Engineering

Technical Guideline

TG 0641 - General Technical Information for Geotechnical Design - Earth Dams

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Only the current revision of this Guideline should be used which is available for download from the SA Water website.

Significant/Major Changes Incorporated in This Edition

This is the first issue of this Technical Guideline under the new numbering format. The original version of the document was last published in 2007 with the name of General Technical Information for Geotechnical Design Part B – Earth Dam Design (TG 10b). A full version history of this document is given in Document Controls. The major changes in this revision are listed in the following table:

Section No. in TG 0641	Section No. in TG 10b	Changes
TG0641 – 3 to 7	N/A	Sections 3 to 7 are added to this TG.
TG0641 – 8	TG 10b – 2	Major Revision
TG0641 – 9	N/A	Section 9 is added to this TG.
N/A	TG 10b – 3	Superseded

Document Controls

Revision History

Revision	Date	Author	Comments
0	1/11/2004	Ed Collingham	First Issue of TG 10b
1	10/1/2007		Nil
2	08/10/2020	Moji Kan	Major Revision, Reformatting to TG 0641

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1 Introduction

SA Water is responsible for operation and maintenance of an extensive amount of engineering infrastructure.

This guideline has been developed to assist in the design, maintenance, construction, and management of this infrastructure.

1.1 Purpose

The purpose of this guideline is to detail minimum requirements to ensure that assets covered by the scope of this guideline are constructed and maintained to consistent standards and attain the required asset life.

1.2 Glossary

The following glossary items are used in this document:

Term	Description
ANCOLD	Australian National Committee on Large Dams
CLSM	Controlled Low Strength Materials
DPTI	Department of Planning and Transport Infrastructure, now DIT
EBS	Earth Bank Storage
HDD	Horizontal Directional Drilling
GRP	Glass Reinforced Pipe
MSCL	Mild Steel Cement Lined
NAIS	Northern Adelaide Irrigation Scheme
PAR	Population at Risk
PLL	Potential Loss of Life
SA Water	South Australian Water Corporation
TG	SA Water Technical Guideline
TS	SA Water Technical Standard
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

1.3 References

1.3.1 Australian and International

The following table identifies Australian and International standards and other similar documents referenced in this document:

Number	Title
ANCOLD Guidelines	The suite of ANCOLD Guidelines, applicable to the design and safety assessments of earth and embankment dams
AS 1141.23	Methods for sampling and testing aggregates - Los Angeles value
AS 1289.3.1.2	Methods of testing soils for engineering purposes - Soil classification tests - Determination of the liquid limit of a soil - One-point Casagrande method (subsidiary method)
AS 1289.3.2.1	Methods of testing soils for engineering purposes - Soil classification tests - Determination of the plastic limit of a soil - Standard method
AS 1289.3.3.1	Methods of testing soils for engineering purposes - Soil classification tests - Calculation of the plasticity index of a soil
AS 1289.3.4.1	Methods of testing soils for engineering purposes - Soil classification tests - Determination of the linear shrinkage of a soil - Standard method
AS 1289.3.6.1	Methods of testing soils for engineering purposes - Soil classification tests - Determination of the particle size distribution of a soil - Standard method of analysis by sieving
AS 1289.3.6.3	Methods of testing soils for engineering purposes - Soil classification tests - Determination of the particle size distribution of a soil - Standard method of fine analysis using an hydrometer
AS 1289.3.8.1	Methods of testing soils for engineering purposes - Soil classification tests - Dispersion - Determination of Emerson class number of a soil
AS 1289.5.2.1	Methods of testing soils for engineering purposes - Soil compaction and density tests - Determination of the dry density/moisture content relation of a soil using modified compactive effort
AS 1289.6.4.1	Methods of testing soils for engineering purposes - Soil strength and consolidation tests - Determination of compressive strength of a soil - Compressive strength of a specimen tested in undrained triaxial compression without measurement of pore water pressure
AS 1289.6.7.3	Methods of testing soils for engineering purposes - Soil strength and consolidation tests - Determination of permeability of soil - Constant head method using a flexible wall permeameter
DS 13-2	USBR, Design Standards No. 13, Embankment Dams, Chapter 2: Embankment Design Phase 4 (Final), 2012
DS 13-6	USBR, Design Standards No. 13, Embankment Dams, Chapter 6: Freeboard, 2012
DS 13-7	USBR, Design Standards No. 13, Embankment Dams, Chapter 7: Riprap Slope Protection, 2014
Fell et al. (2015)	Fell, R., MacGregor, P., Stapledon, D., Bell G., and Foster, M., 2014. Geotechnical Engineering of Dams, 2nd Edition, CRC Press/ Balkema
USBR (1990)	USBR, Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low Level Outlet Works, Technical Memorandum No. 3, 1990

1.3.2 SA Water Documents

The following table identifies the SA Water standards and other similar documents referenced in this document:

Number	Title
TS 0460	Technical Standard for Liners and Floating covers for Earth Bank Storages
TS 4	Packing Sand for Pipe Laying and Trench Fill
TS 0522	Allowable Pipe Size, Class and Materials for Reticulation Water Mains

1.4 Definitions

The following definitions are applicable to this document:

Term	Description
SA Water's Representative	The SA Water representative with delegated authority under a Contract or engagement, including (as applicable): • Superintendent's Representative (e.g. AS 4300 & AS 2124 etc.) • SA Water Project Manager • SA Water nominated contact person
Responsible Discipline Lead	The engineering discipline expert responsible for TG 0641 defined on page 3 (via SA Water's Representative)

2 Scope

The scope of this document is to provide guidelines on geotechnical aspects of the design of new earth dams, or the safety review, renewal, or upgrade of existing earth dams for SA Water infrastructure.

3 Introduction

SA Water owns and operates number of existing earth dams and earth bank storages (EBS). The existing earth dams are normally subject to periodic inspections and safety upgrades as requirements of ANCOLD Guidelines recommend. Among the existing large dams, the Hope Valley Dam for instance is a zoned earthfill dam with puddle clay core (see Figure 1).

SA Water also builds new earth dams and EBSs. Most of the new EBSs are lined with geosynthetic liners, however the existing EBSs may have concrete panel liners, clay liners, or internal low permeability zones with erosion protection measures such as riprap on the outside face of the dam. The following list shows the different types of EBSs in SA Water's portfolio:

- Earth embankments, e.g. see Figure 2 and Figure 3.
- Geomembrane lined and covered, e.g. see Figure 4.
- Geomembrane lined with no cover, e.g. see Figure 5.
- Concrete lined, e.g. see Figure 6 and Figure 7.
- Concrete lined and fixed roof, e.g. see Figure 8.

This Technical Guideline sets out the minimum geotechnical requirements of upgrade works for earth dams and EBSs, as well as the design requirements of the new earth dams. The content of the present Technical Guideline is applicable to all types of earth dams and EBSs, regardless of the liner type or the material of impermeable zone. Figure 9 shows the possible types of earth dams and EBSs in SA Water's portfolio.

SA Water has a dedicated Technical Standard for Liners and Floating Covers for Earth Bank Storages – TS 0460.

Figure 1: Hope Valley Dam, zoned earthfill dam with puddle clay core

Figure 2: Earth embankment (Millbrook Tank Site)

Figure 3: Earth embankment with riprap protection (Aldinga WWTP, Lagoon 3)

Figure 4: Geomembrane lined and covered storage (Upper Paskeville)

Figure 5: Geomembrane lined storage with no cover (NAIS)

Figure 6: EBS with concrete liner (Lincoln Gap)

Figure 7: EBS with concrete liner (Mannum WWTP)

Figure 8: EBS with concrete liner and fixed roof (Hanson)

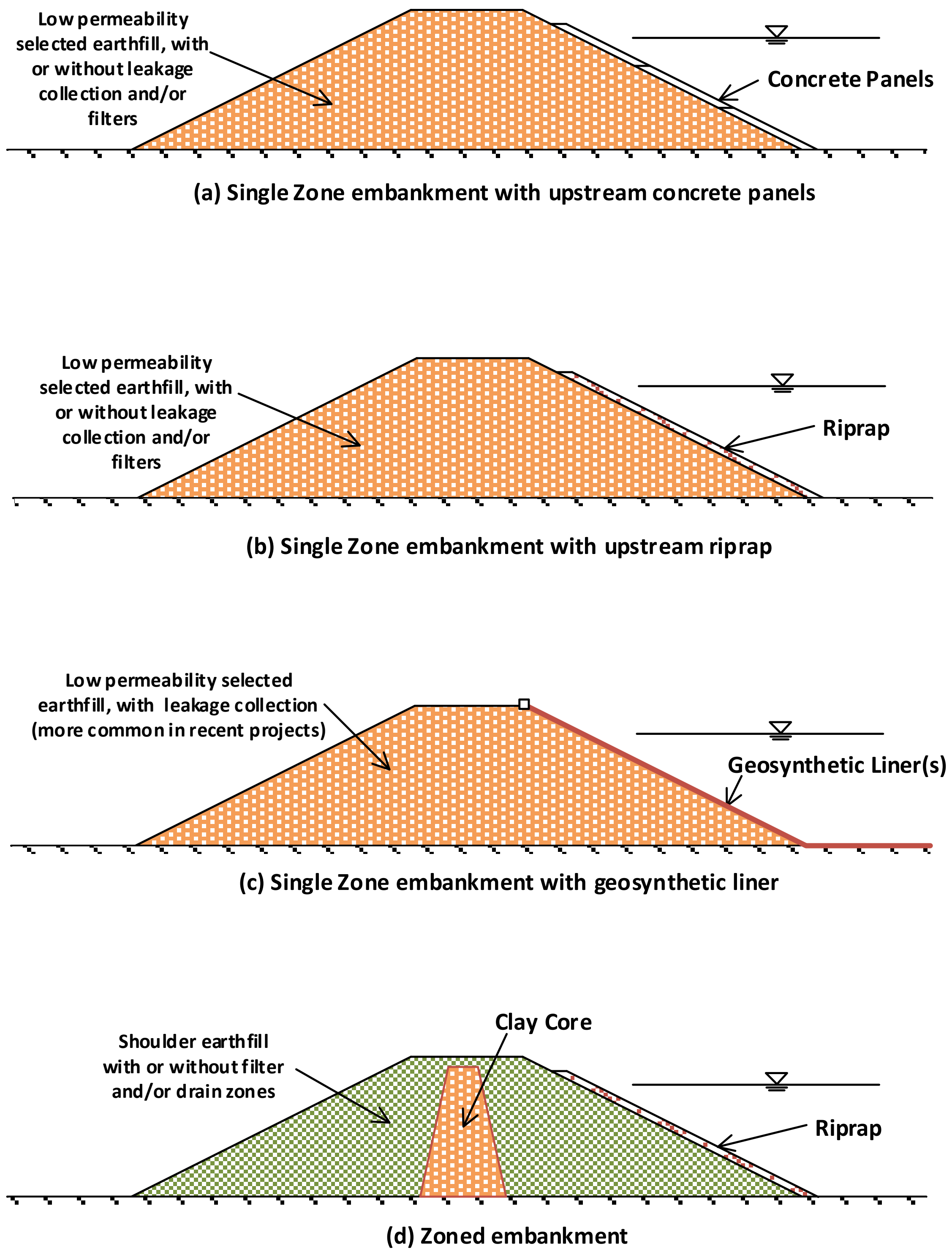

Figure 9: Possible types of earth dams in SA Water portfolio

4 Earthworks

4.1 Single zone embankments – upstream lined storages

Where a new single zone embankment (with no internal filters) is to be designed and constructed as part of a dam or earth bank storage project, with an impermeable upstream barrier such as a geosynthetic or concrete liner, the performance requirements in the following sections should be adopted.

4.1.1 Embankment materials

Where single zoned embankments are constructed for new small dams or earth bank storages with upstream barriers, the materials of the embankment should meet the following performance requirements:

Table 1: Performance requirements for single zone embankment materials

Parameter	Test	Acceptance Limit
Permeability	AS 1289.6.7.3	≤ 1 x 10^{-9} m/sec
Grading	AS 1289.3.6.3 AS 1289.3.6.1	≥ 30% passing the 75 μ sieve ≤ 20 % passing the 19 mm sieve
Emerson Class	AS 1289.3.8.1	≥ Class 4 – (non-dispersive)
Atterberg Limits	AS 1289.3.1.2, 3.2.1, 3.3.1, 3.4.1	Low to medium plasticity CLAY
Remoulded Undrained Shear Strength	AS 1289.6.4.1	≥ 75 kN/m^2

Note: The permeability and shear strength testing is to be undertaken on remoulded samples at 98% standard maximum dry density.

4.1.2 Embankment slope angles (internal and external)

Where embankments are constructed on a relatively flat site and with adequate space, the internal and external slopes should range between 1V:4H and 1V:3H.

Where site constraints make it necessary to adopt steeper slopes than those above, SiD issues associated with the adoption of steeper slopes should be considered, addressed, documented, and agreed with SA Water.

4.1.3 Embankment design

The embankments should be designed for the following load cases using slope stability software, with both upstream and downstream slopes to be assessed.

Table 2: Load cases for embankment design, singe zoned lined dams

Load Case	Factor of Safety Requirements*
Short term total stress conditions	≥ 1.5
Long term effective stress conditions	≥ 1.5
Rapid Drawdown	≥ 1.3
Seismic Loading	≥ 1.0
Plant / Construction Loading	≥ 1.5

* The factors of safety values adopted in the table above are for deep seated failures. Lower factors of safety for shallow failures affecting only the embankment face should be considered by the designer. Where crane lifts are required specific assessments should be undertaken.

4.1.4 Embankment construction

The compaction requirements for embankment material are summarised in the following table.

Table 3: Compaction requirements for embankment materials

Property	Acceptable Limits	Note
Maximum Dry Density	98%	Standard Maximum Dry Density, SMDD ratio of 98%. The material should be compacted with a vibrating smooth drum roller or other approved equipment until the required density is achieved.
Optimum Moisture Content	+/- 2% OMC	The optimum moisture content of the material should be the moisture content that is required to achieve the peak dry density when tested in accordance with the method given in AS 1289.5.2.1.
Max Layer Thickness	250 mm	The embankment material should be compacted in uniform horizontal lifts.
Unsuitable materials	hard clay lumps, organic matter, and industrial by-products	Where these materials are encountered, they should be broken down or removed before being transported to the embankment.

Site compaction trials should be undertaken on the proposed embankment material prior to the placement of any embankment material. The purpose of the trial will be to establish the suitability or otherwise of the compaction equipment proposed, the number of passes required and to determine the optimum layer thickness.

4.1.5 Embankment foundation preparation

The following steps should be followed when preparing the foundation underneath a proposed embankment:

- Pockets of weak or otherwise unsuitable material should be removed below the general foundation level or as directed by the Superintendent,
- The foundation surface, immediately prior to receiving the embankment material, should have all water removed from the depressions and the top 150 mm of foundation material should be sufficiently moistened and compacted,
- Where the foundation for the embankment material is the concrete encasement of the pipework, the surface for the fill is to be placed on the concrete, contact material should

be prepared by removing any loose and unbonded material that will prevent the bonding of the contact material with the foundation.

4.1.6 Earthworks issues in renewal of existing dams

There are a number of earthworks issues that should be anticipated when the works are primarily to reline an existing EBS, these issues are summarised below:

- Sites have shown cracking within the base and side of the EBS due to shrinkage of the clay embankment fill. Prior to placing the new liner, any cracking should be remediated to provide a uniform surface for the acceptance of the new liner,
- There may be weak / soft material within the base and sides of the EBS, where drainage has been ineffective or absent. This will reduce the trafficability of the material and where required the weak / soft material should be removed and replaced with competent material, to allow plant / machines to access the EBS,
- Careful consideration should be given to the removal of the existing liner, as exposure of the base and side of the EBS to the elements can have significant impacts on its integrity. Uncontrolled surface run-off can cause significant damage to the embankments which should be remediated prior to the placement of the new liner.

4.1.7 Erosion protection

The upstream face of the single zoned – lined earth dams or EBSs normally are protected by a liner, either a concrete liner or geosynthetic liner. If the upstream face of the storage is lined with clay or any other type of liner which is prone to erosion due to wave actions in the reservoir, the face of the dam should be protected with riprap or other types of erosion resistance materials. The design of the riprap protection should comply with requirements of relevant standards and guidelines. It is recommended to adopt the USBR recommendations for design of riprap against wave runup and wind setup actions, as set out in DS 13-7 and Fell et al. (2015).

It is essential that the downstream face of the earth dams and EBSs be protected from erosion. The minimum requirement for erosion protection should be a layer of seeded topsoil nominally 150 mm thick. The grass should be of an approved seed mixture and spread at a density not less than 10g/m^2. Maintenance of the topsoil layer may be required prior to the establishment of the grass cover and root system.

Where higher flows or drainage outlets are located on the downstream face then more robust erosion protection should be required, such as riprap or a reno-mattress type structure.

The Contractor needs to plan and carry out the work to avoid erosion, contamination and sedimentation of the site, surrounding areas, and drainage systems. The Contractor should maintain erosion control measures during the course of the work.

4.2 Zoned embankments

Where a new zoned embankment is to be designed and constructed as part of a dam or EBS project, the best dams engineering practice needs to be followed in selection of the earthfill materials. SA Water considers the ANCOLD Guidelines in general, as well as Fell et al. (2015) in particular, as the best current practice for design of zoned earthfill dams. A qualified geotechnical Dams Engineer should undertake the design works and oversee the construction. Any deviations from Fell et al. (2015) should be shared in advance with SA Water to grant dispensations or to agree on prior to the design works proceeding.

The embankments should be designed using slope stability software, with both upstream and downstream slopes to be assessed. Stability analyses should be performed using reputable software packages such as ‘Slope/W’ software, or similar, as agreed with SA Water. Such software should use a 2D limit equilibrium theory as a minimum to compute the factor of safety of slopes. As a minimum, the Spencer (1967) method should be adopted for all slope stability analyses as recommended by Fell et al. (2015), however the resulting factors of safety might also be checked against the likes of the Morgenstern and Price (1965) and Janbu (1968) methods, to understand the variance in the achieved factors of safety.

Consideration should be given to both effective and total stress analyses, using drained and undrained shear strength materials parameters, as suits for the adopted materials and the corresponding load case for which the analysis is being undertaken. A series of sensitivity analyses might be required to understand the effects of assumptions and uncertainties in the results of the analyses.

4.3 Excavation and backfilling requirements

If any excavations are required for pipe laying purposes within or in close proximity to the earthfill dams or the EBSs, the requirements of the following sections should be followed.

If the excavation is not for the pipe-laying, and is for other purposes such as laying a filter or drain zone for the dam safety upgrade, providing access to the dam, or similar, then a competent Dams Engineer should be consulted for proper design of the excavation and the backfill of the trench. As the minimum requirement, the trench backfill should have properties that are compatible with the adjacent embankment. Trenching through the embankment and underlying foundation should be battered and not benched. Battered side slopes will promote acting positive pressures on the contact face and will lessen the potential for differential settlement, vertical cracking and hydraulic fracture or piping.

Any alterations to the embankment of the earthfill dams or EBSs, either for temporary or permanent works, is not permitted unless a design document is prepared by a competent Dams Engineer and submitted to SA Water Engineering for verification and the endorsement is granted.

4.3.1 Pipe trench backfill within embankment footprint

The embankment footprint consists of the area on the plan that the embankment cut and fill occupies (after stripping the unsuitable materials), plus an influence zone which should be determined using a horizontal distance outside of the embankment toe on either side which should be the greater of the following items:

- two times the apparent height of the embankment,
- the length of cut section (shown as “D” in following figure) on the upstream side,
- 5 m on the downstream side.

Figure 10 below schematically shows the extent of embankment footprint for a typical cut/fill EBS for pipe embedment purposes.

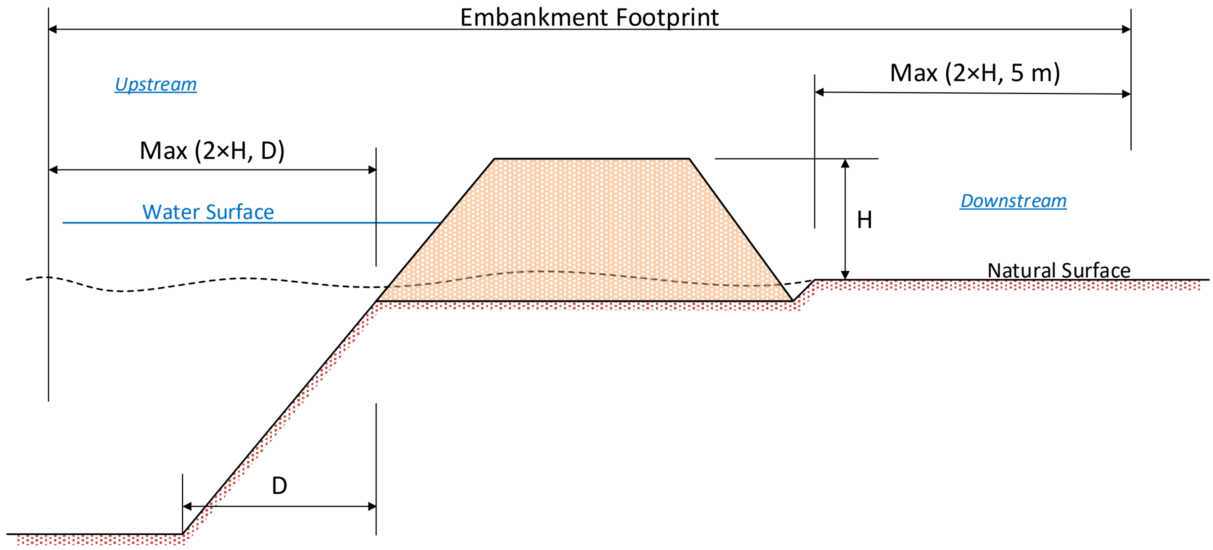

Figure 10: Extent of embankment footprint for a typical cut/fill EBS

Within the embankment footprint as determined above, the backfill of the material for embedment of the pipe should be selected using a risk-based approach.

Based on the risk profile of the dam, the following trench backfill should be provided within the embankment footprint:

For Low Risk: Backfill with Cement stabilised sand.

The sand material should be equivalent to SA Water Technical Standard TS4. The sand should be stabilised with 5% w/w GP cement.

Cement stabilised sand should be supplied and placed in accordance with DPTI Specification for Controlled Low Strength Material: Part R09 Supply of CLSM. The construction method should be suitable to prevent floatation of pipe work during backfilling.

For Medium Risk: Backfill with Reinforced Concrete.

The grade of concrete and the required reinforcement should be designed based on applied construction loads (e.g. compaction impacts), internal and external pressure from the pipe, and from the surrounding soil, assuming no contribution of strength from the pipe. As the minimum requirement, the concrete should be at least 150 mm thick with grade S15 or higher.

If the pipeline is located on the floor of the foundation excavation, the side slopes of the concrete embedment should be no steeper than 1H:10V to encourage high contact pressures against the concrete surface. If the pipeline is located in a trench excavated below the floor of the foundation excavation, the concrete embedment should be poured to the sides and top of the trench, making sure that a proper bond between the concrete and trench walls is achieved.

For High Risk: Backfill with Reinforced Concrete, plus filter collar in the last 1/3 length of the encasement length at the downstream side of the bank.

The design of the reinforced concrete for High Risk should follow the same principles as those outlined for the Medium Risk.

The filter collar consists of a zone of filter material (usually sand) that completely surrounds a specified length of conduit and should be designed by an experienced Dams Engineer to satisfy the filtration criteria based on particle size distribution of the base soil in accordance with Fell et al. (2015). The minimum thickness of the filter collar is 450 mm perpendicular to the pipe; the actual thickness depends upon design requirements.

In situations that placing the filter collar against the concrete encasement is not practical (e.g. due to the pipe trench being located below the foundation excavation floor, or in renewal of the existing EBSs), a filter diaphragm can be used instead of a filter collar. The filter diaphragm should be designed by an experienced Dams Engineer, as a minimum the diaphragm should be 1m thick, extending 1.5 W beneath the embedment zone, 3 W above and each side of the embedment zone, and extending 0.6 m beyond the excavated trench width, where W is the width of the pipe embedment zone (concrete) in plan. A filtered drainage outlet from the bottom of the diaphragm should be provided to the EBS toe, leading to a downstream collection chamber.

The risk profile can be best determined based on consequence category of the dam failure, determined in accordance with ANCOLD guidelines, based on population at risk (PAR) or incremental potential loss of life (PLL) and considering the severity of damage and loss. The following table can be used to correlate the ANCOLD consequence categories with risk profile of the EBS for trench backfill purposes.

Table 4: Risk profile in accordance with ANCOLD Guidelines

Risk profile of the EBS for selection of trench backfill	ANCOLD consequence categories of dambreak
Low	Very Low and Low
Medium	Significant
High	High A, High B, High C, Extreme

A simple assessment of the risk profile may also be achieved using a high-level estimate of the potentials and consequences, using the following table.

Table 5: Risk assessment profile (simplified method)

Risk profile of the EBS for selection of trench backfill	Potential loss of human life	Economic, environmental, lifeline losses
Low	None	Low, generally limited to owner
Medium	None	Yes
High	Probable	Yes, but is not necessary to fall in this risk profile

Around and on top of the pipe embedment zone, the rest of the trench backfill should have properties that are compatible with the adjacent embankment. Ideally, the earth material adjacent to the pipe embedment in a low permeable zone of the embankment should be reasonably well graded, have a maximum particle size no greater than 13 mm, including earth clods, a minimum of 50 percent by weight passing a No. 200 sieve, and a plasticity index between 10 and 30 percent. The water content of the material in this zone should be between 1 percent and 3 percent wet of optimum.

As per general requirements of the trenching through the bank of the earth dams, the pipe-laying trench through the dams and underlying foundation should also be battered and not benched. Figure 11 below show an example of battered slope in pipeline trench within an embankment footprint. Other design requirements of the trench backfill are summarised below:

- Where desiccation or saturation of the exposed excavation occurs, it is essential that any desiccation cracks or low strength saturated materials are removed over their full extent,
- Prior to placing embankment material adjacent to the installed pipework, the encased concrete must have attained its design strength,

- Earthwork materials immediately adjacent to the reinstated pipework should be compacted, so that no layers of material with higher permeability than in the adjacent material extend in an upstream and downstream direction along the pipe,
- The elevation of the earthwork materials should be maintained at approximately the same level on both sides of the conduit during backfilling. This will help to prevent lateral movement of the pipe caused by unequal compaction energy applied to the side of the pipe,
- High trafficked areas need to be ripped prior to placement of earthworks materials. This will aid the removal of tension cracks and moistening of the surface before placing subsequent lifts, to prevent smooth surfaces between lifts,
- Compaction adjacent and over the encased pipework must proceed with caution as to not damage the pipework. Hand or remote operated compaction (tampers or wacker plates) may be required for the initial layers to cover the pipework.

Unless an advanced numerical simulation shows otherwise, excavated slopes in soil for pipe trenches should be no steeper than 1 vertical to 2 horizontal to facilitate adequate compaction and bonding of backfill with the sides of the excavation. This recommendation is appropriate for favourable soil properties. Flatter side slopes should be used for less favourable conditions. Excavation slopes of 3H:1V to 4H:1V are commonly recommended for unfavourable situations.

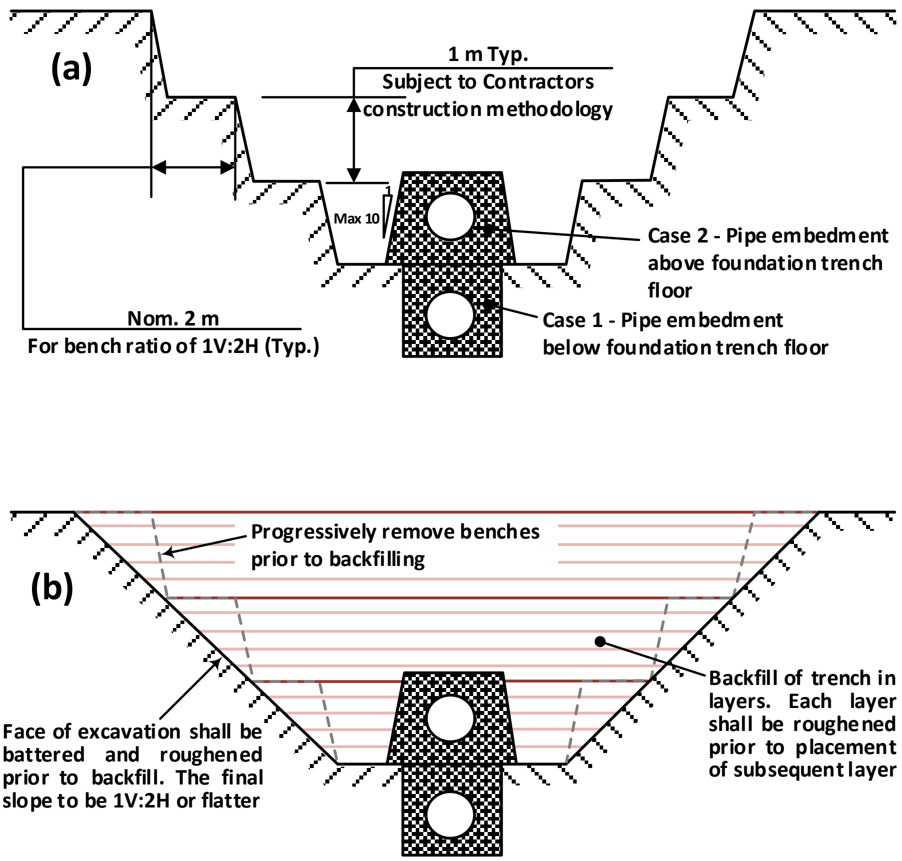

Figure 11: An example of (a) benched embankment excavation for pipe penetration, and (b) backfilling around and on top of the pipe embedment zone with progressive battering

The length of pipe laid within the embankment footprint should be minimized, e.g. by setting the pipe alignments perpendicular to the embankment axis as much as practical. No bends, valves, or abrupt changes of pipe type or pipe diameter should be allowed within the embankment footprint.

In all embankments, hoses or loose corrugated pipes without embedment should not be laid, used, or left active within the embankment footprint, as they pose an unacceptable risk of burst, uncontrolled release of water, erosion, and potentially failure to the embankment.

Pipes laid in trenches within the embankment footprint should be either welded or have solid joints. Use of pipes which may require non-restraint joints (e.g. GRP with rubber joints) are not permitted.

4.3.2 Pipe Trench backfill outside embankment

The material used for trench backfill outside the embankment footprint should be coarse, free flowing pit or beach sand, equivalent to SA Water Technical Standard TS4.

The material should be clean with 100% passing a 4.75 mm sieve and not greater than 5% passing a 0.075 mm sieve, and such that it can be satisfactorily and economically compacted in the dry state. Note that sand backfilling can only be used on trenches outside the embankment footprint, see Figure 10 for the definition of embankment footprint.

4.3.3 Shoring of excavations

SA Work Health and Safety Act 2012 has specific requirements in respect to excavations exceeding 1.5 metres in depth and which permit the entry of a person. The Contractor should ensure that all ground support systems are removed as the excavation is backfilled, in a manner which should prevent damage to any persons, the main and/or any other adjacent structures, unless approval is obtained from the Principal for the support systems to remain in place during backfilling (i.e. for the support systems to be “lost”).

4.3.4 Shoring of trenches and pits

The Contractor should supply, put in place and maintain such shoring in accordance with the relevant statutory requirements and as may be required to support the wall of the excavation and to prevent any movement which can in any way injure personnel or endanger any adjacent pavements, buildings, conduits, or other structures. If the Principal considers that sufficient or proper shoring has not been provided, the Principal may order additional shoring put in at the expense of the Contractor and the compliance with such orders should not release the Contractor from its responsibility for the sufficiency of such shoring.

4.3.5 Micro-tunnelling and Horizontal Directional Drilling

Micro tunnelling and horizontal directional drilling (HDD) are not suitable for use through embankment dams. Both methods have difficulties with obtaining a watertight seal along the conduit and can potentially disturb the embankment during installation. It is recommended that they are not installed through the body of embankment dams.

Micro-tunnelling and HDD are alternative methods that are sometimes used to install pipes below embankments, although this type of installation is also discouraged for earthfill dams due to potential risks to the stability of the embankment should there be any leaks or burst in the pipe in the foundation of the embankment.

5 Access road requirements

5.1 General

Unless stated otherwise instructed by SA Water, all new earth dams and EBSs should include an access road design suitable for SA Water's vehicular equipment, construction vehicles and any permitted vehicles specifically identified by SA Water. In upgrades of existing dams and EBSs, the modification of existing access roads or tracks including embankment crest roads should be included in the design documentation of the upgrade works.

Depending on the site-specific requirements, the access road should be granular pavements sealed with asphalt or equivalent bituminous surfacing. The design life of pavement should at least be 25 years.

5.2 Typical road section

The access road and embankment crest tracks, as a minimum, should be provided with a minimum carriageway width of 4 m suitable for single lane roadway, clear of any roadside kerbs, drains, and signage. Wider roads may be required for larger dams, e.g. in accordance with USBR DS 13-2. The road base materials should be conforming to the requirements of AUSTROADS and as a minimum should be equivalent to DPTI PM1/20QG base-coarse with the properties included in the table below.

A minimum cross-fall of 2% should be provided to facilitate free draining of the road surface.

Figure 12 shows the example typical road cross-section of the embankment crest road.

Table 6: DPTI PM1/20QG base-coarse road base materials properties

Test Procedure	**Manufacturing Tolerance (Grading based)**	
Particle Size Distribution	Sieve Size (mm)	Percentage Passing
	26.5	100
	19	95 – 100
	13.2	77 – 93
	9.5	63 – 83
	4.75	44 – 64
	2.36	29 – 49
	0.425	13 – 23
	0.075	5 – 11
AS 1289.3.1.2	Liquid Limit	Maximum 25%
AS 1289.3.3.1	Plasticity Index	Minimum 1%, Maximum 6%
AS 1289.3.4.1	Linear Shrinkage	Maximum 3%
AS 1141.23	LA Abrasion Grading 'B'	Maximum 30%

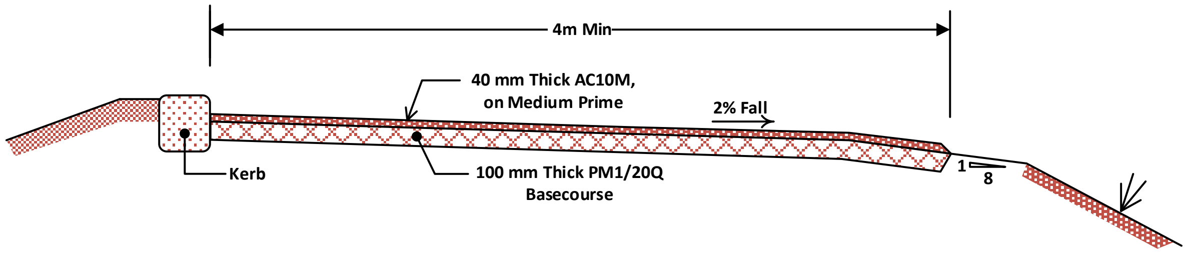

Figure 12: Example embankment crest road section

5.3 Design for vehicles

All vehicles expected to access the dam site should be able to safely travel without damaging the site infrastructure, such as pavement, kerbs, and similar roadside furniture. The design vehicle to be considered for the access road pavements should be in accordance with AUSTROADS and as a minimum includes the following:

- Passenger vehicles
- Service Vehicles
- Single unit truck
- Construction equipment

The access of prime movers or semi-trailers, including B-doubles, should be confirmed with SA Water prior to commencing design documentation and should form part of the design basis report.

5.4 Requirements for asphalt pavements

The supply and construction of asphalt pavements should comply in strict accordance with the requirements of the DPTI Specification Part R27 and Part R28, respectively including all future amendments current at the time of project delivery.

6 Drainage requirements

6.1 Underdrain system requirements

In all lined storages, an underdrain system is required to drain water from under the liner due to liner leaks or groundwater intrusion. The underdrain system should be designed in segments that are independently drained in order to isolate potential leaks in the liner system.

The design flow rate for the underdrain system is to be assessed by the designer during design development and approved by the Principal. Factors influencing design flow rates for the underdrain system include:

- Liner design
- Liner material
- Local hydrogeology
- Geotechnical conditions

Each segment's underdrain system should be self-contained and may consist of slotted pipes connected to a discrete underdrain delivery pipe to a pump system delivery pipe containing a submersible withdrawable pump complete with discharge hose, hose couplings and submersible power cable. The submersible pumps should be accessible from access track level via hauling up through a duct or similar arrangement.

Each submersible pump unit should be able to operate automatically via sensor control or other means of mechanism to discharge water in the pipe.

Discharge should be to a sampling pit at access track level through a duct under the liner. At a minimum, the discharge line from each segment should have fittings suitable to measure and sample the flow at the access track level. The sampling pit should drain to sewer if available. If a suitable sewer connection is not available, discharge to the stormwater system may be appropriate as approved by the Principal, the relevant environmental agency and local government authority.

The electric cable providing power to the underdrain system pumps and the discharge hose should be submersible and of continuous length without jointing.

If the gravity discharge from the underdrain pit is possible, then the flow rate of the discharge system should be measurable and visible, to monitor the quantity and quality of the leakage.

The automatic reading of the flow rate with connection to SCADA would be preferred for existing assets, and mandatory for any new assets.

6.2 Embankment crest and toe drainage requirements

Stormwater should not be left to pond on the embankment crest and should be captured and discharged to the wider stormwater network off the site. On embankment crest roads adjacent batters, the road should be constructed of suitable material and camber to ensure stormwater drains freely across it and down the batter. Care should be taken not to concentrate the flow. Where concentration of flows cannot be avoided care should be taken to provide adequate erosion protection to the batter face.

Stormwater runoff should be collected at the toe of the embankment where necessary to prevent erosion of site access roads or damage to infrastructure.

7 Inlet, outlet, scour, and spillway requirements

7.1 General

Pipe materials are to be selected in line with SA Water TS 0522 and Authorised Items for Water Reticulation Systems. MSCL and Polyethylene should be considered as preferred due to their ability to take small amounts of movement without failure and the ability to form welded joints. Pipe joints are to be proved to a high level of certainty to minimise the risk of leaks at joints, for instance NDT of every welded joint prior to pressure testing. The use of GRP pipes or any types of pipes that require non-restraint joints is not permitted in any of dams or EBS assets.

Inlet or outlet structures are to be positioned no closer than 2 m to the toe of the embankment.

All pipework is to be graded at a minimum grade of 1% to aid draining of pipes. Where significant settlement is expected, this is to be considered for the final pipe grades.

Disinfection of all pipework and valving is to be undertaken as per usual SA Water new assets. Refer to WQ_G35: Code of Practice – Disinfection of Water Supplies and WQ_P034: Mains, Valves & Fittings – Disinfection.

7.2 Valve requirements

Accessibility of all valves and associated equipment is to be considered as part of the SiD process. Where possible, access points are to not be located within the embankment zone. Access points, pits and covers are to be installed to SA Water standards. Security of valves and especially security considerations for above-ground valving are to be included to meet SA Water standards.

Valves for throttling flow are to be resilient seated butterfly valves in accordance with SA Water's approved valve list. Isolation valves are to be gate valves. Where risk of backflow exists, a suitable non-return valve is to be included on the outlet.

Flowmeters should be included on both inlets and outlets wherever possible. Flowmeters are to be selected to meet SA Water standards.

7.3 Scour requirements

A scour system is to be provided where possible to enable complete emptying of the storage.

For the earthfill dams, the scour should be designed based on requirements of USBR (1990) "Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low Level Outlet works".

For the lined EBSs, the scour should be designed to enable emptying from minimum water level within 8 hours, or as specified by SA Water depending on the size of the storage and location specific requirements.

Scours must discharge to an approved location with appropriate erosion protection designed for the discharge of the full volume of the storage.

7.4 Spillway and overflow system requirements

For the lined EBSs, overflows are to be designed to pass a minimum of 120% of the maximum expected inflow rate, or as agreed with SA Water. Level sensors will trigger the full level alarm prior to the overflow operating with an appropriate allowance for operator attendance. The overflow must discharge to an appropriate area based on environmental and safety considerations.

The spillway system in the earth dams should be designed by a competent Dams Engineer, with required competency in design of hydraulic structures, to comply with ANCOLD requirements.

8 Shear Strength Parameters of Existing Dams

8.1 Background

The following section was prepared based on observations during the Hope Valley Dam Safety Upgrade in early 2000s. As part of that project, the shear strength parameters of the embankment were determined in the laboratory, in order to be used in slope stability assessments. SA Water still considers that the adopted approach in the Hope Valley Dam project is the best current practice for evaluating the shear strength of existing old earth dams, therefore those observations and recommendations are collected in this section.

The shear parameters that were eventually used in the analysis of Hope Valley Dam were obtained by carrying out triaxial tests on "reconstituted" triaxial samples prepared by blending bulk samples of "matrix" material selected from pits. The triaxial samples were compacted to in-situ density. In other words, the designers treated the dam simply as a "quarry" and followed the normal "design" process. The sections below provide the reasons that supported this approach, based on a SA Water Technical Note prepared in 2002.

8.2 Earth Dam Design – General Process

The design of the embankment of an earth dam is not a rigorous science, despite the fact that it includes apparently precise analytical steps at several stages. The process at a high level is as follows:

a. Identify potential borrow pits

b. Determine how much of each material there is available

c. Take representative samples of each material

d. Prepare specimens compacted to the densities to be specified for construction

e. Run triaxial tests to obtain saturated effective shear parameters

f. Design the embankment to fit the materials (geometry, zones, drainage, etc.)

g. Check slope stability (static, dynamic, rapid drawdown, etc.)

h. Repeat (f) and (g) as necessary until the "required" factors of safety are met.

Dams designed by this process, to the accepted factors of safety for steady seepage, earthquake, etc., are known to have a good chance of not failing. But this "success" is based more on the observation of the past performance of earth dams designed by this process and to these factors of safety, than it is on the accuracy of measurement of parameters and the rigorousness of the analytical models used, i.e. there is a lot more bundled into those factors of safety than there would be in say structural engineering.

What the design philosophy does NOT say is:

a. That the materials in the dam finished up with the same shear strength parameters as were obtained from the test samples,

b. That the materials in the dam will ever become saturated,

c. That the simplified models used in the stability calculations truly represent conditions in the dam (for example, most earth dams may behave three-dimensionally, not 2D plain strain as most stability analysis models assume), or

d. That the computed factor of safety is the "real" factor of safety.

Earth dam design must therefore be considered, in large part, to be an empirical ***process*** that depends for its validity not just on the assumptions inherent in each stage, but also on a standard ***sequence*** of steps being followed. This being the case, one is not free simply to jump into the design process at any step – the design process must be followed through.

Therefore, great care must be taken when evaluating an existing dam, as the temptation would be to (try to) measure the shear strength parameters of the material ***actually in the dam*** and plug them directly into the stability calculation models. Apart from the difficulty of obtaining representative samples – particularly from old "hand built" dams (more on that later) – such an approach would run counter to the need to follow the basic sequence.

It was probably just such a philosophical misconception that led to early EWS triaxial shear strength tests, on samples recovered from existing dams, being run at their in-situ degree of saturation. The logic was that these results would be the *true* shear strength parameters of the material in the dam. This may be so, but unfortunately it would seem that the design process does not require the *true* shear strength parameters, only the *design* ones.

In summary, all triaxial tests on material from existing dams should be run saturated. Only remoulded shear strength parameters (not "aged" in situ ones) should be determined and used in the slope stability analysis.

8.3 Variability of Materials in Old Dams

Most of the large dams in SA Water portfolio were built in the late 1800s or early 1900s. The design philosophies and construction techniques of the time meant that the materials tended to be won from several different sources simultaneously, especially for the downstream shoulders, and were placed in a similarly random fashion cartload by cartload.

Based on that background, the most obvious feature of the trial hole logs for the downstream shoulders in such dams is, not surprisingly, their variability.

This variability can continue down to the scale of individual triaxial samples -- a typical description of a sample being "mixture of sand, clay and stone, multicoloured, moist, firm".

Variability – in both the materials themselves and in the properties of those materials – is so much the dominant characteristic of these dams, that when preparing the report on the field work and laboratory testing for Happy Valley Dam, it was considered prudent to present the data in such a way as to visually illustrate this variability rather than to try to give an estimate of typical values for the various parameters.

In this way, subsequent users of the data would be forced to be aware of the variability and of the assumptions they were making when they themselves attempted to extract "typical values" for stability analyses, etc.

In summary, the analytical methods used in earth dam design do not want "in situ" shear strength parameters they want "design" (i.e. remoulded) values, and furthermore the variability in old dams prevents the recovery of representative samples. It would therefore seem that a program of sophisticated triaxial testing on undisturbed samples (particularly if won from the downstream shoulder of an old dam) is not justifiable.

8.4 Conclusions

All of the foregoing discussion leads to the conclusion that the best way to check the stability of an existing dam is to follow the same procedure as would be used for the design of a new dam. In other words, simply treat the existing dam as a "quarry", and then follow through the design procedure. This means using reconstituted remoulded material for determining shear strength parameters for existing dams, and would involve the following steps:

1. Study any original design drawings and construction records, and the logs of all trial holes and (ideally) pits, to determine how the dam was zoned and what the typical "matrix" material is in each zone.
2. Assess or determine the in-situ density of each typical material/zone.
3. Obtain several bulk samples of each typical "matrix" material, and blend and screen them as required for the preparation of triaxial test samples. Ideally these should be won from pits but could be obtained from samples recovered from the trial holes.

4. In compaction moulds, compact the blended material to the estimated/observed in-situ densities.

5. Cut triaxial and/or direct shear test specimens from the compacted samples and carry out the triaxial and/or direct shear tests.

9 Seismic analysis of earth dams

As per ANCOLD guidelines, a number of methods are available for assessment of seismic stability and estimating the seismic deformations of embankments and their foundations during and post-earthquake events. These methods, which are only applicable to the dams when they are not subject to any potential for liquefaction in the dam body or their foundation, are summarised in following sections.

9.1 Screening and empirical database methods

These are applicable to embankments and their foundations that do not liquefy or experience significant loss of strength, either due to the build-up of pore pressure, or strain weakening. These methods can only be relied upon if the estimated deformations are much less than the available freeboard.

ANCOLD (2019) allows the use of the pseudo-static method as a screening tool. Based on the studies by Hynes-Griffin and Franklin (1984), the procedure for undertaking the pseudo-static seismic assessment would be as follows:

1. Conduct a conventional pseudo-static stability analysis using a seismic coefficient equal to one half of the predicted peak ground acceleration.
2. Use a composite S-R strength envelope (effective stress strength at low stresses; undrained strength at high stresses) for pervious soils and R undrained strength for clays, multiplying the strength in either case by 0.8.
3. Use a minimum factor of safety of 1.0.

A few empirical relationships have been proposed to predict the crest settlement of earth and rockfill dams subjected to earthquake loading. These relationships have been formulated based on the responses of existing dams after earthquakes. Amongst the first contributions in this area is the relationship proposed by Jansen (1990) as:

$$\Delta = \left[48.26\,(\frac{M}{10})^{8}\,(k_m - k_y)\right] \Big/ \sqrt{k_y}$$

where Δ is the deformation of the dam crest under an earthquake loading of magnitude M, k_m is the maximum induced acceleration at the crest, and k_y is the yield acceleration of a potential sliding mass; that is the horizontal acceleration which results in a factor of safety of unity for the sliding mass and can be obtained by appropriate methods such as limit equilibrium analyses.

Swaisgood (1998) proposed relationships for evaluation of the crest settlement of embankments based on the responses of 54 dams. Swaisgood (2003) used a larger database based on the performances of 69 dams and presented an equation to calculate the seismic settlement of dam crest:

$$S(\%) = e^{(6.07\,PGA + 0.57\,M - 8.00)}$$

where S is the relative settlement of dam crest, as a percentage of the total height of dam and its alluvium foundation, and PGA is the peak ground acceleration.

9.2 Simplified methods for estimating seismic deformations

These methods are also only applicable to embankments and their foundations that do not liquefy. They assume that the post-earthquake deformations are negligible, and the deformations during the earthquake are due to the action of the horizontal inertia forces induced by the earthquake. These methods are commonly based on the Newmark (1965) principle.

Newmark (1965) proposed a method for evaluation of deformation of slopes and embankments under earthquake dynamic loading. This method, which became the base of what are known as the simplified methods, is based on the assumption that the behaviour of a potential sliding block of an embankment under earthquake loading is similar to a sliding mass on an inclined surface. An earthquake loading may cause the block to slide if its acceleration becomes larger than the yield acceleration of the block, k_y. The yield acceleration of a potential sliding block is a horizontal acceleration which results in yielding (or failure) of the block with irrecoverable deformation. Only if the acceleration induced by an earthquake becomes larger than the yield acceleration, permanent displacement of the block could occur. Assuming that the record of the earthquake induced acceleration on a block is known, the displacement of the block can be derived by double integration of the earthquake acceleration record exceeding the yield acceleration of the block.

Makdisi and Seed (1978) modified and improved the original Newmark's method by including the effects of dam deformability during earthquakes. They evaluated the variation of the induced acceleration along the dam height approximately as a function of the crest acceleration. Makdisi and Seed (1978) also evaluated the deformation of potential sliding blocks as a function of the dynamic properties of the dam and the earthquake. The dynamic properties of the dam, in terms of the maximum crest acceleration, $\ddot{u}_{max}$, and the fundamental period, T_o, would be required in this method. Makdisi and Seed (1979) proposed a simplified method to calculate $\ddot{u}_{max}$ and T_o.

The original Newmark (1965) method and the one modified by Makdisi and Seed (1978) used to be known as conservative estimates of deformation of embankment dams under earthquake loading. Results of some recent studies show that simplified Newmark-type methods may not always be conservative. In a recent investigation, Meehan and Vahedifard (2013) compared the predictions of fifteen Newmark-type simplified methods with the displacements records of 122 earth dams and embankments under seismic loading and showed that the results of the simplified methods are not always conservative. The displacements predicted by some of the methods were less than the observed deformations, with differences as high as 1 m for some cases. Kan et al. (2017) sowed that amongst 15 selected simplified methods, only the Bray and Travasarou (2007) method was able to conservatively demonstrate the observed deformations of a large rockfill dam due to strong seismic loading.

ANCOLD (2019) states that the simplified methods should not be assumed as conservative approaches and therefore they cannot always be used as screening tools. Kan et al. (2017a&b) showed that in stable continental regions such as Australia, for embankment dams higher than 20m, or rockfill dam higher than 30m, the use of simplified methods might be non-conservative. In active seismic regions, these thresholds are 60m and 75m, respectively.

In line with these observations, the use of simplified methods in SA Water earth dams is only permitted provided that all the following conditions are met:

- No liquefiable materials are present in the dam or its foundation
- At least two and preferably three simplified methods are used in evaluations and the largest calculated crest deformation is adopted as the outcome. The recommended approaches are as follows:
  - Makdisi and Seed (1978), with calculation of the crest acceleration and the fundamental period of the embankment based on Makdisi and Seed (1979)
  - Bray and Travasarou (2007)
  - Bray et al. (2018)
- The calculated crest settlement is less than half of the available freeboard of the dam.

9.3 Advanced numerical methods

These methods may have a wide range of complexity. The numerical stress-strain models include dynamic analyses using total and effective stress methods and non-linear models. The way that pore water pressures are considered in the models and coupled with the stresses and deformations is also an important factor in these methods. FLAC and PLAXIS can be used in advanced numerical methods, with advanced constitutive models that allow for degradation of materials due to cyclic shearing, and generation and dissipation of pore water pressure.

9.4 Liquefaction-induced displacements of embankments

Simulation of the behaviour of geo-structures subject to liquefaction-induced displacements has always been challenging. Many numerical schemes fail to predict reasonable displacements when the flow-failure due to seismic loading occurs in soil in the field, although they may show good responses in the simulation of simple laboratory test specimens. A robust constitutive model in a fully coupled numerical simulation of seismic loading is a powerful tool for the analysis of geo-structures. Kan (2019) presented the use of an advanced bounding surface model and its implementation in FLAC to perform such analyses. It was showed that flow failure liquefaction can be captured successfully by numerical models, when a constitutive model capable of representing the cyclic behaviour of materials is applied with representative parameters and residual strengths associated with flow failure are separately accounted for in the analysis, if not included in the cyclic constitutive model already.

9.5 References for more details

The following table shows the list of refences that were used to prepare this section. More information should be sought from these references during the design.

Table 7: More references for seismic assessments

Reference	Title
ANCOLD (2019)	Guidelines for Design of Dams and Appurtenant Structures for Earthquake (2019)
Bray and Travasarou (2007)	Bray, J., and Travasarou, T. 2007. Simplified procedure for estimating earthquake-induced deviatoric slope displacements. Journal of Geotechnical and Geo - environmental Engineering, 133: 381–392.
Bray et al. (2018)	Bray, J. D., Macedo, J., and Travasarou, T. 2018, Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes, Journal of Geotechnical and Geo-environmental Engineering, 144 (3)
Hynes-Griffin and Franklin (1984)	Hynes-Griffin, M., and Franklin, A. 1984. Rationalizing the seismic coefficient method. Miscellaneous paper GL-84-13. U.S. Army Corps of Engineers Water-ways Experiment Station, Vicksburg, Miss.
Jansen (1990)	Jansen, R. B. 1990. Estimation of embankment dam settlement caused by earthquake. International Water Power and Dam Construction, 42(12), 35-40.
Kan et al. (2017a)	Kan, M. E, Taiebat, H. A., and Taiebat, M. 2017. Framework to assess Newmark-type simplified methods for evaluation of earthquake-induced deformation of embankments, Canadian Geotechnical Journal, 2017, 54(3): 392-404.

Kan et al. (2017b)	Kan, M. E, Taiebat, H. A., and Taiebat, M. 2017. Seismic performance of existing and new embankment dams: the myth of the reliability of simplified Newmark-type methods, ANCOLD Conference, Hobart, TAS, October 2017
Kan (2019)	Kan, M. E. 2019, Liquefaction-induced displacement of embankment dams: How good we are in predicting the post-earthquake displacements using numerical models?, ANCOLD/NZSOLD Conference, Auckland, NZ, October 2019
Makdisi and Seed (1978)	Makdisi, F. I., and Seed, H.B. 1978. Simplified procedures for estimating dam and embankment earthquake induced deformations. Journal of the Geotechnical Engineering Division, ASCE, 104: 849–867.
Makdisi and Seed (1979)	Makdisi, F. I., and Seed, H.B. 1979. Simplified procedure for evaluating embankment response. Journal of the Geotechnical Engineering Division, ASCE, 105:1427–1434.
Meehan and Vahedifard (2013)	Meehan, C.L., and Vahedifard, F. 2013. Evaluation of simplified methods for predicting earthquake-induced slope displacements in earth dams and embankments. Engineering Geology, 152: 180–193.
Newmark (1965)	Newmark, N. M. 1965. Effects of earthquakes on dams and embankments. Géotechnique, 15: 139–160.
Swaisgood (1998)	Swaisgood, J. R. 1998. Seismically induced deformation of embankment dams. Proceedings of the 6th US National Conference on Earthquake Engineering, Seattle, Washington.
Swaisgood (2003)	Swaisgood, J. 2003. Embankment dam deformations caused by earthquakes. In Proceedings of the 2003 Pacific Conference on Earthquake Engineering, Christchurch, N.Z.