Manual

Geotechnical Design Standard – Minimum Requirements

October 2024

Queensland Government

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Contents

1 Introduction

This document outlines the minimum geotechnical requirements, which shall be met in the design phase of all Department of Transport and Main Roads projects. The requirements stipulated here are the minimum geotechnical requirements and do not preclude the Designer from using other proven methods in addition to those identified within this document. Some construction requirements that may impact the designs are also included.

The scope briefing for all geotechnical works shall be acceptable to the department's Geotechnical Section before commencement of any geotechnical site investigation. Geotechnical site investigation shall be carried out in accordance with the department's guideline for *Geotechnical Investigation* and logging of encountered subsurface materials during geotechnical investigation shall be in accordance with the department's *Geotechnical Logging* guideline. Where there is a conflict between *Geotechnical Investigation* guideline and this *Geotechnical Design Standard* (GDS), the content of this GDS shall take precedence.

Wherever the term 'Administrator' is referred to in this document, it shall be replaced with:

- 'Project Manager' for Design Only Contracts.
- 'Independent Reviewer' or 'Independent Verifier' for Public Private Partnerships.
- 'Design Verification Manager' for Collaborative Project Agreements.

Wherever 'Transport and Main Roads Geotechnical Section' is referred, the contact person shall be Director (Geotechnical) or his / her nominee.

All direct communication between the Designer and Transport and Main Roads Geotechnical Section shall be in accordance with the communication plan for the Contract. Any direct communication about matters that may affect Scope, Cost, Time, Quality must also include the Administrator.

All geotechnical design reports, including drawings, shall be submitted to the department's Geotechnical Section in electronic format (and hard copy if requested) for review. The reports shall clearly state the assumptions, the justification of adopted geotechnical profiles, parameters and the methods used in the design and address all relevant issues or concerns for the design element in question. The reports shall also include geotechnical long and cross sections along with the site investigation location plan(s) drawn to the same horizontal scale for each design element.

The development of a geotechnical model, as discussed in this document shall generally follow the requirements of Clause 5.2 of AS 1726. However, for each geotechnical design element, the specific minimum requirements shall align with the relevant sections of this document.

When the reports are submitted in stages (for example, concept, business case, detailed design stages and so on), each report shall be a standalone report. At the end of the full review process, a final standalone geotechnical document, including geotechnical field and laboratory data, interpretative design report(s) as per above shall be submitted to the department's Geotechnical Section through the Administrator for their record(s).

The design calculations, including any input and output files used, shall be duly documented as the design work progresses. These documents shall be provided to the Administrator upon request. The Administrator will then forward these reports to the department's Geotechnical Section.

The design, construction, maintenance and monitoring of earthworks and associated protective treatments shall ensure that permissible movement or performance of the pavement meets the requirements set out in the departmental pavement designs specifications and that post-construction in-service movements and both subsurface and surface water flows at any time do not:

- impair or compromise pavement support
- impair or compromise support of structures, and/or
- cause pavements to fail to meet the department's pavement performance criteria, provided regular programmed maintenance is undertaken to ensure the durability of the assets.

Under special circumstances, the Contractor / Designer may seek exemption (or departure) from compliance with sections in this document. To obtain such an exemption, the Contractor / Designer shall undertake a geotechnical risk assessment that demonstrates to the department's Geotechnical Section why such an exemption is being sought and under what special circumstance(s).

In addition to the risk assessment, the Contractor / Designer must provide a written report which details how the proposed exemption (non-compliance) will not compromise the performance standards stipulated in this document, covering safety, durability, future performance, constructability and maintenance aspects.

The risk assessment and report must be submitted formally through the Administrator to the department's Geotechnical Section. Consent to proceed with any proposed departure will be solely at the discretion of the department's Geotechnical Section.

Should a departure be consented, the Administrator or departmental Delegate will accept or reject this exemption through written correspondence.

All geotechnical reports shall be certified by a Registered Professional Engineer of Queensland (RPEQ) Civil Engineer, who is competent in the field of geotechnical engineering.

The designs carried out using numerical models, such as (but not limited to) Finite Element Method (FEM), shall be checked and certified by suitably qualified geotechnical engineer(s) with:

- specialist knowledge in soil mechanics and the theory behind the numerical method adopted, and
- a minimum of five years of experience as a practitioner in numerical modelling.

The design calculations carried out using numerical modelling shall be submitted in summary form, including:

- input parameters, ground and constitutive models used, with justification
- assumed construction stages, and
- the adopted model (for example, FEM Model details and the boundary conditions) and the outputs for all construction stages critical to the design.

The outputs from all the numerical calculations / models must be validated or checked using simple hand calculations, another numerical method, or empirical methods.

For critical designs or in the case where the design outcomes are contested by the reviewer(s), complete electronic input and output files (including validated data) must be submitted, upon request, for verification.

The required design life for bridges and other structures foundations are given in the department's *Design Criteria for Bridges and other Structures.* For all other geotechnical design elements, such as embankments, cut slopes, retaining walls covered in this document for new infrastructure projects, the minimum design life shall be 100 years. Refer to Section 7 of this document for the required design life for remediation of existing slopes and embankments.

Where the Department of Transport and Main Roads' specifications, design standards, manuals, guides, or technical notes do not exist or are incomplete for a particular design, an appropriate reference document shall be used in the following descending order of precedence:

1. Australian Standards
2. British Standards
3. American and European Standards
4. other international standards, and
5. other relevant technical publications, standards, guidelines, technical notes, and practice notes issued by recognised industry organisations, as agreed with the department's Geotechnical Section.

The designer may choose to provide project-wide geotechnical parameters within the overarching geotechnical report. However, it is crucial to develop individual geotechnical models for each distinct geotechnical design element using the geotechnical data specific to that element. Additionally, it is important to graphically represent the relevant geotechnical properties (both soil and rock layers) at varying depths, along with the properties selected for design.

A geotechnical design element could be a zone of ground treatment, a retaining structure, a bridge foundation, etc. If the subsoil conditions exhibit significant variability, it becomes necessary to prepare multiple geotechnical models for individual segments (for example, every pile location in a bridge foundation) within a single geotechnical design element. The details shall be agreed upon with the department's Geotechnical Section.

2 Referenced documents

Reference	Title
AS 1170.4	*Structural design actions, Part 4: Earthquake actions in Australia*, Australian Standard
AS 1726	*Geotechnical Site Investigations*, Australian Standard
AS 2159	*Piling – Design and installation*, Australian Standard
AS 2870	*Residential slabs and footings – Construction,* Australian Standard
AS 4678	*Earth-retaining structures*, Australian Standard
AS 5100.2	*Bridge design – Design Loads*, Australian Standard
AS 5100.3	*Bridge design – Foundation and soil supporting structures*, Australian Standard

Reference	Title
Asaoka, A (1978)	*Observational procedure of settlement prediction, Journal of the Soils and Foundations Engineering*, Vol. 18(4), pp 87-101
BS 8006 – Part 2	*Code of practice for strengthened / reinforced soils*, British Standards Institution
BS 8081	*Code of practice for Ground Anchorages*, British Standards Institution
BS 5975	*Code of Practice for temporary works procedures and the permissible stress design of falsework*, British Standards Institution
CIRIA C760 (latest edition)	*Guidance on Embedded Retaining Wall Design*, Construction Industry Research and Information Association
-	*Design Criteria for Bridges and other Structures* Manual
-	*Geotechnical Investigation* Guideline
-	*Geotechnical Logging* Guideline
MRTS03	*Drainage, Retaining Structures and Protective Treatments*
MRTS04	*General Earthworks*
MRTS06	*Reinforced Soil Structures*
MRTS27	*Geotextiles (Separation and Filtration)*
MRTS40	*Concrete Pavement Base*
MRTS63	*Cast-In-Place Piles*
MRTS63A	*Piles for Ancillary Structures*
MRTS64	*Driven Tubular Steel Piles (with reinforced concrete pile shaft)*
MRTS65	*Precast Prestressed Concrete Piles*
MRTS66	*Driven Steel Piles*
MRTS68	*Dynamic Testing of Piles*
Poulos, H. G. (1971)	a) *The behaviour of laterally loaded piles: I. Single piles*. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM5. pp. 711-731. b) *The behaviour of laterally loaded piles: II. Single piles*. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM5. pp. 733-751.
TfNSW (2014)	Roads and Maritime Services, New South Wales. *Guide to slope risk analysis, Version 4.*
TfNSW (2018)	Roads and Maritime Services, New South Wales. *Technical Direction for Geotechnical Design for Remediation on Existing Slopes and Embankments*, GTD 2018/001 I RMS 18.748 – 22 February 2018.
-	*Road Planning and Design Manual – 2nd Edition*
Rowe R K & Armitage H. H (1987)	*A Design method for drilled piers in soft rock*. Canadian Geotech. J. Vol. 24, 126-142.
Turner, J.P. (2006)	*NCHRP Synthesis 360: Rock-Socketed Shafts for Highway Structure Foundations*, Transportation Research Board, National Research Council, Washington, D.C., pp. 43-46.

Note: Current codes of practice, manuals and specifications shall be adopted for all geotechnical designs and constructions works in the execution of the requirements stipulated in this document

3 Embankments

3.1 General requirements

Notwithstanding the requirements stipulated in the department's Technical Specification MRTS04 *General Earthworks*, the following also shall apply:

- Unreinforced embankment batter slopes shall not be steeper than:
  - 1 (vertical) to 2 (horizontal) for earth-fill, and
  - 1 (vertical) to 1.5 (horizontal) for rockfill.
- For embankments in earth-fill, the vertical height of any single continuous batter shall not exceed 10 m. A minimum 4 m wide bench shall be provided at the top of any 10 m high single continuous batter in an earth-fill embankment for erosion control and maintenance purposes. The bench and the batter must be adequately protected against erosion. A berm drain shall be provided at each bench as per MRTS04 *General Earthworks*.
- Designer may eliminate benches for rockfill embankments up to 20 m high.
- Spill-through embankments for bridge structures shall be designed as a standalone element complying with department's Technical Specification MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*
- Material requirements within the Structure Zone are provided in MRTS04 *General Earthworks*.
- Where additional construction requirements exist, a Supplementary Specification must be produced for the construction of the embankment.

3.2 Structure zone

The 'Structure Zone' is defined as a length not less than 25 m (except for Lower Speed Roads) within the approach to any structure (bridges, culverts, non-floating piled embankment, and so on). For Lower Speed Roads, length of the Structure Zone can be reduced to 10 m in consultation with the department's Geotechnical Section. This length shall either be measured from the inside edge of the relieving slab or the outside face of the headstock where relieving slab is not present.

The maximum permissible total in-service settlements (within the first 40 years in service) within the Structure Zone and away from the Structure Zone are given in Table 3.3.

Lower Speed Roads: Roads with post speed limit less than or equal to 70 km/h are defined as Lower Speed Roads for the purpose of implementation of this standard.

If a culvert is designed as a floating culvert, the Structure Zone can be eliminated. However, total in service settlement (including creep) of large culverts shall not exceed 50 mm to ensure the structural integrity of the culvert over its 100 year design life, thereby preventing potentially adverse impacts from differential settlements and other unknown effects.

Floating Culvert: A culvert that will settle together with its approach and supporting embankment with time is termed as floating culvert for the purpose of implementation of this standard.

Large Culvert: A large culvert in Geotechnical context meets at least one of the following criteria:

- width greater than 1.2 m for box or steel arch culverts
- height greater than 1.2 m for box or steel arch culverts, and
- diameter greater than 1.5 m for reinforced concrete pipe culverts.

Sizes mentioned here are the internal dimensions of culvert openings.

3.3 Performance standards

Embankments and their foundations must remain stable and free from movement along any slip surface throughout their design life. For embankments constructed over soft foundations, regular instrumentation monitoring during construction is necessary. This monitoring includes plotting settlements, lateral movements, and pore pressure development over time to provide early warnings of potential failure. These warnings allow for the implementation of safety measures to prevent failures and ensure compliance with minimum FOS during construction. The data obtained shall be submitted through the Administrator to the department's Geotechnical Section for review.

The term 'stable' embankments, as used in this document, refers to road embankments that have been designed and constructed in accordance with all performance and minimum requirements stipulated in this document..

Post-construction in service movements shall not impair or compromise pavement support and shall not exceed permissible pavement movement requirements as per departmental pavement design specifications.

The materials and construction methods used for embankments must ensure resistance to cracking caused by seasonal moisture changes and must not be prone to erosion or dispersion, such as piping or rill erosion.

At the end of construction, any in service total settlement of embankments shall not compromise the flood immunity requirements.

Any in service movements shall not cause deformation of the cross-section profile to an extent that compromises subsurface drainage efflux or increases the depth of surface runoff flow. Both the design and maintenance phases should consider treatment options that accommodate potential deformation of the cross-section profile.

Embankment settlements and lateral movements of the subsoils shall not impose adverse impact on existing and/or new structures, earthworks, and public utility plant (PUP) infrastructure to an extent that would compromise their serviceability and/or structural integrity.

Batter erosion control measures such as revegetation and surface drainage shall be included in the design to minimise erosion and deterioration of the embankment batters. Flammable erosion control products shall not be used where the risk of fire exists. Designers shall consult the Administrator for any exemption.

If the differential settlement exceeds the values given in Table 3.3, the Contractor shall undertake the following:

- For flexible and concrete pavements surfaced with asphalt, re profile the pavement to the original design level or an alternative road surface geometry that complies with the design requirements of the Contract, prior to practical completion and during the Defect Liability Period.
- For concrete pavements not surfaced with asphalt where unplanned cracking has occurred, the Contractor shall 'slab jack' the pavement with a suitable medium and process to restore the original design level or an alternative road surface geometry that complies with the design requirements of the Contract, prior to practical completion and during the Defects Liability Period.

  Where unplanned cracking in the concrete base has occurred, the Contractor shall, unless approved otherwise by the Administrator, remove and replace the cracked slabs with new pavement in accordance with MRTS40 *Concrete Pavement Base*.

Wherever the term 'Defect Liability Period' is referred in this document, it shall be replaced with:

- 'Defect Correction Period' for Design and Construct Contracts using a Collaborative Project Agreement; and
- 'Operation and Maintenance Period' for a Public Private Partnership.

To confirm that the performance of embankments meets the requirements stipulated in Section 3.3, the Contractor shall carry out adequate instrumentation monitoring and analysis. Before handing over the asset to the department at the end of Defect Liability Period, the Contractor shall demonstrate that the performance of embankments complies with the settlement criteria defined in Table 3.3. That is, the projected settlements based on the monitoring shall be less than the permissible amounts. The extrapolation of settlement over the design period for compressible subsoil areas shall be carried out using Asaoka's (1978) method in addition to any other method(s).

Table 3.3 – Settlement criteria

Location	Maximum total in-service settlement permissible within 40 years of pavement construction (Design and handover requirement)	Maximum differential settlement at any time (Design and handover requirement)	Maximum differential settlement at any time (Intervention requirement)
Within Structure Zone (as per Section 3.2)	50 mm	Design change of grade due to differential settlement over any 5 m length of pavement shall be limited to 0.5% for sprayed seal granular asphalt over granular and full depth asphalt pavements and 0.3% for all other pavement types, in any direction of the carriageways.	Design change of grade due to differential settlement over any 5 m length of pavement shall be maintained to 0.5%, in any direction of the carriageways during the Defects Liability Period. Settlement shall not create any abrupt step larger than 5 mm.
Away from Structure Zone	Sprayed seal granular, asphalt over granular, full depth asphalt and continuously reinforced concrete pavements, 200 mm. Other pavement types, 100 mm.		

Note: In addition to meeting the design change of grade requirements due to differential settlement, the pavement shall meet the 'Aquaplaning' standards outlined in the department's *Road Planning and Design Manual – 2nd edition*. Furthermore, no part of the embankment should experience lateral movement in any direction.

3.4 Geotechnical design for unreinforced embankments

The geotechnical design report shall include the following:

a) the development of geological models, and geotechnical long and cross-sections, which depict the stratigraphy of the subsurface materials with delineation of potential drainage boundaries

b) the interpretation of subsurface strata along with their geotechnical properties / parameters and the adopted design strength and compressibility parameters – the adopted design strength and compressibility parameters shall be justified

c) the design pore water pressures, both the existing and the anticipated worst conditions, shall be adopted where relevant with justification

d) stability analysis in accordance with the requirements in Section 3.4.1

e) settlement analysis in accordance with the requirements in Section 3.4.2

f) the development of a geotechnical monitoring program (as per Section 3.9), in respect of possible pore water pressures and/or embankment / subsoil movements during construction and maintenance, must include the department's long-term maintenance after completion of the construction contract

g) anticipated construction related issues including, but not limited to, the rate of filling, and

h) characterisation of materials proposed for use in construction, including control measures proposed to mitigate against the risk of incorporating slaking and/or dispersive soils into the embankment.

3.4.1 Stability analysis

Stability analysis for the geotechnical design of embankments shall incorporate and comply with the following:

a) Design philosophy:

i. Limit equilibrium methods based on traditional FOS (that is, Factor of Safety from two dimensional limit equilibrium analysis) shall be used.

ii. Soft clay foundations shall be modelled for short-term behaviour using total stress analysis (that is, 'Total Stress Basis'), as well as for long-term (in service) behaviour using effective stress parameters ('Effective Stress Basis').

iii. The embankment material shall be modelled using drained strength parameters (that is, 'Effective Stress Basis').

iv. The minimum FOS during construction (short-term) shall be 1.30 and in service (long-term) shall be 1.50. However, for the bridge spill-through abutments, a minimum long-term FOS of 1.40 shall be achieved as a standalone element by excluding the contribution of the bridge foundation but considering the restraint provided by the earth pressure against the abutment headstock.

v. The minimum FOS for rapid drawdown and seismic condition shall be 1.20 and 1.10 respectively, while supporting live load as specified in this document (see Section 3.4.1(b). For seismic assessments, the minimum annual probability of exceedance (refer to Clause 3.1 of AS 1170.4) shall be 1/500; however, 20% reduction of shear strength parameters may not be required for seismic stability assessments if the risk of potential liquefaction is low.

vi. The following potential modes of failure shall be investigated where relevant:

- both circular and non-circular slip surfaces
- sliding failure across the top of basal reinforcements
- bearing capacity failure, and
- settlement of the embankment, resulting from excessive elongation of the basal reinforcement.

vii. Global stability analysis shall confirm that the embankment foundation is not subject to long-term creep movements of pre-existing landslides or other forms of intrinsic land instability.

viii. The influence of any disturbance due to ground improvement schemes and the loading imposed by the proposed constructions on any adjacent structures and earthworks elements and services shall be investigated and reported.

ix. The relevance of seismic stability issues shall be investigated.

x. Sudden drawdown effects, if relevant, shall be checked (refer Section 3.6).

b) Loads and geometry:

   i. Minimum of 20 kPa (for roadway) uniformly distributed live loading for long-term conditions and a minimum of 10 kPa uniformly distributed live loading for initial construction shall be adopted across the top of the embankment cross-section. For footpaths and cycleways, 10 kPa shall be used for long-term conditions unless need for larger vehicles.

   ii. The impact of any existing excavations and/or known proposed (or future) excavations on the embankment stability shall be assessed.

c) Material parameters:

   i. The minimum unit weight of embankment materials shall be 20 kN/m^3 unless otherwise substantiated by the use of lightweight material.

   ii. Embankment shear strength parameters for earth-fill shall not exceed $c' = 5$ kPa and $\Phi' = 30°$ (for 'Class A1' and 'Class B' materials as per Table 14.2.2 in MRTS04 *General Earthworks*) while for rockfill, $\Phi' = 40°$.

   iii. For embankment greater than 10 m height, laboratory shear strength testing, for example, triaxial CU (Consolidated Undrained) tests with pore water pressure measurements as a minimum, shall be carried out on recompacted samples to evaluate the shear strength of the embankment fill materials if other than 'Class A1' or 'Class B' materials or rockfill as per MRTS04 *General Earthworks* are intended to be used.

   iv. In addition to the geotechnical model requirements outlined in Section 1, the design geotechnical parameters adopted in the assessments shall be moderately conservative. These parameters should typically be equal to or above the lower quartile value but lower than the median value when characteristic values are determined using an appropriate Probability Density Function (PDF) such as a lognormal PDF.

d) Geotechnical model:

   i. Scaled cross-sections of the embankment with subsurface models depicting the design material properties, representative ground water condition, and ground improvement elements and their associated design parameters shall be established.

e) Method of analysis:

   i. Two dimensional Morgenstern and Price method shall be the primary method of limit equilibrium analysis.

   ii. In addition to this deterministic analysis either a sensitivity or probabilistic analysis is required for all GE3 related works:

      - for a sensitivity analysis, the material strength shall be varied by one standard deviation, and

      - for a probabilistic analysis, a lognormal distribution shall be considered where appropriate.

f) Software:

i. Industry accepted software SLOPE / W or SLIDE shall be used to carry out limit equilibrium analyses required by Section 3.4.1. The submission shall include critical sections analysed, and if requested by the reviewer, the data files compatible with SLOPE / W or SLIDE shall be submitted to the Administrator, who will then forward them to the department's Geotechnical Section for further review. Any potential increase in shear strength of the soil above water table due to suction shall not be considered in these assessments.

g) Presentation of stability analysis:

i. The geotechnical design documentation shall include a report on the embankment stability analysis. The embankment stability analysis report must:

- clearly indicate the geotechnical models, design strength parameters and pore water pressure conditions adopted, and the assessment method – these shall be supplemented with design calculations where appropriate.
- include cross-sections with chainages marked. These cross-sections shall show the centres of slip circles investigated and shape of the most critical circle or non-circular surface for the different critical stages of the embankment construction phase and for the design life.

3.4.2 Settlement analysis

Settlement analysis for geotechnical design of embankment(s) shall comply with and address the following:

a) Design philosophy

i. Settlement analysis based on conventional Terzaghi's 1D consolidation theory shall be used as the primary method. 2D or 3D numerical models can only be considered as secondary method(s).

ii. The influence of strain rate effects and structural phenomena shall be addressed where relevant.

iii. Secondary consolidation of the foundation (creep) shall be considered.

iv. The influence of continuing deformations, both vertical and horizontal, imposed by the proposed construction on any adjacent structures and earthworks elements and services shall be investigated and addressed.

v. The performance of existing services or adjacent structures or infrastructures in the light of settlements induced by the new construction should be documented as part of the design process.

vi. The influence of preloading, surcharging, staging and ground modification shall be investigated with respect to both primary and secondary settlements.

vii. Creep of the embankment itself where relevant (for instance, in high embankments, say more than 10 m) shall also be considered in the long-term settlement calculations.

> Elastic settlement of embankments typically occurs instantaneously during construction. Predicting and measuring such settlements pose practical challenges due to various factors. Therefore, allowances for this settlement should be factored into the estimation of material quantities during tendering.

b) Geotechnical model

The geotechnical model for settlement analysis must clearly show the following in addition to requirements presented in Section 1:

i. geotechnical long and cross-sections

ii. natural moisture content compared with liquid limit and plastic limit

iii. the profiles of pre-consolidation pressure

iv. coefficient of volume compressibility (m_v)

v. compression index (C_c)

vi. recompression index (C_r)

vii. initial void ratios

viii. coefficient of consolidation (c_v)

ix. coefficient of secondary compression (that is, creep coefficient), and

x. adopted over consolidation ratios (if applicable).

Any embedded sand layers must also be shown. Where primary consolidation of the foundation will not occur under the applied embankment loads, the geotechnical model shall include elastic moduli for each geological unit.

c) Settlement parameters

In assessing the geotechnical parameters for settlement analysis, their stress dependence shall be taken into consideration, if applicable.

d) Presentation of settlement calculations

The geotechnical design documentation shall include a report on the embankment settlement analysis. The embankment settlement analysis report shall:

i. clearly indicate the critical geotechnical design profiles with design settlement parameters, drainage boundary conditions adopted, design standards complied with and loading conditions adopted, and

ii. provide the settlement time history plots along with preloading and surcharging details (if applicable) and the embankment location.

3.5 Additional design requirements for side-long embankments

Embankment foundations shall be excavated to a competent material in accordance with the design and as assessed / verified by an experienced RPEQ Civil Engineer who is competent in the field of geotechnical engineering / Engineering Geologist after stripping all loose materials and/or uncontrolled fill. Designs must define the expected depth to a competent material for the foundations.

Side-long embankments are road embankments along the side of natural slopes (or hills). Often the road is constructed by excavating material from the uphill side and placing it on the downhill side to form a level surface.

The stability of the side-long embankments is often affected by the changes to the groundwater during prolong rainfalls and storms. Therefore, the geotechnical slope stability of the identified critical side-long embankment shall be assessed for the most critical groundwater condition that could reasonably be anticipated over its design life.

In addition, the embankments on side-long slopes shall be free from any in-service movements along slip surfaces.

Surface and subsurface drainage design should consider both existing and future worst anticipated groundwater conditions, magnitude of rainfall events, topography and nature of anticipated maintenance over the design life of the road.

For side long embankments traversing natural slopes of steeper than 14º (that is, greater than 1 (vertical) to 4 (horizontal)), the following drainage measures shall be addressed in the design, especially for an embankment height greater than 10 m (toe to crest):

- toe drainage, and
- basal drainage (longitudinal and transverse drains).

These are subjected to groundwater conditions and the size of the site catchment area.

An example is presented in the sketch below (see Figure 3.5(a) and (b)) for clarity.

Figure 3.5(a) – An example of sidelong embankment with basal and toe drains – Typical cross-section

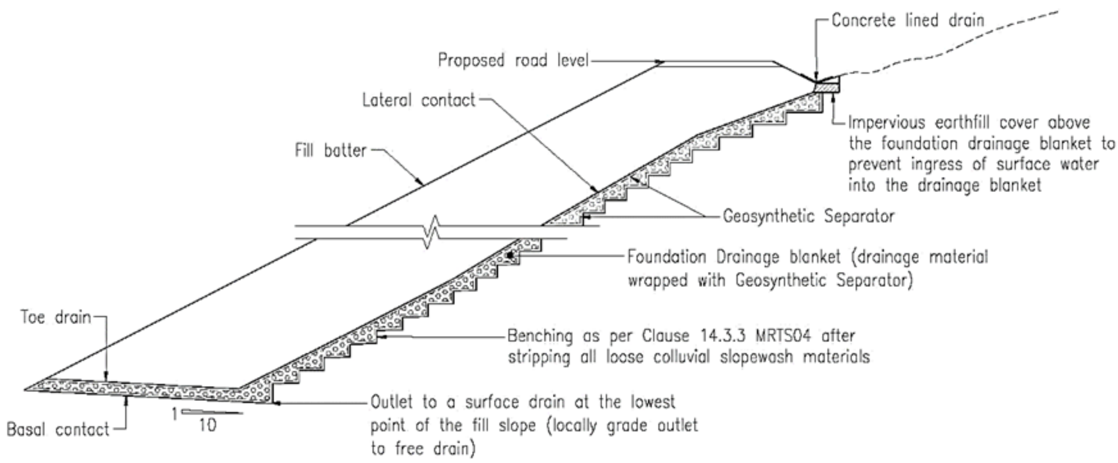
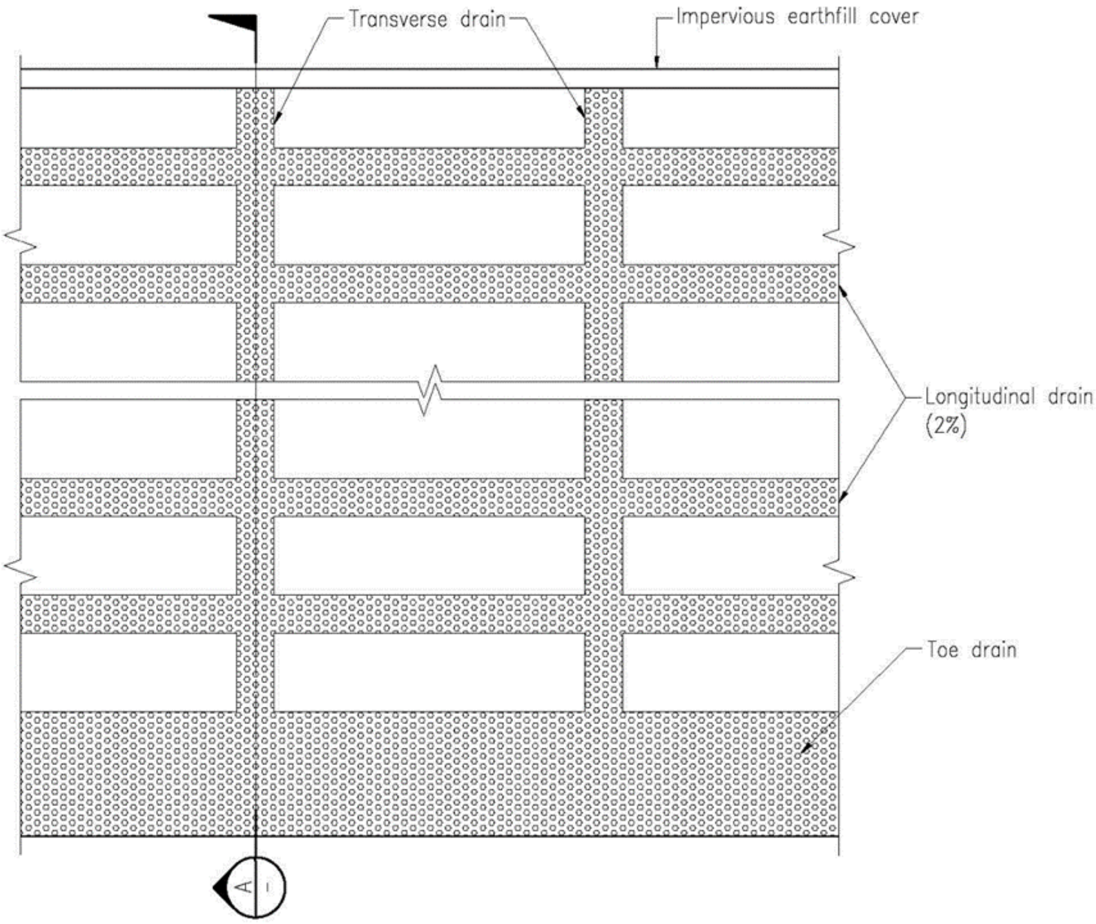


Figure 3.5(b) – An example of sidelong embankment with basal and toe drains – Plan view showing typical drain layout



3.6 Embankment subject to permanent/semi-permanent toe inundation

Permanent inundation: where an AEP 5% ARR2016 flood event with climate change in a relevant creek or river is predicted to inundate the toe of the embankment for a duration equal to or greater than 12 hours.

Semi-permanent inundation: where an AEP 5% ARR2016 flood event with climate change in a relevant creek or river is predicted to inundate the toe of the embankment for a duration of less than 12 hours.

In addition to requirements stipulated in Section 3.1 to 3.4, the following requirements shall be fulfilled in the design and construction of embankments subject to permanent and/or semi-permanent inundation:

- Embankments below permanent or semi-permanent inundation levels shall be constructed with moisture insensitive material with respect to strength, dispersion, and volume reactivity in addition to satisfying the requirements of Clause 14.2.5 *Water Retaining Embankments* of MRTS04 *General Earthworks*.

- In addition to the department's *Road Planning and Design Manual – 2nd Edition* and notwithstanding the above requirements, embankment batters shall be designed and protected to ensure that the road can be opened to traffic following any flood up to an AEP 1% ARR2016 flood event.
- The stability analysis of embankments subject to permanent and semi-permanent inundations shall demonstrate their safety against flood velocities, seepage forces, drawdown effects and ponding / wave action. Water level within the embankment for drawdown analysis shall not be lower than permanent inundation level.

For approach embankments (that is, within the structure zone) to bridges over watercourses and culverts within waterways (existing or manmade), the following additional requirements shall be fulfilled in the design and construction of embankments subjected to permanent and/or semi-permanent inundation:

- The embankments batters shall be protected against saturation, seepage, erosion and scouring at the toe. Therefore, the following preventive measures shall be provided as a minimum:
  - covered by a material such as sheet filter (for example, refer to MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*) that prevents fines from leaching from the embankment during all conditions, including drawdown.
  - The surface of the sheet filter material shall be covered by outer sheathing materials (for example, rockfill) that:
    - meet the urban design requirements in accordance with the department's *Road Landscape Manual*
    - have a design life of 100 years
    - hold the sheet filter material in place under all conditions and protects it from degradation
    - are flexible and accommodate potential movements in the embankment
    - protect the embankment batters from any damages including damage caused by flood flows, and
    - include treatments that address hydrostatic pressure and pore water pressure where appropriate (such as weepholes).

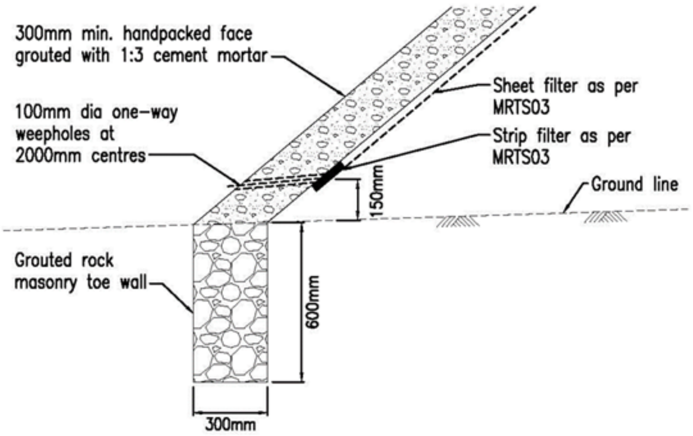
Embankments with flood immunity less than AEP 1% shall be assessed and treated to mitigate potential flood damage in accordance with departmental requirements including the department's *Road Planning and Design Manual – 2nd edition.*

In addition to the above requirements for the batters of the embankments below permanent inundation, the following additional requirements shall be met:

- the outer sheathing materials shall be a water-resistant material, and
- include a toe wall that must be sufficiently deep to prevent undermining of both the embankment and the outer sheathing materials due to any changes in the watercourse.

An example of batter protection is shown in Figure 3.6. Grouting of the rockfill is not required for semi-permanent inundation case. Spill through embankments shall also be designed as per Clauses 38 to 43 of MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*.

Figure 3.6 – Example of a batter protection within structure zone



3.7 Reinforced embankments

For reinforced embankments with face angle up to 70° from the horizontal, the primary method of design shall conform to British Standard 8006 *Code of practice for strengthened/reinforced soils and other fills* (BS 8006) in addition to the requirements stipulated in Section 3.4 which requires analyses to be carried out using Morgenstern & Price method as opposed to the Bishop method stated in BS 8006.

Embankments with face angle steeper than 70° are considered as a Reinforced Soil Wall (or Reinforced Soil Structure) and Section 6.5 shall apply.

3.8 Ground improvement

Any adopted ground improvement schemes shall either have a proven record of successful in-service performance from similar projects in Queensland with comparable geological conditions, cause an acceptable level of environmental impact, and be demonstrated as appropriate for the site conditions. This is demonstrated by completing the following:

a) detailed analyses presented as a report, which shall be submitted to the Administrator for independent review by the department's Geotechnical Section at the time of proposal, or

b) conducting appropriate field trials, accepted by the Administrator based on the advice of the department's Geotechnical Section, to verify that the proposed method can satisfy the critical performance aspects outlined in Section 3.3 and limit the impact on adjacent structures and utilities.

It is also the responsibility of the designer to ensure the adequacy of the ground investigation and testing for choosing a cost-effective ground improvement scheme. The interpreted ground conditions and any proposed ground improvement measures shall be discussed with the departmental Geotechnical Section through the Administrator before commencing on any ground improvement design. This discussion requires the designer to submit appropriate geotechnical long sections and cross-sections to characterise the interpreted geological conditions to the Administrator. In addition, graphical presentations of index properties, strength, stress states and consolidation parameters for layers to be treated shall also be prepared and submitted to the Administrator.

The design of basal reinforced embankments (for example, basal reinforcement beneath embankments and basal reinforcement for rigid column, including piles, supported embankments) over compressible foundation soils shall be in accordance with BS 8006. For the rigid column supported embankments, the basal reinforcement (that is, the load transfer platform (LTP)) shall not be omitted, and the spacing of the columns shall be limited to maximum five times the diameter of the columns.

3.9 Geotechnical instrumentation monitoring for embankments

The geotechnical monitoring program for embankments, where relevant (refer to Section 3.4(vi)), shall be documented on the drawings.

The geotechnical monitoring program for embankments shall:

- address the instrumentation provisions for monitoring of pore water pressures, embankment and subsoil movements, with justification for their use, and the design objectives they are expected to clarify, and
- detail the nature of the instrumentation, the locations (physical surveys with Universal Transverse Mercator (UTM) coordinates and an elevation from AHD i.e. Easting, Northing, RL) within the ground where the instruments are to be installed (on cross-sections), monitoring frequency and contingency plans with other relevant details.

The geotechnical monitoring program for embankments shall be implemented and maintained throughout the construction of embankments and pavements.

All geotechnical instruments shall be protected from vandalism and construction activities over its operational life. Damaged or malfunctional instruments shall be reinstated or reinstalled immediately to reduce the impact on the monitoring program.

Instrumentation at identified critical locations shall be provided to enable the continuation of monitoring of critical elements during the Defect Liability Period or maintenance period (whichever is longer) of the project.

All monitoring data and reports shall be submitted through the Administrator to the department’s Geotechnical Section in electronic form.

The department’s preferred method for capturing and storing monitoring results is through a web-based data acquisition system. Consideration shall be given to adopting this method, including providing the department’s Geotechnical Section with access to both live and historical data, as well as implementing robust archiving methods for future access.

> The in-service settlement requirements are typically small, for example, less than 50 mm over a 40 year period. This underscores the need for highly precise surveying techniques and stable benchmarks. The Designer / Contractor should take this into account when choosing suitable surveying techniques and installing target monuments.

3.10 Performance monitoring

The geotechnical monitoring program for embankments shall be developed by the designer and agreed by the department's Geotechnical Section. This program shall continue to be implemented and maintained throughout the Defect Liability Period or maintenance period (whichever is longer) until the Final Completion. The department may choose to extend this program for longer term maintenance.

In addition to the geotechnical instrumentation monitoring:

- the Designer / Contractor shall select locations for the physical survey monitoring program to establish longitudinal and transverse settlement profiles and other movements as required to confirm the performance standard in Section 3.3, and
- visual inspections and straight edge measurements shall be undertaken to capture surface subsidence and deformations.

The geotechnical monitoring program for embankments shall include the production of inspection reports, interpreted instrumentation monitoring reports and improvement works reports.

The results of the embankment geotechnical monitoring program during the Defect Liability Period shall be used:

- to assess the need for any remedial / maintenance works. If monitoring identifies the need for remedial works, this need will be considered a Defect under the contract, and it is the contractor's responsibility to remediate that Defect.
- in the design of remedial works, if required, and
- to assess any requirements for ongoing monitoring.

At the Final Completion, the following information must be provided to the Administrator:

- all monitoring data in electronic format, and
- details of all active instrumentation for further monitoring by the department.

The Administrator will forward this information to the department's Geotechnical Section for review and acceptance of performance.

4 Cut slopes

4.1 General requirements

For unreinforced cuts, cut slopes shall not be steeper than 2 (vertical) to 1 (horizontal). The maximum vertical height of any single continuous cut shall, in most cases, shall not exceed 10 m. A minimum 4 m wide bench shall be provided for erosion control, control of rockfall and maintenance purposes at the top of any 10 m high single continuous cut slope.

The bench and the batter must be adequately protected against erosion. Berm drain shall be provided at each bench as per MRTS04 *General Earthworks*. Cuts in erodible or dispersive geology may require different strategies, for example, flattening without benches.

Needs for such cut slope treatments shall be submitted to the department's Geotechnical Section in writing and agreed prior to its implementation.

For reinforced cuts (for example, soil nail / rock dowel walls) cut slopes shall not be steeper than 10 (vertical) to 1 (horizontal). A minimum 4 m wide bench at every 10 m in height as per unreinforced cuts shall be provided.

4.2 Performance standards

The cut slopes shall be stable for the full duration of their design life and shall require low whole of life maintenance, with due consideration of the influence of local climatic and geological conditions on stability and erosion issues.

> The term 'stable' cut slopes, as used in this document, refers to cut slopes that have been designed and constructed in adherence to all performance and other minimum requirements stipulated in this *Geotechnical Design Standard*.

Suitable construction techniques and interventions during construction and maintenance shall ensure to mitigate the impact on road users, residents and their dwellings, commercial properties, services and the environment.

Slope protection measures shall be carried out in a timely fashion, soon after the completion of each batter, to mitigate the development of instability and erosion issues and deterioration of the cut face. In addition, all batter protection works for each cut shall be completed no later than one month after the full cut construction. The slope treatments shall incorporate finishes aesthetically compatible with the surrounding streetscape and environment.

Flammable slope protection products shall not be used where the risk of fire exists. Designers shall consult the Administrator for any exemption.

Where ground reinforcement techniques are used, proof testing of selected slope reinforcement elements shall be carried out as required by the relevant Standards and departmental Technical Specifications.

4.3 Design requirements

4.3.1 General

A geotechnical risk assessment based on preliminary analyses shall be carried out to identify whether the issues in Section 4.3.2 and 4.3.3 need to be addressed in order to satisfy the performance standards stipulated in Section 4.2. This risk assessment shall be submitted through the Administrator to the department's Geotechnical Section, who will advise on the suitability of the risk assessment. A written confirmation, stating that the department has no objection to the risk assessment, must be obtained from the Administrator before the requirements under Section 4.3.2 and 4.3.3 are dispensed with. A representative groundwater condition shall be considered in the design. Particular attention shall be given to long-term stability conditions as this would be generally critical for cut slopes and excavations.

4.3.2 Unreinforced cuts

The geotechnical design for a cutting shall include:

- The development of design geological profiles, which show the different subsurface strata with their lithologies, weathering states and structural defects, where practicable, based on factual data, geological mapping, borehole imaging and knowledge of local geology.
- Design life.
- Imposed and dead loads.
- Impact of the proposed cuttings on existing and new structures.
- Stability analysis in accordance with the requirements in Section 4.3.2.1 below.
- A quantified estimate of the stress relief effects associated with the cutting and an assessment and mitigation of impacts that these may have on the long-term stability of the cutting.
- The development of a geotechnical monitoring program that considers groundwater level and slope stability / movements during both construction and maintenance. For cuts assessed as posing a high risk to road users and/or adjacent properties, continuous remote monitoring should be implemented.
- Assessment of the erodibility / dispersivity of the slope materials and the design of appropriate batter treatments or protection where required.

4.3.2.1 Stability analysis

Stability analysis for a geotechnical design of a cut slopes shall comply with and address the following:

a) Design philosophy:

   i. In parts of cuttings characterised by soil and 'soil like' extremely weathered rock, circular and non-circular failure mechanisms shall be considered in design. Whereas, in parts of cuttings characterised by moderately weathered (MW) or better rock, structurally controlled failure mechanisms shall be investigated (including toppling, planar sliding or wedge failure modes).

   ii. Moderately conservative values of design parameters as per Section 3.4.1(c) shall be adopted for the assessments.

   iii. The minimum FOS during construction (short-term) shall be 1.30.

   iv. At any parts of cuttings, the minimum FOS shall be 1.50 (long-term, in service), with a representative ground water condition. As a minimum, a pore water pressure coefficient (R_u) of 0.15 shall be used even with appropriate drainage systems.

   v. The potential for instability due to undermining because of differential weathering and erosion shall be addressed.

   vi. Potential susceptibility to rapid softening and deterioration of some lithologies shall be investigated.

   vii. Any requirements for a staged excavation approach shall also be assessed.

viii. Cut slope designs based on prescriptive measures using observed performance of existing road cuttings in similar geological conditions with consideration of long-term stability and low maintenance costs may be acceptable. Such departures shall be submitted to the department's Geotechnical Section in writing through the Administrator for review and acceptance.

ix. The design considerations which shall be addressed include, but shall not be limited to, the influence of groundwater on stability, recognition of soft infill materials in discontinuities, and allowance for disturbance effects associated with excavation techniques, surface water run-off and erosion.

b) Fissured soil:

i. Mass operational strengths which capture the relatively lower strength of fissures / slickensides surfaces shall be adopted.

c) Method of analysis:

i. Two-dimensional limit equilibrium analysis using Morgenstern and Price method shall form the primary method of analysis for soil like stability problems. For structurally controlled rock stability problems and for characterising discontinuities in rock, kinematic stability analysis shall be carried out.

d) Software:

i. As per Section 3.4.1(f) for soil and soil like materials.

4.3.2.2 Geotechnical monitoring

A geotechnical monitoring program of cut slopes, addressing groundwater and/or slope movements as outlined in the beginning of Section 4.3.2, shall include the following details:

- nature of the instrumentation
- locations (including physical surveys with Universal Transverse Mercator (UTM) coordinates and elevations from AHD; i.e. Easting, Northing, RL) within the ground where the instruments are to be installed (on cross-sections)
- monitoring frequency, contingency plans with other relevant details, and
- instrumentation at identified critical locations shall remain to enable the continuation of monitoring of critical elements beyond the Defect Liability Period.

4.3.2.3 Presentation of stability analysis

The geotechnical design documentation shall include an RPEQ certified report on the stability analysis of the cut slopes. The stability analysis report shall include:

- geotechnical models, including any geotechnical domains, rock mass classification, the design strength parameters, pore water pressure conditions adopted, design standards complied with and supplemented with design calculations where appropriate
- analyses of kinematic and/or circular and non-circular failure modes, and
- design of cut face and stabilisation treatments including associated drawings.

4.3.2.4 Rock fall analysis

Rock fall modelling shall be carried out on all major rock cuttings with an overall height greater than 10 m in height, with appropriate design to ensure rock fall debris does not present a hazard to the road users.

4.3.3 Reinforced cut slopes

For reinforced cut slopes up to 70°, the following requirements shall apply, in addition to those stipulated for unreinforced cuts in Section 4.3.2. Reinforced cut slopes steeper than 70° are considered as walls, and the requirements of Section 6.5 shall apply.

The design of insitu slope stabilisation measures shall be based primarily on BS 8006, as well as Technical Specification MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protection*s. The use of BS 8006 will override the FOS stipulated in Section 4.3.2.1.

The design shall also consider the following:

- overall stability and internal failure mechanisms both during construction and the long-term
- durability and allowance for construction damage of reinforcing elements, and
- the behaviour of the ground under stressing loads.

In addition to the requirements in Section 4.3.2, any design involving in-situ stabilisation treatments must be documented, along with the associated drawings.

4.3.4 Construction

A geotechnical monitoring program for groundwater and/or slope movements shall be documented in the Contractor's earthworks and construction plans and drawings.

The geotechnical monitoring program for groundwater and/or slope movements shall be implemented and maintained throughout the construction of cuttings until final completion of the Contract.

The following activities shall be undertaken by the Contractor / Designer as part of the geotechnical monitoring program during construction:

- visual inspection of slope materials during excavation to verify the design assumptions as well as water / movement monitoring
- progressive review of conditions and data that become available during construction and, if necessary, modification of cut slope design, subsurface drainage requirements and construction sequencing
- regular monitoring of installed ground instruments including during critical phases of construction and after significant rainfall events
- regular auditing of instrument status and fixing / replacing damaged or malfunctioning instruments as soon as practicable
- progressive review of excavation methodology during construction including temporary support systems
- identification and assessment of potential local instability and adoption of remedial measures as soon as practicable to mitigate the progression of such local failures.

In addition, appropriate action (as acceptable by the Administrator, in consultation with the department's Geotechnical Section) shall be taken if such local conditions are deemed to:

- compromise the cut slope stability during its design life, and/or
- present an unacceptable environmental impact (or the potential for an unacceptable environmental impact), and/or
- impact on the safety of the road user or construction and maintenance workers.
- execution of required proof testing for slope reinforcement.

The performance monitoring requirements in Section 3.10 shall also apply to cuts.

5 Bridge and other structure foundations

5.1 General

5.1.1 Structural aspects

Reference shall be made to the department's *Design Criteria for Bridges and Other Structures* for durability, structural and other requirements not covered here.

5.1.2 Geotechnical aspects – geotechnical investigation and reporting

Geotechnical investigation for the design and construction of foundation shall be carried out for all bridges and other structure foundations. Scope briefing for all geotechnical works must be acceptable to the department's Geotechnical Section before the commencement of any geotechnical site investigation as per Section 1.

The geotechnical investigation shall adequately inform the design while ensuring that the site geological model can reasonably be established. Unless otherwise approved or directed by the department's Geotechnical Section, a minimum of two boreholes shall be drilled at every abutment and pier location.

With a view of further reducing the chances of latent conditions during construction, the number of boreholes to be drilled at a particular site will depend on how well the site geology could reasonably be established. To achieve this aim, the subsurface geological model shall be updated as the drilling is continuing.

The geotechnical and structural engineers responsible for a project shall be satisfied that the information obtained from a particular site is adequate for the foundation design before the drilling contractor demobilises from the site. Generally, the boreholes shall be drilled at intervals not exceeding 10 m along the width of every abutment and pier of all bridges. It is recommended to use a 3D geological model to help assess the adequacy of the information.

To avoid doubt, twin bridges shall be treated as separate bridges. For other structures, the details of Geotechnical Investigations shall be discussed and approved by the Administrator in consultation with the department's Geotechnical Section.

For sites where Prestressed Concrete (PSC) driven piles are likely to be the foundation option, all boreholes shall be extended to at least 5 m into substrata with consecutive Standard Penetration Test (SPT) number greater than 50 (SPT N > 50), if the expected toe level cannot be estimated at the time of the investigation.

For sites where Cast in Place (CIP) piles are likely to be the foundation option, all boreholes shall be extended to a minimum of 5 m into competent bedrock (moderately weathered and medium strength or better rock).

Geotechnical investigations for Driven Tubular Steel (DTS) pile foundations shall penetrate at least 5 m beyond the expected toe of the proposed DTS piles. If the expected toe level is not known at the time of the investigations, the investigation depths for DTS piles shall be discussed with and approved by the Administrator in consultation with the department’s Geotechnical Section.

The geotechnical design report(s) for foundation shall, at a minimum, include the following:

- Geological models prepared for each foundation location in complex geological terrain. These should capture essential geological elements that may assist in the design, including subsurface stratigraphy within the investigated depths. They should also illustrate various lithologies and their weathering grades, demarcate potential zones of water ingress, and highlight structural defects, such as clay seams, fault, and sheared zones. Wherever possible, 3D geological models shall be used.
- Design parameters with supporting justification.
- Design calculations for geotechnical ultimate limit state axial and lateral capacities of pile(s), and serviceability limit state of movement, where applicable.
- Design calculations for bending moments, shear forces and deflections in the pile(s) under lateral loading where relevant.
- Group effects when estimating settlements and the distribution of load within the piles in a group.
- Design of approach embankments (Refer to Section 3.2. and Table 3.3).
- Spill through and retaining walls for abutment supports, as applicable, and
- Construction considerations including, but not limited to, staging of earthworks and piling operations.

5.2 Deep foundations

5.2.1 Design philosophy

Piles shall be designed to support the design loads with adequate geotechnical and structural capacity, while ensuring tolerable settlements and lateral deflections in accordance with the performance requirement of the structure. Although not exhaustive, a compliance design shall:

- ensure that all piles satisfy the ultimate limit state requirements with appropriate load and resistance factors
- ensure that, at the serviceability limit state, foundation settlements, differential settlement between the foundations (abutments / piers), and any lateral movements are consistent with the performance requirements of the superstructure
- recognise the overriding influence of site geology, construction methodology and quality control on rock mass properties in the case of CIP piles

- eliminate any contribution from base in cases where there is uncertainty regarding the end bearing in the design of CIP piles
- ensure that the piles are constructible, considering subsurface conditions, site setting, and constraints. For driven piles, a drivability analysis shall be performed to demonstrate that the piles can be driven to their design toe levels.

5.2.2 Design methodology

5.2.2.1 Axial capacity of piles

Driven prestressed concrete (PSC) and steel piles (for example, H piles):

- Design of driven piles shall be carried out based on Australian Standard 5100.3 (AS 5100.3) and MRTS65 *Precast Prestressed Concrete Piles* or MRTS66 *Driven Steel Piles*, where relevant; however, the geotechnical reduction factor (øg) shall not be greater than 0.65.
- The axial capacity of the piles shall only be based on static capacity calculations using moderately conservative design parameters as per Section 3.4.1(c) and site-specific geotechnical profiles.
- The design skin friction and end bearing values shall be derived using the widely accepted methods (such as effective stress method, alpha method, CPT based methods, SPT based method). Driving allowances, that is, underdrive and overdrive, shall only be based on the static capacity calculations based on upper and lower bound geotechnical models, respectively. Particularly, a clear justification for the 'underdrive' should be provided. Pile driveability assessments shall be based on geotechnical models exerting the worst possible driving resistance and driving stresses. Setup shall not be considered in pile design.
- Piles at bridge abutment locations shall not be driven until the estimated post construction settlement of the approach embankment is reduced to less than 100 mm over 100 years by preloading or otherwise. Any expected residual settlement of the approach embankment after a pile is driven shall be considered in the design. Consideration shall be given to the settlement of individual piles and pile groups resulting from negative skin friction caused by settlement of the surrounding ground.
- Driven piles shall be tested to ascertain their capacity and integrity. The testing shall be carried out with Pile Driving Analyzer (PDA) as per MRTS68 *Dynamic Testing of Piles*.
- The minimum number of piles PDA tested shall be the greater of:
  - 15% of piles per pier / abutment bent, or
  - minimum one pile per pier / abutment.
- First pile to be installed in each abutment or pier shall be subjected to high strain dynamic (PDA) testing over the full length of the drive to determine driving stresses, impact energy and geotechnical capacity in addition to establish pile driving parameters for installation of the rest of piles in the foundation system.
- The outputs from the PDA testing shall include an estimate of mobilised axial capacity, an indication of the load-settlement characteristics and an indication of the pile integrity.

- Monitoring of pile driving shall be undertaken on all piles in accordance with the requirements of MRTS68 *Dynamic Testing of Piles*.
- The supplier and operator of the pile driving analyser shall be a company independent of the piling contractor.

CIP Piles not socketed into rock:

- The design shall be carried out based on AS 5100.3 and MRTS63 *Cast-In-Place Piles*, but the geotechnical reduction factor (øg) shall be not greater than 0.55.

Driven Tubular Steel (DTS) piles (with reinforced concrete pile shaft):

- The design shall be carried out based on AS 5100.3 and MRTS64 *Driven Tubular Steel Piles (with reinforced concrete pile shaft)*, but the geotechnical reduction factor (øg) shall be not greater than 0.60. The geotechnical reduction factor (øg) up to 0.70 can be considered, in consultation with Transport and Main Roads Geotechnical Section, if all the piles (100%) are subject to PDA testing.
- In addition, the following requirements are to be satisfied:
  - The designer can establish the design skin friction and end bearing values using local experience from CAPWAP analyses of similar piles in comparable geological settings and under similar driving conditions. In the absence of such data, widely accepted methods (such as effective stress method, alpha method, CPT based methods, SPT based method) shall be used.
  - In deriving the above, the DTS piles are to be considered as a non-displacement (and non-preformed) piles.
  - Moderately conservative design parameters as per Section 3.4.1(c) shall be used for establishing skin friction and end bearing values. For projects where the design shear strength parameters for cohesive soils are established for different consistencies (e.g., soft clay, stiff clay) using project-wide or bridge-specific data, the procedure outlined in Section 1 shall be followed. Additionally, rock strength versus depth plots, indicating the chosen strength line, shall be created for each pile in the bridge pier and abutment.
  - As the self-weight of the concrete shaft in the pile is transferred to the steel tube via shear keys, the concrete shaft self-weight shall be added to the pile load, i.e.to Ed as per AS 2159, incorporating appropriate partial load factors. The unit weight of the reinforced concrete shall not be less than 25kN/m³ unless structural engineers advise otherwise.
  - Adequacy and effectiveness of the shear connectors in the "Stress transfer and composite action zone (refer to MRTS64 *Driven Tubular Steel Piles (with reinforced concrete pile shaft)* for the definition)" to support the self-weight of the concrete shaft and the bridge loads shall be ensured.
  - Weight of the steel tube shall be treated in accordance with Clause 4.4.1 (and equation 4.4.1(1)) of AS 2159. The unit weight of the steel shall not be less than 77kN/m³ unless structural engineers advise otherwise.
  - Axial capacity of the piles shall be based only on static capacity calculations.
  - Design shall be based on unplugged condition.

- Depending on the hydrogeological condition and pile-shoe configuration, the internal shaft friction from the soil column below the bottom of the concrete plug level may be considered in the DTS pile design at the discretion of the designer based on previous local experience. However, the internal unit shaft friction shall be limited to 25% of the external skin friction when internally thickened driving shoes are used and 50% of the external unit shaft friction on the piles with no driving shoe.
- End bearing is only allowed on the steel annulus.
- Driving allowances, that is, underdrive and overdrive, shall only be based on the static capacity predictions based on upper and lower bound ground conditions respectively.
- Particularly, a clear justification for the 'underdrive' should be provided.
- Piles shall not be designed for setup.
- The hammer and the driving gear selection shall be able to drive to the design depths even through any premature 'plugged' conditions.
- Pile driveability assessments shall be based on the geotechnical models imposing worst possible driving resistance and driving stresses in the pile.
- The driving shoe may be omitted if the piles can be driven to the design depths without any delay and damage.
- Piles shall not be terminated above the designed 'underdrive' level without prior approval from the Administrator. The Approval requires a detailed written demonstration that the piles can be terminated above the 'underdrive' level subject to the review and acceptance of the department's Geotechnical Section.
- Plugged condition is not acceptable as a reason for terminating piles above the 'underdrive' depths.
- Detailed methodology related to PDA testing and signal matching procedure as per MRTS68 *Dynamic Testing of Piles* shall be provided by Designer / Contractor before any driving is planned and accepted by the Administrator prior to establishment of piling equipment for the works.
- DTS piles shall be tested in accordance with the requirement of MRTS64 *Driven Tubular Steel Piles (with reinforced concrete pile shaft).*

High Strain Dynamic Testing (HSDT) and analyses, which is commonly known as PDA and CAPWAP, essentially serves only following purposes in this context:

- As a tool for driveability assessment and setting driving parameters for rest of the pile installations in the system
- To confirm driving stresses do not exceed limiting values, and
- To verify that the design pile capacities are achieved without compromising the design intents.

CIP socketed into rock:

- The design of socketed piles shall be explicitly addresses the socket / pile interface (that is, sidewall slip) to obtain the full load deformation response to assist in confirming the ultimate and serviceability criteria.
- Additionally, the following requirements are to be satisfied:
  - Geotechnical reduction factor shall not be greater than 0.55.
  - The ultimate end bearing of the piles may be calculated using the methods given in Turner, J.P. (2006) for massive rocks, jointed rock mass, layered rocks, and fractured rocks. The ultimate end bearing shall not exceed 4.8√qu (MPa), where qu (MPa) is the design uniaxial compressive strength of the intact rock at the pile base or the compressive strength of the concrete, whichever is lower.
  - The design uniaxial compressive strength of rock shall be set at moderately conservative values as per Section 3.4.1(c).
  - The settlement of the piles shall be assessed using the method proposed by Rowe and Armitage (1987), which accounts for sidewall slip, with necessary adjustments for limit state design. As this method is derived for a single rock layer, special care must be taken when applying it to multiple rock layers with significant variations in strength and stiffness. In such cases, careful selection of the socket length for serviceability design is required.
  - For rocks that are stronger than concrete, the concrete strength will govern the available end resistance and side friction.
  - The final design shall be checked with at least a second design method which explicitly addresses the socket / pile interface (that is, sidewall slip) to obtain the full load deformation response to assist in confirming the ultimate and serviceability criteria.
  - For piles socketed into rock, an iterative design methodology developed based on socket inspections to validate the geotechnical model and the design assumptions needs to be ensured. In particular, the load transfer mechanism between the shaft and the base adopted in design needs to be justified based on the socket inspections. Site inspection and verification of constructed sockets shall be carried out in accordance with MRTS63 *Cast-In-Place Piles*. Other requirements which are mandatory for a successful design and construction of sockets are contained in MRTS63 *Cast-In-Place Piles* and MRTS63A *Piles for Ancillary Structures*.

5.2.2.2 Lateral capacity and lateral deflection of piles

Piles shall be designed to have adequate lateral load carrying capacity. The requirement of Clause 4.4.7 of AS 2159 shall also to be satisfied.

The primary method for establishing lateral deflection and capacity shall be the p-y method using non-linear p-y curves for soils and rocks. Secondary methods can include the elastic continuum approach of Poulos (1971a/1971b), subgrade reaction theory (Winkler Foundation), or the characteristic load method (CLM).

5.2.3 Construction

The overriding influences of geology and construction techniques on the performance of CIP needs to be considered. Reference should be made to MRTS63 *Cast-In-Place Piles* for construction related issues that may influence the design and construction of CIP. An objective of all piling construction is to make piles free of defects; therefore, low strain or non-destructive integrity tests shall be carried out to ensure integrity of the constructed CIP. The supplier and operator of the pile dynamic / integrity tests shall be a company independent of the piling contractor.

5.3 Spread footings and strip footings

The design of these footings (for all structures including bridges and culverts but excluding Reinforced Soil Structure Wall foundations) must satisfy the following:

- Spread footings and strip footings shall be designed in accordance with the requirement of AS 5100.3.
- Settlement and differential settlement shall be limited to values that are consistent with the performance requirements of the superstructure.
- Where the footings are founded on natural or cut slopes, the design must ensure both the short-term and long-term stability of the slopes with minimum FOS of 1.50. Due consideration shall be given to factors such as reduced bearing capacity due to loss of ground resulting from slope, groundwater, geological weathering, fissuring, softening, structural defects and climate.
- The effect of volumetrically active soils that manifest in the form of shrink swell shall be considered for all structures, especially for bridges and culverts and light loaded structures such as pavements. Guidance shall be sought from relevant Australian Standards and departmental Technical Notes, such as AS 2870 and *Western Queensland Best Practice Guidelines* WQ35 and WQ37.
- Foundation Inspection and certification, prior to structural constructions, shall be carried out by a Geotechnical Assessor (GA) having qualifications in accordance with Clause 11.2 of MRTS63 *Cast-In-Place Piles*.

6 Retaining structures

6.1 General

All retaining structures shall be designed to ensure that the asset is fit for purpose and guarantees long-term performance. In addition to the requirements stipulated in this section, reference shall be made to the department's *Design Criteria for Bridges and Other Structures* for durability, structural, and other requirements not covered here.

Except for embedded retaining wall, soil nailed wall, and reinforced soil wall, all other walls covered in this document shall satisfy the requirement of AS 4678 for loads and their combinations.

The minimum, long-term design vertical live load shall be 5 kPa unless noted otherwise. Vertical and lateral loads from earthworks (or other effects including structure and infrastructure) on, or adjacent to, the walls shall be included in the design.

Traffic impact and safety barrier loads and other superimposed structural loads (for example, noise barriers) shall be considered in the design of walls where the barrier is connected to the wall or where the wall is within the area of influence of the barrier.

Compaction-induced stresses shall also be taken into consideration.

Supplementary Specifications shall be included with the design for any specific requirements for ground and/or foundation improvement or construction methodology that is not included in current Technical Specification (MRTS) documents.

Retaining walls founded on weak and/or compressible soils require ground improvement unless supported on deep foundations to achieve adequate bearing capacity, global stability, and limit total and differential settlements. Construction of such walls shall only proceed after successful ground improvement works, verified via appropriate instrumentation and monitoring. This monitoring, continuing during and after the works, will assess performance.

The settlement of any portion of the retaining wall shall not exceed the stricter of the settlement criteria applicable to the associated embankment (as given in Table 3.3) or the tolerable wall deformation.

Adequate site investigation and testing along the retaining wall footprint are required for ground improvement, bearing capacity / stability, and settlement estimates. The department's Geotechnical Section shall review and approve the scope of such additional site investigation works before commencement.

6.2 Embedded retaining walls

Design of embedded retaining walls, for example, sheet pile wall, contiguous pile wall, secant pile wall, and so on shall follow the recommendations of CIRIA C760 or the relevant Australian Standard.

The design report shall include the following as a minimum:

- geological model
- geotechnical model
- design parameters
- groundwater conditions
- cross-section and long-section details of the wall
- bending moment, shear force, and deflected shape diagrams for different load cases and anchor / prop loads if any
- anchor / prop details if any
- proof testing program for anchors
- construction sequence, and
- short-term and long-term monitoring programs.

Certification of design and construction shall be as per Section 6.8.

6.3 Reinforced concrete cantilever retaining walls

The design of reinforced concrete retaining walls (RC Walls) shall satisfy the requirement of AS 4678.

The design report must include the following as a minimum:

- geological model
- geotechnical model
- design parameters

- groundwater conditions, and
- cross-section and long-section details of the wall.

Certification of design and construction shall be as per Section 6.8.

6.4 Soil nailed walls

The design of insitu cut stabilisation measures shall be carried out based on BS 8006 or relevant Australian Standard and the department's Technical Specification MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*. The design shall take into account the following:

- overall stability and internal failure mechanisms, both during construction and in the long-term
- impact of the proposed excavation on existing and new structures
- durability and allowance for construction damage of reinforcing elements
- behaviour of the ground under stressing loads, and
- minimum pore water pressure coefficient (R_u) shall be 0.15 even with appropriate drainage systems such as horizontal drains.
- The design report shall include the following as a minimum:
- the design of insitu stabilisation treatments shall be documented with associated drawings and these shall include geological long-sections, site specific cross-sections pertaining to critical chainages with details on reinforcement layouts and drainage details
- a clear documentation indicating the geotechnical models and design strength parameters and pore water pressure conditions adopted, with justification, design standards complied with and supported with design calculations where appropriate
- construction staging and sequence
- proof test loads, and
- short-term and long-term monitoring programs.

Certification of design and construction shall be as per Section 6.8.

> Where ground anchors are used, they shall be designed to the requirement of BS 8081 and/or relevant Australian Standard and departmental Technical Specifications such as MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*.

6.5 Reinforced Soil Structure (RSS) walls

The design of Reinforced Soil Structure (RSS) walls shall conform to MRTS06 *Reinforced Soil Structures*. The design report shall include the following as a minimum:

- geotechnical model
- design parameters and justification
- groundwater condition
- actual cross-section and long-section details of the wall (not typical sections)

- design calculations for internal and external stabilities of the wall
- design calculations for global stability of the wall, certified by an experienced RPEQ Civil Engineer, who is competent in the field of geotechnical engineering
- Supplementary Specifications for any specific ground and/or foundation improvement or construction methodology, and
- assumptions made on design parameters used as select backfill and general backfill. Testing requirements shall be as per MRTS06 *Reinforced Soil Structures*.

Certification of design and construction shall be as per Section 6.8.

6.6 Gabion retaining walls

Gabion retaining walls shall be designed to the requirement of AS 4678. The maximum height of a gabion wall shall be limited to 6 m.

Gabion walls are not allowed under bridge abutments, except for the purposes of facing or for scour and erosion control purposes.

Precautionary measures against fire hazard must be considered in the design of gabions located in high fire hazard areas.

In addition to the requirements outlined in Clause 36 of MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*, the following design / construction requirements stipulated for Clause 53 in the MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections* and Section 6.7 of this document shall be met for gabion walls:

- foundation treatments, including concrete slurry fill and Supplementary Specifications for any specific ground and/or foundation improvement or construction methodology
- foundation construction requirements
- stability
- design report and drawings
- tolerances and level control
- surface runoff behind the wall
- certification of design and construction shall be as per Section 6.8, and
- drainage as per AS 4678.

6.7 Boulder retaining walls

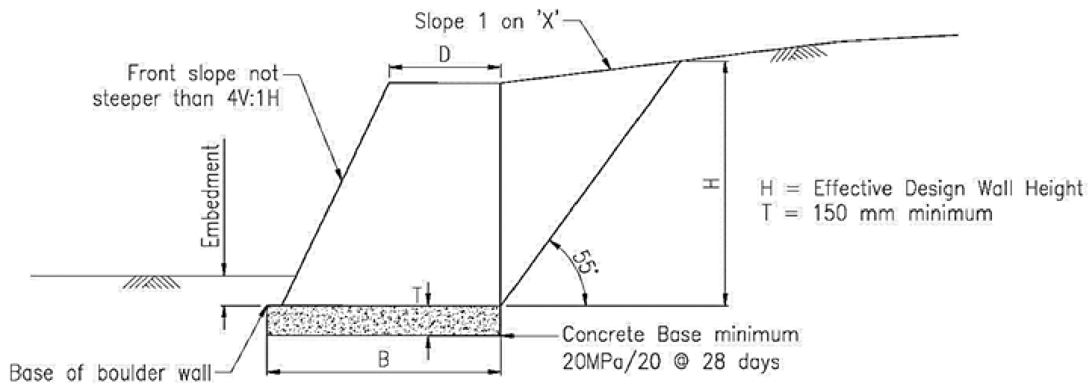
6.7.1 Introduction

In the absence of specific design codes covering boulder retaining walls and the difficulties of carrying out compliance testing, the maximum effective design wall height (refer to Figure 6.7.2) of a boulder wall is limited to 3 m.

6.7.2 Definition of terms

The terms used in this specification shall be as defined in Figure 6.7.2.

Figure 6.7.2 – Typical boulder wall section



6.7.3 Materials

Refer to Clause 53 of MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*.

6.7.4 Design

Design shall be to AS 4678 or traditional (lumped) factor of safety (FOS) approach. For traditional FOS, the minimum FOS given in Table 6.7.4(a) shall be satisfied.

Table 6.7.4(a) – Minimum FOS

Mode of failure	Minimum FOS
Sliding	2.0
Overturning	2.0
Bearing	2.5
Global	1.5

Minimum wall dimensions shall be in accordance with Table 6.7.4(b) below.

Table 6.7.4(b) – Geometric details of wall

Effective design wall height, H (m)	Minimum wall base dimensions, B (m)	Minimum width of top of wall, D (m)
1.5	1.4	0.5
2.0	1.5	0.5
2.5	B/H = 0.7	0.75
3.0	B/H = 0.7	1.0

Notes:

1. For the definition of effective design wall height, 'H', refer the typical wall section (Figure 6.7.2).
2. A minimum foundation embedment of 0.5 m of the boulder wall into natural ground shall be provided.
3. Front slope of wall shall not be steeper than 4 Vertical to 1 Horizontal.

The stability of the wall shall be checked against the following criteria, in addition to other requirements that may be warranted. Wall friction must be ignored in the analysis:

- sliding: effective cohesion to be assumed zero, both total and effective stress calculations for sliding to be carried out, passive resistance in front of the wall shall be ignored
- overturning: shall meet the requirements of the middle third rule of structural mechanics
- bearing failure: total stress calculations shall be carried out, and
- global failure: both total and effective stress calculations shall be carried out.

The design friction angle of rockfill / backfill shall be limited to a maximum of 36°.

Design report(s) certified by an experienced RPEQ Civil Engineer, who is competent in the field of geotechnical engineering, and all relevant drawings shall be included in the design documentation.

Design report(s) shall include the following as a minimum:

- source of rockfill (if known) and methodology for production control
- properties of the rockfill
- properties of the backfill material
- foundation conditions
- wall dimensions, and
- design calculations.

The drawings shall include the following details as a minimum:

- plan showing the location of the wall along with adjoining structures
- wall elevation (vertical joints must be staggered)
- wall cross-sections showing the width of the courses at every change of wall height greater than 0.5 m
- drainage details: provision of a full height 300 mm minimum thickness granular drainage blanket (see Clause 53.2.2 of MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*) behind the boulder wall – continuous geosynthetic filter fabric complying with MRTS27 *Geotextiles (Separation and Filtration)* shall be provided at the drainage blanket / backfill interface, and
- the allowable bearing pressures to be stipulated.

6.7.5 Construction

Construction requirements shall conform to Clause 53 of MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections*. Certification of design and construction shall be as per Section 6.8.

6.8 Certification of retaining structures

The design documentation shall include a certificate from the RPEQ Designer which confirms that the design:

- adequately allows for the site conditions, applied loadings, and relevant material properties for all components of the design, and
- ensures the structural integrity and serviceability of the wall for the nominated design life.

The Design Documentation shall include the following, in addition to the Design Certificate:

- design calculations
- construction drawings
- construction specifications, including wall construction sequence
- Supplementary Specifications for any specific requirements for ground and/or foundation improvement or construction methodology, and
- arrangements for monitoring the performance of the wall over the nominated period.

The design documentation shall be submitted to the Administrator who shall forward to the department's Geotechnical and Structural Sections for review. Until the design is acceptable to the department, construction of the wall shall not be commenced.

At the end of construction as part of the constructed drawings, the Contractor shall submit to the Administrator, a report certified by the Contractor's RPEQ Civil Engineer with competency in the field of geotechnical engineering (or other suitably qualified RPEQ Engineer) who supervised the construction of the wall. The report shall demonstrate that the wall has been duly constructed as per the relevant departmental Technical Specifications, Australian Standards or codes, other relevant international standards mentioned in this section and this document while conforming to all the design requirements.

7 Remediation of existing slopes and embankments

The minimum design life for embankments and cut slopes in new infrastructure projects shall be 100 years as indicated in Section 1 of this document. Their design and performance requirements shall be as per Section 3 and 4 of this document; however, for the remediation works on existing slopes and fill embankments (excluding remedial works of any new infrastructures less than 10 years old since the completion of the construction), a reduced design life and a lower FOS may be considered in consultation with and at the sole discretion of the department's Geotechnical Section as outlined in the following subsections.

7.1 Design life for remediation works

The required design life for remediation of existing slopes or embankments generally depends on the remnant design life and the future upgrades of the associated road network, available funding, and/or the District's requirements. If the design life shorter than 100 years is justifiable, lower durability requirements of the structural and the earth reinforcement elements in remediation works can be considered; for example, the level of corrosion protection requirements for soil nails and rock dowels specified in MRTS03 *Drainage Structures, Retaining Structures and Embankment Slope Protections* may be relaxed if a reduced design life is justifiable. Any such reduction in design life and the level of corrosion protection shall be agreed for each design, with the relevant Transport and Main Roads district in consultation with the department's Geotechnical Section, prior to adopting it in the design.

7.2 FOS to be used in the remediation design

Depending on the level of risk and associated consequences posed to road users, properties, public utilities, buildings, and so on, a lower FOS may be adopted in the geotechnical design for remedial works of failed slopes.

Table 7.2 provides guidance on the minimum two-dimensional FOS required in the design of remedial works weighted against consequential effects. This table and the procedure outlined herein are generally in accordance with Transport for NSW (TfNSW) Roads and Maritime Service Technical Direction (2018) for Geotechnical Design for Remediation on Existing Slopes and Embankments.

The FOS to be adopted by the design consultants or contractors shall be agreed and accepted by the department's Geotechnical Section prior to the commencement of designs.

Table 7.2 – Consequence class and minimum FOS for remediation design

Consequence Class	C1	C2	C3	C4	C5
Long-term FOS	1.50	1.40	1.30	1.25	1.25
Short-term FOS	1.25	1.25	1.20	1.20	1.20

Notes: Definition of consequence class can be found in TfNSW, Roads and Maritime Services, *Guide to Slope Risk Analysis* (2014).

In the absence of any verified groundwater observations and assessments, the porewater pressure coefficient (Ru) to be adopted in the assessments shall not be less than 0.15 for cut slopes even with appropriate drainage systems in place.

Geotechnical designs of retaining structures, that may be required for the stabilising of failed slopes or embankments, shall be carried out in accordance with the relevant sections in this document except that the design life shall be determined in accordance with Section 7.1.

7.3 Catch probability of rockfall catch fence

The bounce height and Maximum Energy Level (MEL) containments of rockfall catch fence shall be 100%.

Under special circumstances and depending on the level of risks and associated consequences posted to road users, properties, buildings and so on, a lower bounce height and MEL containment probability not less than 90% may be considered. In such a case, residual risks shall be mitigated by other appropriate mitigation measures. Such departures shall be submitted to the department's Geotechnical Section in writing through the Administrator for review and acceptance.

8 High risk temporary work design

Temporary works, that typically defined in Clause 5.1.1.1 of BS 5975, are considered high risk when they are undertaken adjacent to live traffic or have the potential to cause adverse impacts on existing road infrastructure, associated utilities and / or safety of the road users and the construction personnel, for example, temporary retaining structures or excavations adjacent to live traffic. Any temporary works required for the safe operation of cranes, piling rigs or drilling rigs shall also be considered as high-risk.

Temporary works designs, including high-risk temporary works, shall be carried out in accordance with Clause 13 of BS 5975. The design of all geotechnical components related to the temporary works shall be undertaken by adequately experienced engineers on similar works and such temporary works shall be certified by an experienced RPEQ Civil Engineer, who is competent in the field of geotechnical engineering.

Temporary works that are later incorporated into permanent works shall meet the following requirements:

- temporary works that are intended to be incorporated into permanent works shall be designed to meet the requirements of the permanent as well as temporary works, and
- temporary works that are incorporated into permanent works without initial intent to do so shall be verified by the permanent works designer as having met the requirements of the permanent as well as temporary works.

All high-risk temporary works shall be instrumented and monitored unless agreed otherwise in writing with the Administrator.

Serviceability requirements of adjacent roads (e.g. rideability requirements in accordance with Table 3.3), services and PUP shall also be assessed, and measures shall be proposed to mitigate any adverse impacts.

Notwithstanding the rest of low-risk temporary works, the high-risk temporary works designs require specialist input from professional engineers experienced in similar high-risk temporary work designs, with familiarity of departmental requirements, to achieve acceptable safety and economic outcomes.

The design documents that are submitted for departmental review shall be standalone design reports and drawings that clearly outline the problem(s) and solution(s) including construction sequence(s) as required by Section 1.

> Since temporary work designs often have a restricted scope, contents other than Section 8 of this document may not necessarily be applicable to their design. The design life is often very short, durability requirements are less stringent, and the required safety margins could be lower compared to permanent work designs due to the different risk profiles. Additionally, the design loadings and their critical combinations for temporary works typically differ from those for permanent works.

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