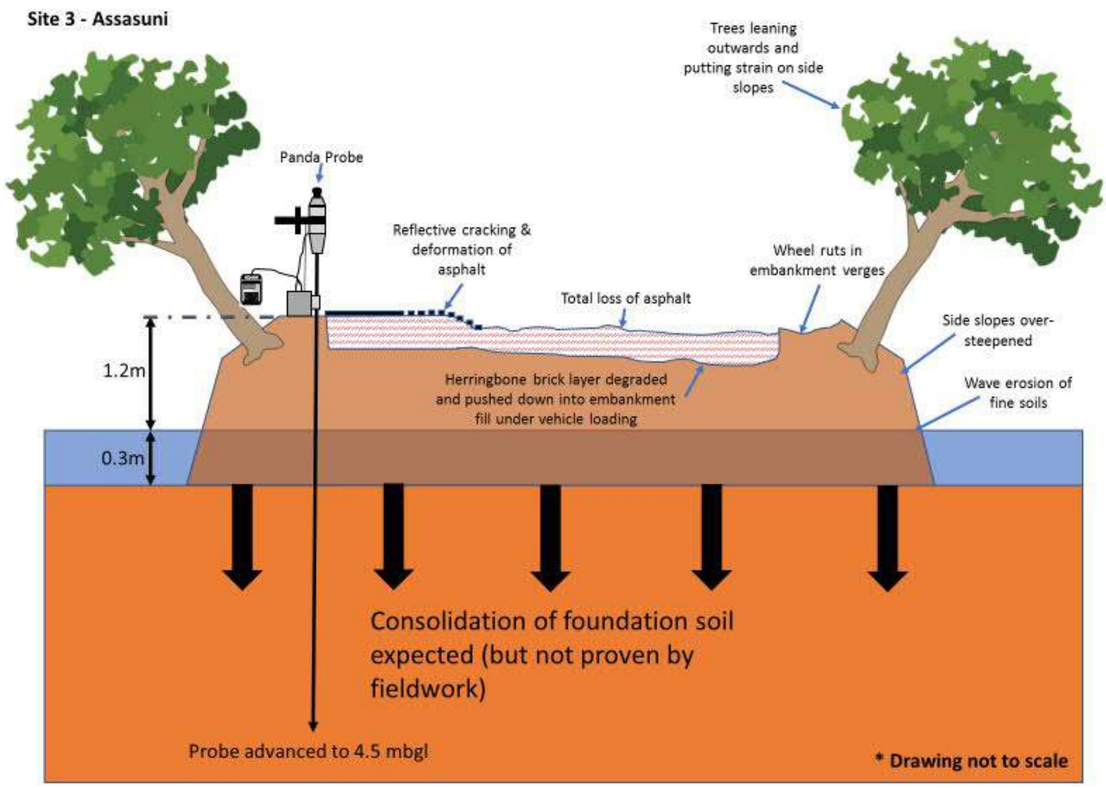
Also, as the water level changes considerably at this location due to flooding and adjacent land use, inundation of the embankment fill will be significant and the Fat clay fill present will swell in response, and shrink as the material dries. This effect also contributes to the oversteep slopes and the poor edge support that results from this.

The key contributory factors are illustrated below: -

Table 12.2: Summary of key contributory factors (Site 3)

Features observed	Key contributory factors
poor surface conditions	vehicle overloading, embankment construction materials and possibly methods, total settlement resulting from foundation soil consolidation and shrink/swell of the embankment fill
over-steep slopes / erosion.	Flooding and adjacent land use, together with shrink/swell of the embankment fill

Figure 12.1: Site 3 Assasuni Observational Ground Model



Site 10 Dumuria A Observational Ground Model

Site 10 – Dumuria A was selected during the Field Situation Analysis to investigate the causes of longitudinal cracking.

Key features	Site photograph
Longitudinal cracking towards the edge of the road pavement at Site 10	

The main body of the embankment is composed of a Lean Clay and a Lean Clay with Sand. This placed embankment fill material generally has a cone resistance of 2.5 to 9.5 MPa. Resistance within the embankment fill drops from the more competent near surface layers and then increases towards the base of the embankment fill and into the foundation soil. This illustrates probable poor compaction of the fill material, leading to low strength and support characteristics.

The foundation soil has been subject to loading from the embankment above and an increase in resistance between 6 and 10 MPa from 2 mbgl is due to normal consolidation and the increased overburden resulting from the embankment over time (although the proportion of resistance that can be attributed to the embankment cannot be determined). The ~2m of fill (approx. 36kN/m^2) is estimated to have resulted in ~150mm of vertical deformation (see Section 6.3 Analytical Ground Model) of the foundation soil, with the inherent impact on surface conditions, and could result in the cracking observed.

The longitudinal cracking is also likely to be as a result of embankment spreading, overloading the palisade walls. The lateral load on the piled wall is not only applied through the embankment layers, but also the soft foundation soil consolidates and exerts a lateral load on to the supports, that only have very limited embedment, and consequently rotate outwards.

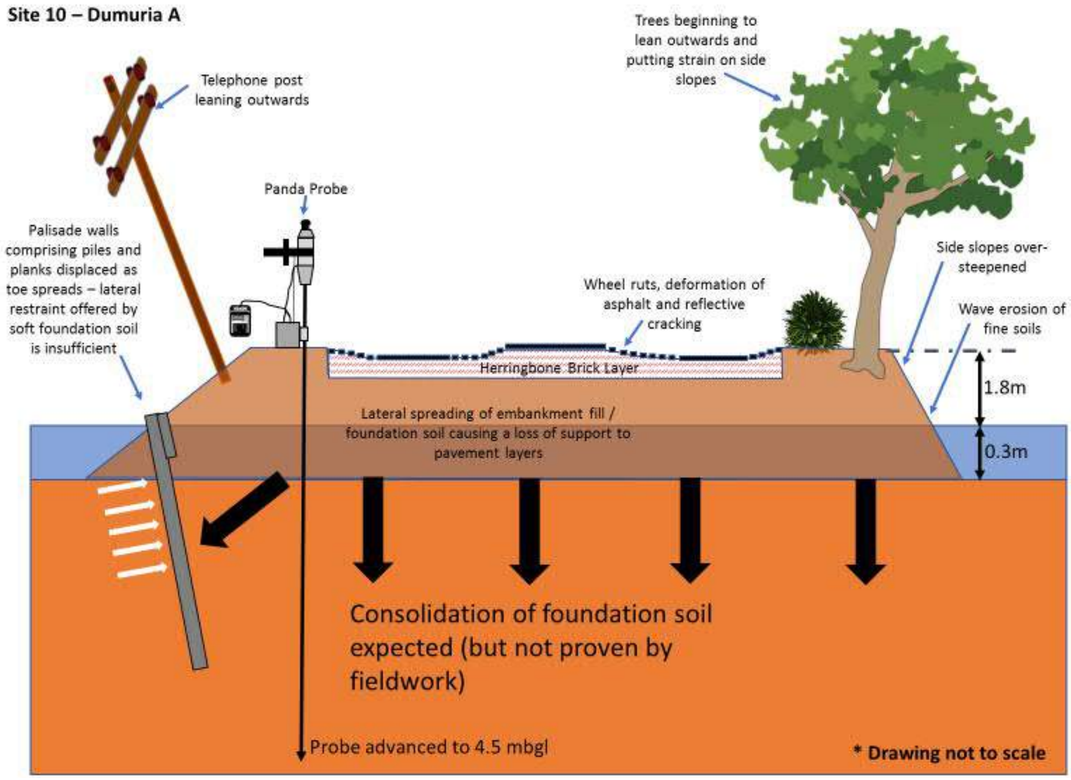
The road use also contributes to the longitudinal cracking observed, with wheel track ruts evident. These are likely to be the result of inadequate pavement strength and embankment fill construction.

The key contributory factors are illustrated below: -

Table 12.3: Summary of key contributory factors (Site 10)

Features observed	Key contributory factors
Longitudinal cracking	vehicle overloading, embankment construction materials and possibly methods, total settlement resulting from foundation soil consolidation, palisade walls offering little lateral resistance to embankment fill and foundation soil spreading.
Deformed palisade walls	Soft foundation soil has little resistance to the loads applied to low embedment walls.
over-steep slopes / erosion.	Flooding and adjacent land use, together with shrink/swell of the embankment fill

Figure 12.2: Site 10 Dumuria A Observational Ground Model



Site 11 Dumuria B Observational Ground Model

Site 11 – Dumuria B was selected for further investigation during the Field Situation Analysis due to the observed spreading behaviour and poor road surface conditions.

Key features	Site photograph
Lateral spreading of the embankments at Site 11	

The main body of the embankment is composed of a Fat Clay and a Fat Clay with Sand. This placed embankment fill material generally has a varying cone resistance of less 2 MPa to approximately 10 MPa. There is a distinct reduction in strength (Test 2 and 3) below the pavement layers, before this recovers from 1mbgl. The Fat clay will experience volume change during wetting and drying, due to the known changes in water levels that this site experiences during flooding.

The foundation soil consists of a Fat Clay and has a very consistent strength with depth, demonstrating a cone resistance of around 10 MPa from 1.5 to 2 mbgl to depth. Consolidation of the foundation soil is likely to have occurred due to the 1.6m of fill applied, and 120-130mm of vertical deformation is likely to have resulted from the change in volume (see Section 6.3 Analytical Ground Model).

The poor road surface conditions are predominantly due to pronounced lateral spreading of the embankment fill and consolidation of the underlying foundation soil. It is possible that bearing capacity of the soil was exceeded at construction stage and continuing deformation has resulted, although due to the low height, this is unlikely.

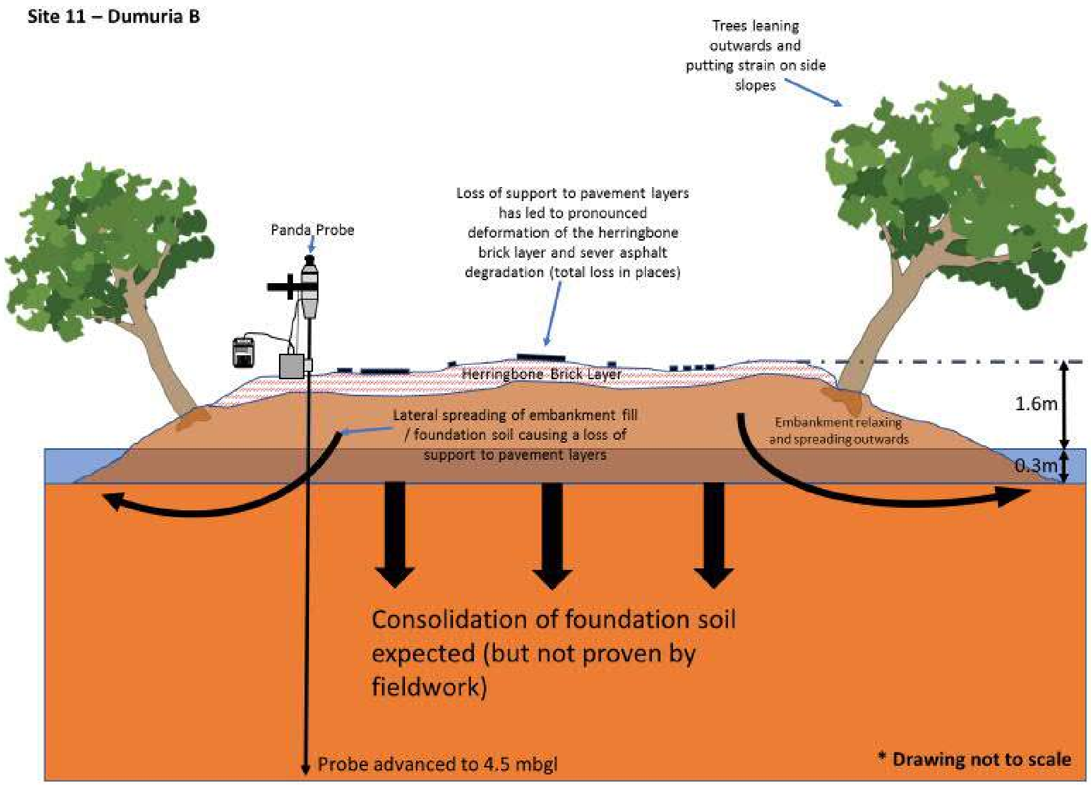
The lateral spreading of the embankment material has resulted in a loss of material underlying and supporting the pavement layers. Significant movement and deformation has occurred to the herringbone brick layer, leading to the degradation of the bricks. The deformation and degradation of the brick layer has resulted in pronounced reflective cracking in the asphalt and consequently, significant asphalt loss.

The key contributory factors are illustrated below: -

Table 12.4: Summary of key contributory factors (Site 11)

Features observed	Key contributory factors
Spreading of embankment fill	vehicle overloading, embankment construction materials and possibly methods, total settlement resulting from foundation soil consolidation, shrink/swell during inundation

Figure 12.3: Site 11 Dumuria B Observational Ground Model



Site 12 Rupsa Observational Ground Model

Site 12 – Rupsa was selected during the Field Situation Analysis to investigate the condition of the palisade wall, bridge approaches and loss of edge support to the highway.

The embankment fill comprises Lean Clay and a Fat Clay. This placed embankment fill material generally has a varying cone resistance (Test 2) reducing below the pavement layers from >20 MPa down to 2 MPa. This may reflect the varying repairs to the embankment where depressions have been filled with a mixture of crushed brick and soil.

The natural ground underlying the embankment consists of Lean Clay and Fat Clay and is anticipated to have experienced consolidation over time (both normal consolidation and consolidation due to embankment loading), demonstrating a cone resistance of around 5 to 10 MPa from 1 to 2 mbgl to depth. Theoretical consolidation of the foundation soil due to the 1.5m of fill applied, is anticipated to result in ~100mm of vertical deformation (see Section 6.3 Analytical Ground Model).

Longitudinal cracking was evident at Site 12 and has resulted from the development with time of wheel track ruts. These are largely due to inadequate pavement design and construction, overloaded vehicles etc. It is highly likely that the herringbone brick subbase is pressing down into the underlying embankment fill subgrade, leading to the creation of the wheel track ruts and resultant longitudinal reflective cracking.

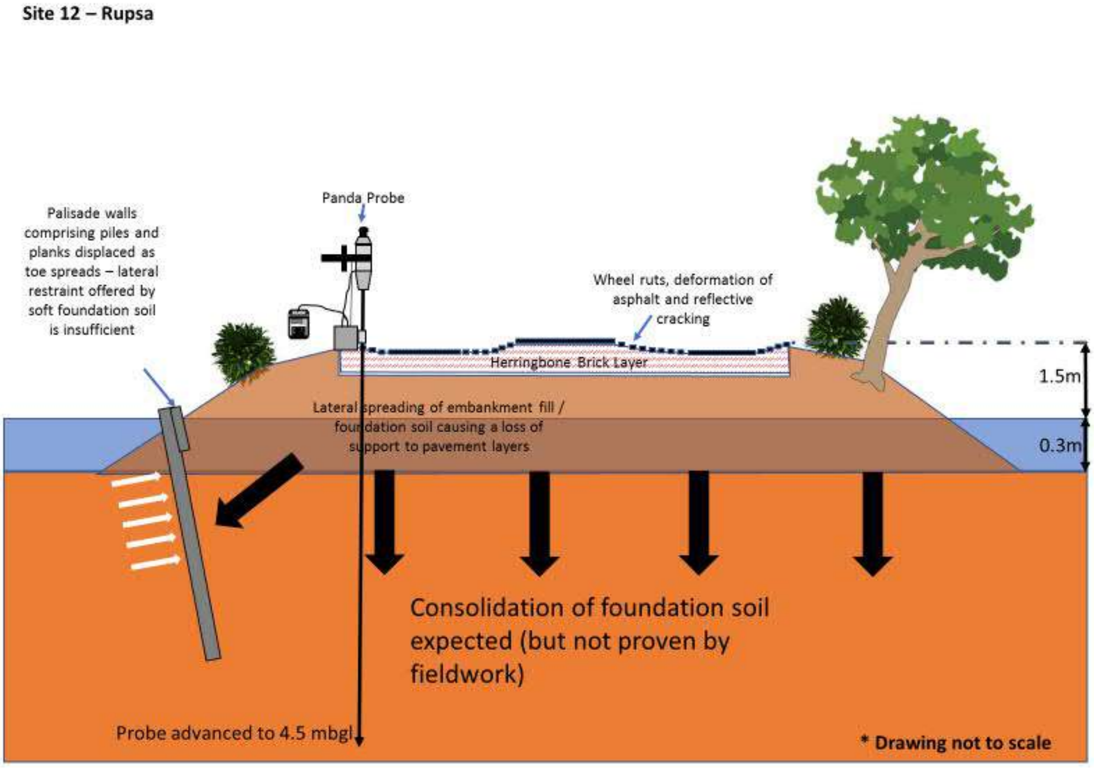
Lateral spreading was observed on at least one side of the embankment. This lateral spreading of the embankment material results in a loss of support to the pavement layers. The palisade wall at the toe of the embankment offers little resistance to the lateral forces applied by the embankment and consolidation of the foundation soil as embedment is very limited (3m).

The key contributory factors are illustrated below: -

Table 12.5: Summary of key contributory factors (Site 12)

Features observed	Key contributory factors
Spreading of embankment fill	vehicle overloading, embankment construction materials and possibly methods, total settlement resulting from foundation soil consolidation, palisade walls offering little lateral resistance to embankment fill and foundation soil spreading
Deformed palisade walls	Soft foundation soil has little resistance to the loads applied to low embedment walls.

Figure 12.4: Site 12 – Rupsa Observational Ground Model



Site 15 Terokhada A Observational Ground Model

Site 15 – Terokhada A was selected during the Field Situation analysis to investigate the poor road surface conditions, deformation of retaining wall and steep side-slopes.

The main body of the embankment is composed of a Lean Clay. This placed embankment fill material generally has a cone resistance of 2.5 to 15 MPa, the higher values reflecting the influence of broken brick aggregate near the surface.

The natural ground tested underlying the embankment consists of a Lean Clay and is likely to have experienced consolidation as a result of the loading of embankment fill, approximately 40kN/m^2. The cone resistance starts to steadily rise from 2-3MPa at 1.5-2mbgl to around 10 MPa at 3 to 4mbgl. Theoretical vertical deformation of the foundation soil is estimated at between 180mm as a result of this loading. The bearing capacity of the foundation soil may have been exceeded during construction (due to height of fill), and this may be the cause of the significant deformation noted on one side of the road. Even if the bearing capacity was not exceeded, the closer the loading to the ultimate bearing capacity, the greater the degree of deformation experienced (see 6.3 Analytical Ground Model).

Wheel track ruts were evident, and these are principally inadequate pavement design and construction for the vehicular usage.

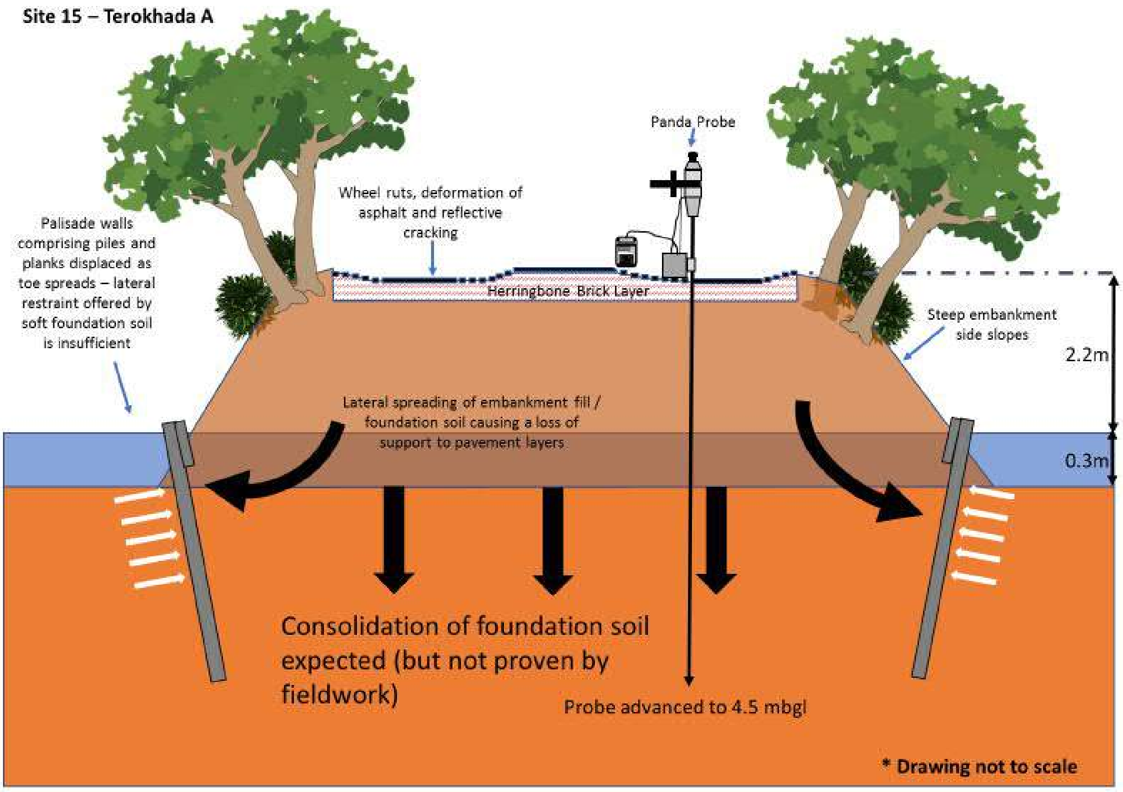
As noted, Site 15 has higher embankment than the others observed, with steep side slopes exacerbating slope instability, particularly problematic during periods of inundation when floodwaters are high. Lean Clay may erode more readily due to higher silt content and cannot accommodate changes in moisture content. Slope support in the form of palisade wall was present but this toe reinforcement was displaced and providing little resistance to slope movement.

The key contributory factors are illustrated in Table 12.6 and Figure 12.5 below: -

Table 12.6: Summary of key contributory factors (Site 15)

Features observed	**Key contributory factors**
poor surface conditions	vehicle overloading, embankment construction materials and possibly methods, total settlement resulting from foundation soil consolidation – possible bearing capacity issues after construction has weakened foundation soils
over-steep slopes / erosion.	Flooding and adjacent land use with already steep / high slopes. Lean materials erode and properties change more readily than Fat clay during wetting/drying
Deformed palisade walls	Soft foundation soil has little resistance to the loads applied to low embedment walls.

Figure 12.5: Site 15 – Terokhada A Observational Ground Model



Site 13 Terokhada B Observational Ground Model

Site 13 – Terokhada B was selected during the Field Situation Analysis to investigate the loss of support / slope instability at highway edge.

The road embankment is constructed from Lean Clay and Fat Clay. This placed embankment fill material generally has a cone resistance of 2.5 to 9.5 MPa, the higher values reflecting the influence of broken brick aggregate near the surface. Test 3 revealed much softer embankment fill however with cone resistance ranging between only 1 and 1.6 MPa.

The natural ground tested underlying the embankment consists of a Lean Clay and Fat Clay with a cone resistance that starts to steadily rise from 2 to 2.5 mbgl to around 10 MPa. It is unlikely that the foundation soil is an influencing factor with the slope instability observed.

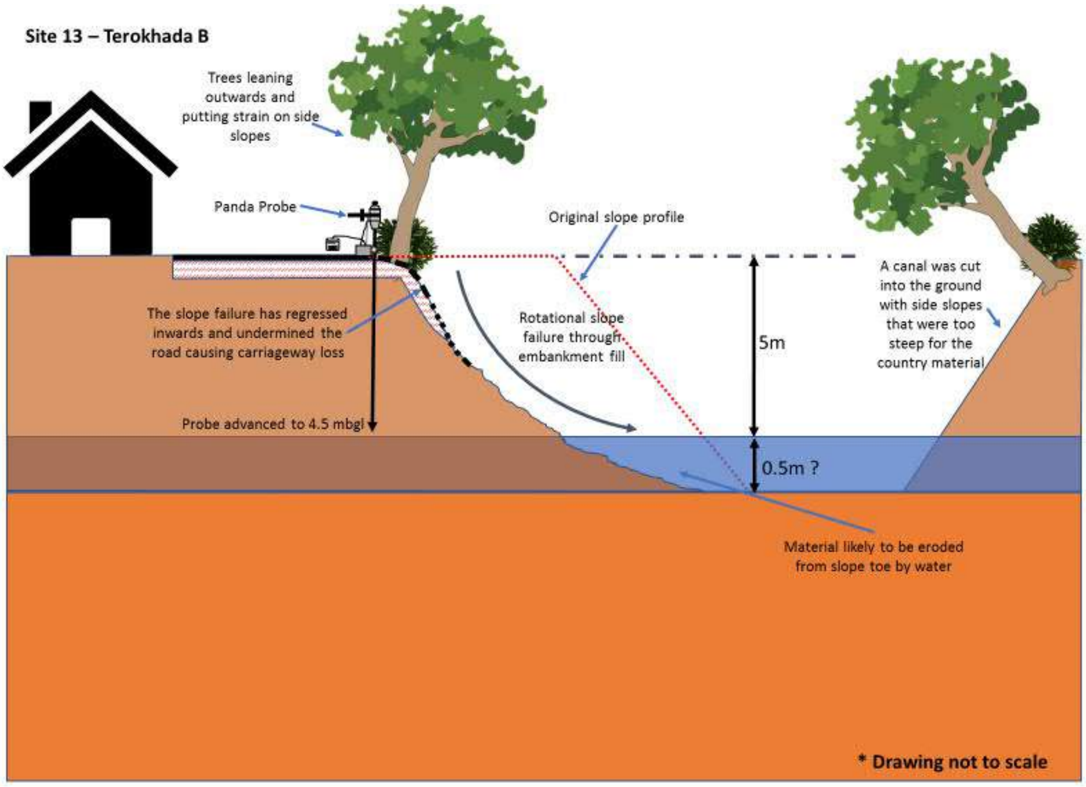
Two slope failures have occurred close to each other in the canal slope adjacent to the carriageway. The cause of the slope failures is due to the geometry of the slopes (over-steep), the materials forming the slope and the water in the canal. Any fine soil material (clays and silts) cut at the slope angle present at the site would be unstable in the long term. Changes (increase) in water levels in the canal influence pore water pressure in the slope, leading to reduced strength, which is particularly problematic as water levels recede after inundation (rapid drawdown). Slope failures (within the embankment fill) would often result in steep slopes exposed to fluctuating water levels.

The key contributory factors are illustrated below: -

Table 12.7: Summary of key contributory factors (Site 13)

Features observed	Key contributory factors
Slope instability through embankment	The presence of the canal and geometry is the key factor. Fluctuating water levels and adjacent land use with steep / high slopes.

Figure 12.6: Site 13 – Terokhada B Observational Ground Model



Site 21 Mongla A & B Observational Ground Model

Site 21 – Mongla A & B was selected during the Field Situation analysis to be investigated for bridge approach, differential settlement and retaining wall supports.

Key features	Site photograph
Differential settlement and erosion adjacent to structure at Mongla B - Site 21.	

The road embankment is constructed predominantly from Lean Clay and Fat Clay with Sand. At Mongla A - the placed embankment fill material generally has a cone resistance of 1.5 to 4 MPa. At Mongla B - the placed embankment fill material generally has a cone resistance of 20 MPa near the surface, dropping to around 4 MPa, the higher values reflecting the influence of broken brick aggregate near the surface.

The natural ground underlying the embankment consists of a Lean Clay and Fat Clay with Sand and demonstrates a cone resistance that starts to steadily rise from 4 MPa at 1 – 2 mbgl to > 10 MPa at 4.5 mbgl.

The level of the approach embankment has dropped relative to the abutment and concrete bridge deck leaving a pronounced step. This is the result of consolidation of the underlying foundation soil and embankment materials. Differential movement in the order of 200-250mm would be expected. The step has likely been addressed on multiple occasions and further loss of level through erosion and use has occurred. Bearing capacity of the foundation soil could have been an issue during construction, due to the higher level of embankment, although this mechanism cannot be confirmed at this location.

Damage to the bridge structure has been caused by river scour severely undermining the wing-walls, leading to an ensuing loss of fill from behind the wall – this has led to a further drop in the level of the approach embankment fill, located behind the wing walls, relative to the bridge deck.

It is clear, that the embankment slopes are steeper than the internal angle of friction of the fill material can accommodate. Typically, slopes in excess of 1:2(v:h) would be marginally stable when constructed from Lean or Fat Clay, and with the addition of water, instability would be anticipated. The slope angles presented in LGED (1999) Road Pavement Design Manual provide indicative safe slope angles for different earthworks materials, and whether dry or wet.

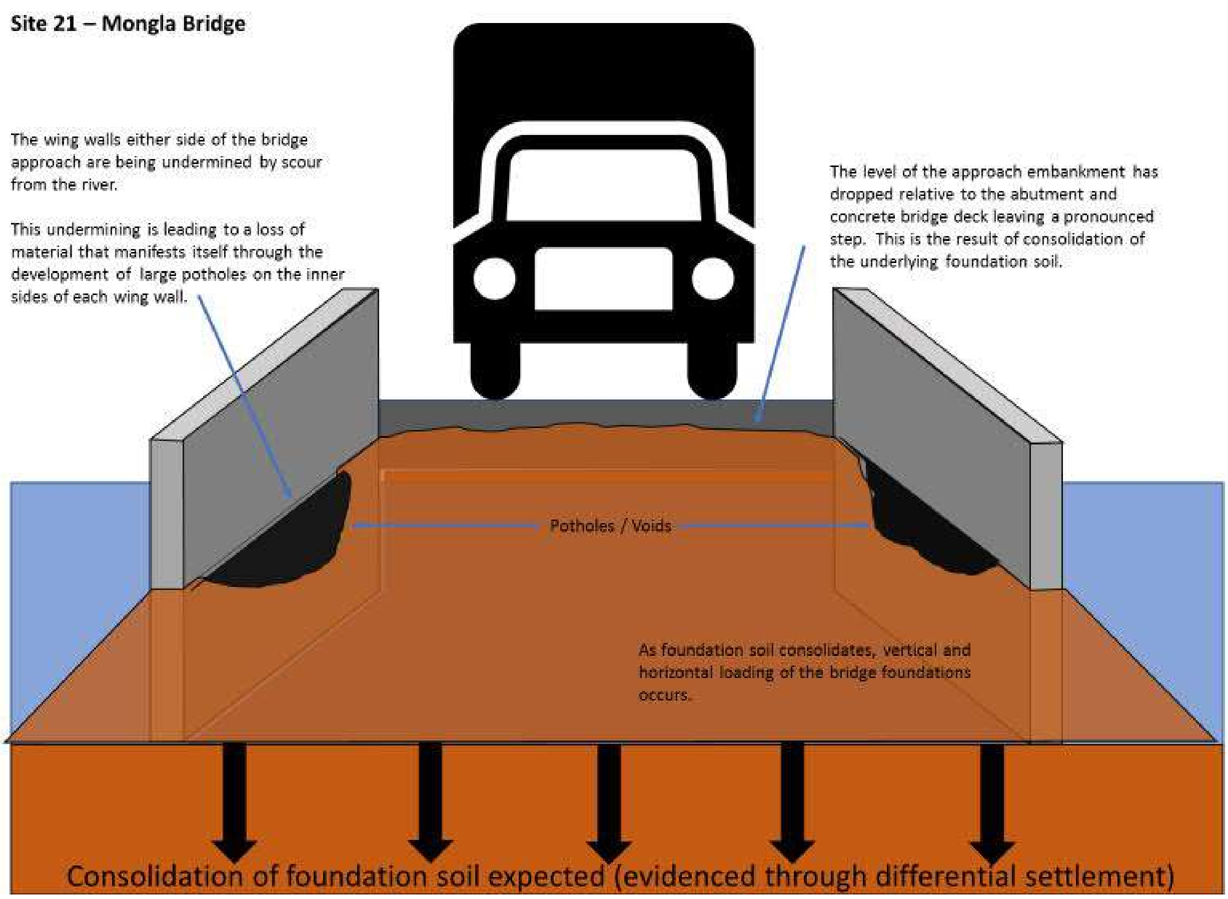
The reinforcement that has been provided in places at the slope toe is insufficient and has been displaced. The loads translated onto the walls cannot be sustained by the low foundation soil strength (at shallow depth) and limited depth of embedment of the wall.

The key contributory factors are illustrated in Table 12.8 and Figure 12.7 below: -

Table 12.8: Summary of key contributory factors (Site 21)

Features observed	Key contributory factors
Step between bridge and approach embankment	Differential settlement between the stiff bridge foundation support and the foundation soil below the approach embankment. Erosion below the wing walls and construction materials / methods.
Deformed palisade walls	Soft foundation soil has little resistance to the loads applied to low embedment walls.

Figure 12.7: Site 21 - Mongla Observational Ground Model



Appendix G: Compendium of photographs

Site 3: Assasuni	
 Photo: Site 3, Test Pit 1	 Photo: Site 3, Test Pit 2
 Photo: Site 3, Test Pit 3	 Photo: LGED Upazila Engr. assisting in sample collection

Site 10: Dumuria A

Photo: Site 10, Test Pit 1

Photo: Site 10, Test Pit 2

Photo: Site 10, Test Pit 3

Site 11: Dumuria B

Photo: Site 11, Test Pit 1

Photo: Site 11, Test Pit 2

Site 12: Rupsa

Photo: Site 12, Test Pit 1

Photo: Site 12, Test Pit 2

Photo: Site 12, Sample collection

Site 15: Terokhada A

Photo: Site 15, Structure inspection

Photo: Site 15, Test Pit 2

Site 13: Terokhada B

Photo: Site 13, Test Pit 1

Photo: Site 13, Test Pit 2

<table>
<tr><td colspan="2">Site 21: Mongla</td></tr>
<tr><td>
Photo: Site 21, Structure Inspection</td><td>
Photo: Site 21, Test Pit 1</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td>
Photo: Site 21, Test Pit 3</td><td></td></tr>
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Appendix H: Example Geotechnical Risk Register

Table 12.13 is an example Geotechnical Risk Register. The risks associated with other aspects of a scheme, such as procedures, highway design and contractual and strategic issues are not dealt with here and the scheme risk register should be prepared for information on these elements.

The Geotechnical Risk Register should be considered as a live document and updated throughout the course of the scheme. It is incumbent on all parties involved in the scheme to advise the other members when the risks change.

Various threats are identified and the potential consequences of these occurring are described. The risk assessment is qualitative and the various threat are assessed using the following criteria:

- Cost;
- Programme;
- Health and Safety; and,
- Environment.

The risk is derived by considering the impact and likelihood for each threat and opportunity. Both the impact and likelihood have been assessed using a scale of 1 to 5, corresponding to “very low” to “very high” for impact and “negligible / improbable” to “very likely / almost certain” for likelihood. These ratings are summarised in Table 12.9 and Table 12.10.

Table 12.9: Hazard Impact Table

		Impact	Cost	Programme	Health and Safety	Environment
1	Very Low	Negligible	Negligible	Negligible effect on programme	Negligible	Negligible
2	Low	Significant	1% Budget	5% effect on programme	Minor injury	Minor environmental incident
3	Medium	Serious	10% Budget	12% effect on programme	Major injury	Environmental incident requiring management input
4	High	Threat to future work and Client relations	20% Budget	25% effect on programme	Fatality	Environmental incident leading to prosecution or protestor action
5	Very High	Threat to business survival and credibility	50% Budget	50% effect on programme	Multiple fatalities	Major environmental incident with irreversible effects and threat to public health or protected natural resource

Table 12.10: Hazard Likelihood Index

	Likelihood	Probability
1	Negligible / Improbable	< 1%
2	Unlikely / Remote	> 1%
3	Likely / Possible	> 10%
4	Probable	> 50%
5	Very Likely / Almost Certain	> 90%

The risk score is calculated by multiplying the impact score by the likelihood score, giving the scores shown in Table 12.11.

Table 12.11: Risk Level Matrix

		Impact				
		1	2	3	4	5
Likelihood	1	N	N	N	N	A
	2	N	N	A	A	H
	3	N	A	A	H	S
	4	N	A	H	S	S
	5	A	H	S	S	S

Table 12.12: Designers Actions

Risk Product (I x L)	Risk Level	Description	Action by Designer
1 to 4	N	Negligible	None
5 to 9	A	Acceptable	Check that risks cannot be further reduced by simple design changes
10 to 12	H	High	Amend design to reduce risk, or seek alternative Option. Only accept Option if justifiable on other grounds.
15 to 25	S	Severe	

Table 12.13: Geotechnical Risk Register

Hazard	Consequence	Impact	Likelihood	Current Risk	Risk Type	Potential Control Measures	Impact	Likelihood	Residual risk
Alluvium Deposits	Variable lithologies of poor engineering quality. Soft, compressible soils (in places), variable thickness.	4	3	H	C,T,H&S,E.	Detailed ground investigation and associated geotechnical laboratory testing to allow a detailed ground model and set of parameters be determined for use within design.	3	2	A
Embankment Fill materials	Variable lithologies and engineering properties.	3	2	A	C,T,H&S,E.	Detailed ground investigation to identify any areas of the Embankment Fill (Made Ground) and associated properties.	2	2	N
Inadequate ground investigation.	Unforeseen ground conditions, inappropriate design parameters	4	4	S	C,T,H&S,E.	Conduct a ground investigation based on a detailed desk study	2	1	N
Changes in the groundwater and flooding conditions.	Detrimental effect on earthworks stability y.	3	1	N	C,T.	Monitoring of groundwater levels during and after ground investigation.	3	1	N
Lack of suitable material for earthworks on site.	Excessive import of acceptable materials and / or disposal of unacceptable onsite materials.	3	4	H	C,T,H&S,E.	Schedule appropriate earthworks acceptability testing as part of the ground investigation. Programme earthworks into a season with favourable weather. Consider improvement of onsite soils. Monitoring and testing of soils throughout earthworks.	2	2	N

Appendix I: International Ground Improvement Techniques

As stated in the CIRIA C573: A Guide to Ground Treatment, virtually all engineering construction involves the ground. When constructing in poor ground conditions, there are five available options:

- To bypass the poor ground, by moving to a new site, or using deep foundations to stronger ground.
- To remove the poor ground, replacing it with better material.
- To design the structure to allow for the behaviour of the poor ground under load.
- To treat the poor ground to improve its properties (i.e. ground improvement).
- To abandon the project (the promoter's decision).

The fourth option, of ground treatment, gives considerable scope to engineers for finding a viable solution to the problems of poor ground. A wide range of treatments are available, techniques can be selected and combined to cope with different aspects of the poor ground, and there is increasing confidence both in what can be achieved by well executed treatment and in its proper integration into the overall scheme for the construction. All these points are evidence of how valuable this option is.

The objective of treatment is of course improvement. When ground treatment is being considered as an option, it is important that all who will be involved in it should recognise not only what can reasonably be achieved by a particular technique, but also the extent of their responsibility if it is chosen.

The term "ground improvement" is open to different interpretations. First, it is an intention or objective, not the process of achieving it, although the term is often used in that sense. Second, improvement is a relative condition as to which aspect and to what degree there is improvement.

Ground treatment techniques have been in use around the world for many centuries but have developed greatly over the past 40 years.

CIRIA C573: A Guide to Ground Treatment states that: 'In the United Kingdom, some 75 per cent of the ground improvement contracts using the techniques of vibro-replacement and dynamic compaction are for man-made ground. These two techniques, including their application to loose or soft natural soils, are probably the commonest type of ground treatment used in the UK. For overseas work, the proportions of specialist ground treatments are reversed, i.e. 30 percent are for man-made ground and 70 per cent for natural ground'.

The techniques of ground improvement used around the world have been grouped into 8 No. broad categories:

- Improvement by vibration:
    - Vibro-compaction;
    - Vibro-replacement;
    - Dynamic compaction;
    - Vibratory probing;
    - Compaction piles;
    - Blasting;
- Improvement by adding load (or increasing the effective stresses):
    - Pre-compression;
    - Vertical drains;
    - Inundation;
    - Vacuum preloading;

  - Dewatering fine soils;
  - Pressure berms.
- Improvement by structural reinforcement:
  - Reinforced soil;
  - Soil nailing;
  - Root and micro-piles;
  - Slope dowels;
  - Embankment piles;
- Improvement by structural fill:
  - Remove and replacement;
  - Displacement;
  - Reduced load;
- Improvement by admixtures:
  - Lime / cement columns;
  - Mix-in-place by single augur;
  - Lime stabilisation of slopes;
  - Stabilisations of subgrades;
- Improvement by grouting:
  - Permeation;
  - Hydro-fracture;
  - Jet grouting;
  - Compaction grouting;
  - Cavity filling.
- Improvement by thermal stabilisation.
  - Freezing.
  - Heating.
- Improvement by vegetation.
  - Vegetation planting.

NB: Although the above headings for the groups of methods reflect what is being undertaken to the ground to improve it, they do not characterise the way the ground is to be improved, nor do they show the purpose of the improvement. Many of the techniques can be used for different purposes and by enhancing one aspect of soil behaviour other aspects are also improved.

In accordance with the breakdown of each category in CIRIA C573, we can define simply, the general method by which the ground is improved for each of the eight abovementioned ground improvement categories:

Improvement by vibration:

Vibration can be used to compact soils and fills. The densification is achieved by a combination of ground displacement and vibration, in most cases with the addition of new material into the ground.

Improvement by adding load (or increasing the effective stresses).

Increasing the load on the ground causes it to compress. How much compression and how long it takes to happen depends on the arrangement of the ground particles, on the degree of saturation, and on how freely the soil can drain. For loose and particularly unsaturated fills, adding load induces rapid settlement; soft, saturated clays, on the other hand, take months or years to consolidate under an added load while pore pressures dissipate and the effective stress in the soil increases. The improvement techniques of adding load fall into two, not necessarily exclusive categories:

- Where the improvement largely comes about by the increase in total stress
- Where the improvement depends upon the increase in effective stress and the technique encourages or accelerates that.

Improvement by structural reinforcement.

Many ground improvement techniques could be considered as a form of reinforcement. Stone columns, for example, are introduced materials that stiffen the ground; some grouts strengthen the mass of soil into which they are injected. The distinction drawn for the classification used in this report of structural reinforcement is that prefabricated tensile or shear elements are installed in the ground with the purpose of forming a composite material.

Improvement by structural fill.

The principle of ground improvement by structural fill is to replace a weak soil with a better one. Another, more recent option is to use lightweight materials instead of heavier earth fills above weak ground. These options include:

- Displacement.
- Reducing load.
- Removal and replacement.

Improvement by admixtures.

The use of admixtures, such as lime, cement, oils and bitumens, and even sulphur, is one of the oldest and most widespread methods of improving a soil. Usually the purpose is to strengthen a locally available earth fill to construct a low-cost road base, e.g. cement stabilised soil or soil-cement, or to mix lime into highly plastic clays. Plant was developed either to mix the stabiliser in place, i.e. to strengthen foundation soils or layers of the fill, or for central mixing to which the soil is transported.

Improvement by grouting.

A general definition of grouting for ground improvement is: “the controlled injection of material, usually in a fluid phase, into soil or rock in order to improve the physical characteristics of the ground”. Such a definition does not cover all types or purposes of grouting in the ground, e.g. grouting to raise ground slabs or road pavements, but it does cover grouting to fill voids in the ground, whether natural (such as in karstic limestone) or resulting from human activity.

Improvement by thermal stabilisation.

Even in the temperate UK, everyone is familiar with the way that surface soils are hardened, albeit temporarily, by frost and hot, dry weather. The removal of heat from the soil turning its pore water into ice is a very powerful technique rendering the ground impermeable and, for unconsolidated materials, making them stronger. Applying heat to clays to drive out free pore water and, at higher temperatures, the water adsorbed on particle surfaces, creates a very hard, durable material - in effect, the same methods as when making brick or mud (adobe) building blocks. Ground freezing is a long established and particularly effective method of ground stabilisation for temporary works. Ground heating is rare, but when it has been used its purpose was longer-term improvement.

Improvement by vegetation.

Vegetation as ground improvement is the biological reinforcement of ground by plant roots to retain earth masses and prevent soil loss.

Appendix J: Ground Improvement Techniques for Rural Roads in Bangladesh

The following Table presents the list of all ground improvement techniques available (as per Appendix I), those that are used in Bangladesh and the techniques considered applicable for use on rural road infrastructure.

Table 12.14: Ground improvement techniques for rural roads in Bangladesh

Ground Improvement Category	Technique	In common use in Bangladesh	Applicable for use on Rural Roads
Improvement by vibration:	Vibro-compaction;		
	Vibro-replacement;		
	Dynamic compaction;	Y	unlikely
	Vibratory probing;		
	Compaction piles;	Y	Y
	Blasting;		
Improvement by adding load (or increasing the effective stresses):	Pre-compression;	Y	Y
	Vertical drains;	Y	Y
	Inundation;		
	Vacuum preloading;		
	Dewatering fine soils;		
	Pressure berms.		
Improvement by structural reinforcement:	Reinforced soil;		Y
	Soil nailing;		
	Root and micro-piles;		
	Slope dowels;		
	Embankment piles;		
Improvement by structural fill:	Remove and replacement;	Y	Y
	Displacement;	Y	Y
	Reduced load;		
Improvement by admixtures:	Lime / cement columns;		Y
	Mix-in-place by single augur;		Y
	Lime stabilisation of slopes;		
	Stabilisations of subgrades;		Y
Improvement by grouting:	Permeation;		
	Hydro-fracture;		
	Jet grouting;	Y	unlikely
	Compaction grouting;		
	Cavity filling.		
Improvement by thermal stabilisation.	Freezing.		
	Heating.		
Improvement by vegetation.	Vegetation planting.		

Appendix K:Ground Investigation Techniques for Rural Roads in Bangladesh

The following Tables are presented as a guide to the type and quantity of field and laboratory testing that may be carried out for typical rural road applications.

*Note, if available, Cone Penetrometer Testing (CPT) e.g. piezocone / Dutch cone may supplement (or in part replace) the boreholes and in situ testing requirements.

Table 12.15: Typical investigation and sampling regime for minor bridge

Investigation / sampling	Description
Boreholes*	2 -4 Cable percussion / rotary boreholes – 1-2 at each abutment. In typical conditions in Khulna region, 20-25m depth.
In situ testing*	Standard penetration testing – 1 to 1.5m intervals within BHs. With correlations, can be used to develop undrained strength and stiffness profiles for use in stability and settlement analysis.
Monitoring	Hydrology likely to be clear from observation. If not, install 1 or 2 standpipes (as appropriate) and monitor.
Soil sampling – Shelby tubes / U100	Regular intervals – for purposes of strength and consolidation testing
Soil sampling – tub / bag samples	Regular intervals – for purposes of soil classification
Laboratory testing – strength	Unconsolidated Undrained Triaxial to determine undrained shear strength. Combine with results from standard penetration testing to provide strength properties of ground. Possible, but unlikely to undertake Consolidated Undrained Triaxial testing for small scale structures.
Laboratory testing – consolidation	Oedometer testing to determine anticipated quantity and rate of settlement.
Laboratory testing – classification	Atterberg Limits, particle size distribution, organic content, moisture content,
Laboratory testing – chemical	Chemical analysis e.g. pH, sulphates, chlorides testing to determine aggressivity of ground in order to specify corrosion resistance of concrete elements.

Table 12.16: Typical investigation and sampling regime for minor culvert

Investigation / sampling	Description
Boreholes*	1-2 Cable percussion / rotary boreholes within footprint of proposed culvert. In typical conditions in Khulna region, 20-25m depth.
In situ testing*	Standard penetration testing – 1 to 1.5m intervals within BHs. With correlations, can be used to develop undrained strength and stiffness profiles for use in stability and settlement analysis.
Monitoring	Hydrology likely to be clear from observation. If not, install 1 or 2 standpipes (as appropriate) and monitor.
Soil sampling – Shelby tubes / U100	Regular intervals – for purposes of strength and consolidation testing
Soil sampling – tub / bag samples	Regular intervals – for purposes of soil classification
Laboratory testing – strength	Unconsolidated Undrained Triaxial to determine undrained shear strength. Combine with results from standard penetration testing to provide strength properties of ground.
Laboratory testing – consolidation	Oedometer testing to determine anticipated quantity and rate of settlement.
Laboratory testing – classification	Atterberg Limits, particle size distribution, organic content, moisture content,
Laboratory testing – chemical	Chemical analysis e.g. pH, sulphates, chlorides testing to determine aggressivity of ground in order to specify corrosion resistance of culvert structure.

Table 12.17: Typical investigation and sampling regime for approach embankments

Investigation / sampling	Description
Boreholes*	2 -4 Cable percussion / rotary boreholes. 1-2 for each approach embankment depending on scale. In typical conditions in Khulna region, 20-25m depth.
In situ testing*	Standard penetration testing – 1 to 1.5m intervals within BHs. With correlations, can be used to develop undrained strength and stiffness profiles for use in stability and settlement analysis.
Trial Pits	Specify in borrow areas to characterise the fill materials. Only necessary if embankment fill to be recovered from adjacent to proposed alignment
Soil sampling – Shelby tubes / U100	Regular intervals – for purposes of strength and consolidation testing
Soil sampling – tub / bag samples	Regular intervals – for purposes of soil classification
Laboratory testing – consolidation	Unconsolidated Undrained Triaxial to determine undrained shear strength. Combine with results from standard penetration testing to provide strength properties of ground.
Laboratory testing – strength	Oedometer testing to determine anticipated quantity and rate of settlement.
Laboratory testing – classification	Atterberg Limits, particle size distribution, organic content, moisture content,
Laboratory testing – chemical	If structures present only. Chemical analysis e.g. pH, sulphates, chlorides testing to determine aggressivity of ground in order to specify corrosion resistance of structures where present e.g. palisade toe walls, gabions.
Laboratory testing – other	Compaction type testing on material from borrow areas; including optimum moisture content, maximum dry density, laboratory CBR.

Table 12.18: Typical investigation and sampling regime for general embankments

Investigation / sampling	Description
Boreholes*	1 Cable percussion / rotary boreholes through embankments at 50-100m spacing along alignment. In typical conditions in Khulna region, ideally 20-25m depth, but could reduce depth or increase spacing if necessary.
In situ testing *	Standard penetration testing – 1 to 1.5m intervals within BHs. With correlations, can be used to develop undrained strength and stiffness profiles for use in stability and settlement analysis.
Trial Pits	Specify in borrow areas to characterise the fill materials. Only necessary if embankment fill to be recovered from adjacent to proposed alignment
Soil sampling – Shelby tubes / U100	Regular intervals – for purposes of strength and consolidation testing
Soil sampling – tub / bag samples	Regular intervals – for purposes of soil classification
Laboratory testing – consolidation	Unconsolidated Undrained Triaxial to determine undrained shear strength. Unlikely to undertake Consolidated Undrained Triaxial testing for small scale structures.
Laboratory testing – strength	Oedometer may be undertaken to determine anticipated quantity and rate of settlement.
Laboratory testing – classification	Atterberg Limits, particle size distribution, organic content, moisture content,
Laboratory testing – chemical	If structures present only. Chemical analysis e.g. pH, sulphates, chlorides testing to determine aggressivity of ground in order to specify corrosion resistance of structures where present e.g. palisade toe walls, gabions.
Laboratory testing – other	Compaction type testing on material from borrow areas; including optimum moisture content, maximum dry density, laboratory CBR.